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Timber Bridges

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Contents

An Overview of Timber Bridges

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ABSTRACT

This paper contains a review of literature on timber bridges. It presents recent developments and evolving concepts in timber bridge technology, including aspects concerning wood material, bridge design, construction, inspection, rating, and maintenance. This review indicates that timber bridge technology has advanced with the design and construction of a prototype of a prestressed, laminated timber bridge in Ontario, Canada. In the United States, the main effort of government and the timber industry is to promote timber bridge technology transfer.

Timber was probably the first type of material that humans used to construct a bridge. Although concrete and steel replaced wood as the major materials for bridge construction in the 20th century, the use of wood in short-span bridges remains as great as ever. Of United States bridges that have a span of more than 20 ft (6 m), 12.6 percent (or 71,200) are made of timber. In the Forest Service (U.S. Department of Agriculture) alone, approximately 7,500 timber bridges are in use, and more are being built each year. The railroads have more than 1,500 mi of timber bridges and trestles in service. In addition, timber bridges recently have attracted considerable attention from many international organizations and foreign countries, including the United Nations, Canada, England, Japan, Kenya, and Honduras (1-3).

Timber is a highly desirable raw material because it is an abundant renewable resource. It has several advantages as a material for bridge construction. Timber bridge structures present a natural and aesthetically pleasing appearance, particularly in wooded surroundings. The timber sections can be constructed in any weather, including cold and wet conditions, without experiencing detrimental effects. Timber bridges cannot be damaged by continuous freezing and thawing and are resistant to the effects of deicing agents. Because of wood's energy-absorbing ability, timber bridges are also able to sustain overloads for short periods of time. The light weight of timber allows for easier fabrication and construction since smaller equipment is needed to lift the beams into place. A timber bridge's light weight also benefits repair and rehabilitation efforts including superstructure replacements because abutments can be reused and the available load-carrying capacity of the remaining existing structure can be increased. Initial and maintenance costs of timber bridges are lower than for most other alternatives and are certainly competitive with the materials usually considered to be best (4). For example, a prestressed, treated timber bridge costs only two-thirds of its counterpart constructed with conventional steel and concrete (5).

Wood does have several shortcomings as a bridge material. First, because wood is a biological material, it is vulnerable to damage by fungi, fire, accidents, and insects. Second, the deeper beam sections may significantly reduce the hydraulic operation, reducing the flood flow capacity beneath the bridge. Third, the fabrication of glued-laminated (glulam) timber members may take longer than the construction of steel beams or concrete sections (6); however, a newly developed prefabricated modular system of production may eliminate this time delay (3).

Several studies have summarized the results of research on timber bridges. The first study was performed by the American Institute of Timber Construction in 1973 (7). This study summarized the significant advances in the engineering and construction of timber bridges that occurred in the 1950s and 1960s. The Committee on Wood in the Structural Division of the ASCE made the second attempt in 1975 (8). The ASCE endeavor compiled an extensive bibliography on timber bridge design and classified a set of selected standard specifications into primary and supplementary criteria for both glulam and wood bridge members. The third study, conducted in 1980 by the ASCE Technical Committee on Timber Bridges, presented an excellent summary on the state of the art of timber bridges and included discussions of the development of new wood composite products and manufacturing techniques, the improvements in preservative treatment methods for resisting decay, the development of new wood bridge system concepts, and the advancements in timber bridge design and analysis techniques (9).

Manufacture and design were a major concern of these reports on the state of the art of timber bridges. The studies overlooked other aspects, however, such as inspection and maintenance. Given the safety requirements under the law, methods for inspecting and maintaining a safe timber bridge are as important as a cost-effective design. The present study will review the literature on timber bridges and emphasize all the important aspects of timber bridges, including wood materials, bridge design, construction, inspection, rating, and maintenance. It is hoped that this study will enhance the transfer of technology in the use of timber bridges.

WOOD AS A CONSTRUCTION MATERIAL

One of the major concerns in using wood as a bridge building material is strength. Because wood is orthotropic, its strength properties are different in different directions--that is, longitudinal, radial, or tangential to the grain or axis of fiber orientation. Wood strength is greatest in the longitudinal, or parallel-to-grain, direction, and weakest across the grain. This strength varies among tree species (10). In addition, growth variations, defects, and manufacturing processes also affect

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strength significantly. Therefore, the strength properties of each individual board are determined by a number of wood characteristics, including slope of grain, knots and their locations, pitch, wane, density, checks or splits from uneven drying, and size variations (11).

Wood Degradation

Because wood is a biological material, it is subject to seven types of degradation: decay-causing fungi, wood-destroying insects, marine borers, discolorations, weathering, chemicals, and fire.

Decay-causing fungi produce spores that develop into very small, threadlike hyphae that spread through the wood in all directions. A favorable environment for the fungi requires four factors: (a) available moisture, (b) adequate air, (c) favorable temperature, and (d) suitable food--the wood (11). The resulting decay causes losses of density and strength, and increased permeability in the wood. The loss of strength results from the enzymatic degradation of the wood cellulose and lignin. The decayed wood rapidly loses its toughness, or capacity to withstand loading, and its resistance to bending and crushing. For example, the Forest Service's Forest Products Laboratory conducted bending tests on stringers from 12-year-old native timber bridges in Southeast Alaska and found that the strength of the decayed logs was 25 percent lower than that of fresh logs (12).

Another type of degradation, weathering, is affected by light, water, and heat. Weathering can change the equilibrium moisture content and result in changes in the strength of wood and its dimensions. As the moisture content falls below the fiber saturation point, wood shrinks. A 12-percent change of moisture content in a 12-in. piece of wood can result in a shrinkage of .25 in. Shrinkage leads to warping, checking, and splitting, which can cause connectors to loosen and reduce bridge capacity.

One of wood's distinguishing characteristics, however, is its good energy-absorbing properties. This enables the wood structure to sustain overloads for short periods of time. However, the wood's strength will decrease over time as a result of the degradation.

Wood Preservation

Chemical preservation of wood is often used to maintain the material in serviceable condition. Two main preservation methods consist of brushing, spraying, or dipping and vacuum-pressure processes (13). Brushing, spraying, or dipping provide shallow penetration, low chemical absorption, and superficial treatment of the wood surface. Vacuum-pressure processes penetrate the wood more deeply. They consist of the Bethel, Lowry, and Rueping processes, each of which uses timed pressure and vacuum treatments. Vacuum-pressure processes are referred to as full- or empty-cell processes depending on the amount of preservative solution or toxicant that remains in the wood after treatment (13). The Bethel process is a full-cell process and is applied to products such as marine piling where maximum protection is required; both the Lowry and Rueping methods are empty-cell processes and provide good distribution with a limited amount of preservative. The Rueping process is more flexible and can be used to achieve a wider range of results than the Lowry process. However, the Lowry process is simpler to perform, particularly when the wood is readily treatable.

Three groups of preservatives are used for wood treatment--creosote, pentachlorophenol (penta), and water-borne salts. Although each compound or mixture of compounds has unique characteristics, all preservatives must have the following properties:

1. They must be toxic to organisms that degrade wood, and they must have some degree of permanence.
2. They must be capable of being forced into the wood by pressures of one atmosphere or greater.
3. They must not be unduly flammable or explosive, highly poisonous to man or animals, or have an undesirable color or odor.
4. They must not be corrosive to metals.
5. They must be easy to detect by standard assay methods.

Chemical treatment makes wood resistant to most agents of deterioration, fire, swelling, and shrinking. For instance, either pressure impregnation or fire retardant wood coatings such as phosphates, zinc chloride, boric acid, and so forth, will produce some reduction in flame spread. Fire retardants may increase ignition temperatures or reduce the tendency of the wood to catch fire or to glow after flaming has ceased. In addition, treated wood cannot be damaged by continual freezing and thawing and is not affected by temperature, alkali soil, or acids. Although using water-borne preservatives and fire retardants may harm wood's stiffness and bending strength, this damage should be insignificant if modern treating practices are followed.

The preferred preservatives are creosote or penta in heavy oil because they provide a more uniform moisture content over a longer period of time. Creosote treatments are used to protect deck panels and stringers, while penta in heavy oil is a commonly specified treatment for both stringers and deck panels. The water-borne salts or penta in light solvents are better suited for bridge components, such as traffic rail elements, that may come into human contact. In temperature climates, creosote treatments can extend the serviceable life of wood considerably (14). Thus, the life expectancy of treated timber bridges can be as high as 50 years under normal conditions (6,7).

Summary

Today, much is known about wood, yet its propensities are not fully exposed. Given its many advantageous characteristics, treated wood is a good source for construction materials. Research on its cost-effective use is important and is continuing (11,13, 15-18).

DESIGN

U.S. timber bridge design follows the requirements of Section 13, Timber Structures, Division I, Design, of the thirteenth edition of the Standard Specifications for Highway Bridges (19), published by AASHTO. The AASHTO design specifications are adjusted or amended by state designers or Forest Service bridge engineers to fit local conditions based on new experience or research results (20). The AASHTO specifications include a list of allowable stresses for stress-grade lumber and glulam timber under normal loading; permanent loading; and wind, earthquake, or short-term loading conditions. The specifications provide formulas for the computation of stresses, as well as detailed design procedures for simple columns, spaced columns, pile and framed bents, and trusses. The Forest Products Laboratory also has documented in detail a design procedure for glulam bridge decks (21-23) that was adopted by AASHTO and included in the bridge specifications.

The AASHTO design code does not address the design of log bridges for trails and roads. The Forest Service often uses untreated logs for temporary structures when short-term (5 to 10 years) needs justify use. However, some of these "temporary" structures are up to 25 years old. Glulam bridges also have been designed and constructed for temporary structures in cases in which the load duration characteristics of wood (time dependency; that is, change in strength over time of use) were a consideration (24). Logs used for road bridges range in diameter at the tip from about 24 to 40 in. (61 to 102 cm). The bridge spans reach up to 80 ft (24.4 m). Although the loading varies in road designs, the minimum loading for the Forest Service is the AASHTO HS 20-44 truck. Markedly higher loads are used for logging roads. In Forest Service engineering practice, loadings for trail bridges include hikers, livestock, motorcycles, snowmobiles, and sometimes, four-wheel-drive vehicles. The usable width of trail bridges varies from 3 to 8 ft (0.92 to 2.44 m) (25).

Timber Bridge Design

Traditionally, the deck design has involved a nail-laminated assembly of nominal, 2-in. (5-cm) dimension lumber placed transverse to the supporting stringers. Connections include through-nailing of laminations and toe-nailing to the stringers. The major shortcoming of this system is that nail connectors gradually loosen as a result of the deck deflections and the shrinking and swelling caused by repeated wetting and drying cycles. Because of this problem, the deck cannot serve as a roof over the entire bridge structure, protecting the supporting members and the deck itself from the deteriorating effects of rain and snow.

Glulam Bridge Design

To overcome the deficiencies of the nail-laminated deck and to increase deck stiffness, the composite timber-concrete deck was designed and constructed in the early 1930s (9). In the 1950s, an effort was made to examine the glulam concept in terms of the effect of knot size and distribution of glulam timbers (26). Glulam is an assembly of individual wood laminations bonded together with structural adhesive. In the early 1960s, the Forest Products Laboratory further developed the glulam technique, introducing prestressed glulam (27-29). In 1976, the Ontario Ministry of Transportation and Communications used transverse prestressing for rehabilitating existing nailed decks (30). In 1979, the possible use of prestressed glulam for wood bridges was further examined by the U.S. researchers (31).

In the late 1960s and early 1970s, the Forest Products Laboratory and the glulam industry continued their research on prestressed glulam, evaluating the concept of increasing beam strength by increasing the grade requirements for the tension zone and defining the grade requirements for fabricating glulam members using lumber that has been both E-rated as well as visually graded (32). The results of this research appear in the 1973 version of AASHO design specifications (32) and in "Standard Specifications for Structural Glued-Laminated Timber of Douglas Fir, Western Larch, Southern Pine, and California Redwood" p lished by the American Institute of Timber Construction (33). In the late 1970s and early 1980s, the Ontario Ministry of Transportation and Communications also conducted considerable research on the design of prestressed wood decks (30,34), which resulted in a comprehensive set of design specifications (35,36). These new specifications were developed using probability-based methods of timber stress analysis and have been included in the 1983 edition of the Ontario Highway Bridge Design Code (36).

However, the analytical methods used in the specifications for AASHTO and for the Ontario design code differ. AASHTO uses allowable stress to design and evaluate timber bridges, while the Ontario design code uses probability-based methods as an analytical tool (37). The AASHTO approach does not provide for the determination of the actual safety reserve in the structure (38). However, developing a comprehensive probability-based design code would require considerable research because of the current lack of a good data base on the mechanical properties of wood (39,40).

BRIDGE LOAD-CARRYING CAPACITY

An important aspect of bridge design is analyzing the adequacy of the bridge structure to carry the expected traffic load. Nine independent elastic constants are used in the analysis: three moduli of elasticity, three shear moduli, and three Poisson's ratios. However, because of large variations in the longitudinal modulus of elasticity of wood, actual values of dead and live load moment and shear stresses in timber bridges are difficult to determine. Many methods have been used to estimate the longitudinal modulus of elasticity, such as the orthotropic plate theory (21,30,41,42), the Grillage analogy (43,44), the AASHTO simplified approach (19,45), the statistical approach (46), and probability-based methods (38). These methods were developed based on various assumptions. For example, in the orthotropic plate theory, the bridge structure is considered to be a plate of uniform thickness that has different flexural and torsional properties in two orthogonal directions. In the grillage analogy analysis, bridges are considered to be idealized grillages. The analytical procedure is similar to that used in nontimber bridges, except for the discretization of the structure and the calculation of the properties of idealized members. The statistical approach is to calculate mean values of longitudinal moments and shears and their coefficients of variation by assuming a uniform value for the longitudinal moduli of elasticity of various longitudinal members. Probability-based methods use several mathematical models to compute reliability indices, for instance, using the Rackwitz and Fiessler procedure for single beams and Monte Carlo simulations for whole bridges. This method allows the investigation of a structure's limit states (47).

Overload behavior is a major concern in the timber bridge design (48). In several cases, overloading caused shear failures in laminated-timber bridge girders (49). Some early investigations of load distribution on timber bridges centered on structures with decks and timber girders (50-54), while others concerned wood floors on steel joists (55) or timber deck and steel girders (56). The Forest Products Laboratory and the FHWA conducted tests on the strength of log bridge stringers and piles (13, 57-61).

In the past two decades, researchers have investigated the behavior of layered wood systems. In the late 1960s, glulam wood was treated as steel and reinforced concrete material and analyzed for stress based on an assumption that the material properties of the individual layers are constant along the entire span (62,63). In the 1970s, researchers studying glulam bridges applied a finite-element approach, based on the principle of minimum potential energy (64, and an unpublished analysis of stiffened deck panel systems by Weyerhauser Company, Tacoma, Washington, in 1977), to define key factors such as gaps between individual deck panels and interlayer slip at the deck-stringer interface that have significant effects on the system performance (65). The result of this research indicated that the composite design of glulam bridges can add strength and stiffness to working loads because layered systems perform at maximum structural capacity if the individual components interact as a single unit. Thus, the finite-element approach is becoming an important tool for evaluating the deck systems performance (66).

A recent study conducted by the researchers at Iowa State University verified this finding and recommended that the existing load distribution criteria for glulam longitudinal deck bridges as shown in the current AASHTO bridge specifications (19) be updated according to the distribution behavior defined by the finite-element method (67). In this recent study, finite-element analysis was used to develop an analytical model for studying the load distribution behavior of glulam longitudinal deck bridges.

CONSTRUCTION

The thirteenth edition of AASHTO Standard Specifications for Highway Bridges (19) sets forth the requirements for constructing glulam timber bridges. The specifications cover materials; timber connectors; holes for bolts, dowels, rods, and lag screws; pile and frame bents; nail-laminated or strip floors; wheel guards and railing; and so forth. In United States practice, some of these specifications apply to other types of bridges, such as log bridges and prestressed glulam timber bridges. However, the construction procedures for log bridges and prestressed glulam timber bridges differ from those for glulam timber bridges. Therefore, the AASHTO standard specifications are adjusted or amended by bridge engineers in state and forest service based on the local environment (20,68).

Log bridge construction is simple. The logs are normally greater than 10 in. in diameter and are all as nearly equal in size as possible so that they will uniformly support the 2-in.-thick flooring. They are placed perpendicular to the stream channel. The number of logs varies from two for wheel-tracking spacing to three or more equally spaced for a solid bed of logs. In some cases, dirt fill or gravel is used to cover the plank flooring, which permits smoother truck crossing. Depending on the size of the bridge, constructing a log bridge may require a two- to three-man crew for a period ranging from 2 days to 2 weeks (69).

The fabrication and construction of the longitudinal, laminated-deck timber bridge is also simple (26,70), requiring a small crew consisting of a carpenter foreman, a machine operator, and one or two laborers. The construction does not require much sophisticated equipment or many highly skilled workers (71).

Although the concept of prestressed timber bridges has been studied for 20 years, it was not until 1981 that the world's first bridge of this kind was constructed--in Ontario, Canada (5). The Ontario experience indicated that constructing a 37-ft prestressed timber bridge took several weeks with a crew consisting of three experienced construction workers and three supplemental laborers. In this project, all steel hardware was hot-dip galvanized for protection. All wood materials were cut and drilled before undergoing pressure preservative treatment with creosote, except for the holes in the deck that were drilled on site.

The process for the fabrication of the frame geometry used in Ontario differs from the prestressing routine used by timber craftsmen (72). It enables the prestressing of the entire bridge to be done at the same time and allows the legs and deck to be constructed as an integrated, prestressed, laminated system. The Ontario experience also indicates that the cost of prestressed, treated timber bridges is one-third lower than that for conventional steel and concrete bridges.

A recent investigation has found that a weak glue bond may fail in shear. The weak glue bond is caused by a premature curing of the glue before final clamping. When a girder is assembled during hot, dry weather, the extended "layup" time caused by its large size may cause this effect (49).

INSPECTION

Federal and state legislation require that highway and railroad bridges periodically be inspected, evaluated, and rated as to their safe load-carrying capacity. Inspections cover physical and mechanical properties of timber, such as strength, porosity, anisotropy, impact resistance, durability, fire resistance, and so forth. The purpose of the inspections is to ensure the early detection of damage or deterioration and to prevent structure failure (73-75).

Inspections are also important in developing economic plans for bridge replacements or rehabilitations. Bridge engineers, therefore, need reliable data for assigning priorities and scheduling maintenance on timber bridges. These data come from tests conducted on a bridge during an inspection. The techniques used for testing fall into two categories: destructive and nondestructive. Destructive testing includes probing and core sampling. Nondestructive testing consists of visual inspection, sounding, radar, ultrasonic technique, infrared thermography, microseismic survey, resistivity survey, and electronic potential. They are described below.

Destructive Testing Methods

Destructive testing methods can be classified into probing and bore or core sampling. The probing sampling approach detects external decay using a pointed tool, such as an awl, an ice pick, or a prospector's rock pick. The bore or core sampling method detects and defines the limits of internal decay using such instruments as an electric drill or an increment borer.

These destructive sampling tests can impose undue strain on the tested members and increase the loss of cross-sectional areas at the location where the test takes place. Impairing the usefulness of wood material is the major disadvantage of destructive testing methods. Another shortcoming of these sampling techniques is the assumption that the tested pieces represent the entire population not tested. If the sample is not truly representative, then the results of the test are not accurate (76).

Nondestructive Testing Methods

Any inspection technique that does not impair the usefulness of the material under examination is categorized as a nondestructive method. Visual inspection, sounding, radar, ultrasonic technique, and infrared thermography are discussed in the following paragraphs. Other methods, such as microseismic survey, resistivity, and electronic potential, have been detailed in other reports (77,78). Some of these nondestructive methods are applied to timber and others may have potential for the application.

Visual Inspection

Visual inspection of timber bridges is based on two groups of visual indicators (79). The first group includes three visual indicators of the presence of decay: (a) characteristic fungus fruiting structures, (b) abnormal surface shrinkage or sunken faces, and (c) insect activity. The second group of visual indicators is used to identify six conditions conducive to decay. The indicators are

1. Excessive wetting (evidenced by water marks or stains);
2. Rust stains on wood surfaces;
3. Growth of vegetation on bridge members;
4. Accumulation of soil on any wood surfaces, which can trap water and increase decay hazard;
5. Joint interfaces, mechanical fasteners, field fabrication, and wood adjacent to other water-trapping areas, which are potential sites of decay fungi growth; and
6. Water-catching seasoning checks in exposed wood faces. Of these, the first condition is probably the most common and noticeable indication of the development of decay.

The advantage of visual inspection is its simplicity and quickness. However, it depends a great deal on the inspecting engineer's judgment. This method has two major limitations. First, it is less accurate for inspecting internal decay; for example, a timber pile may be completely decayed even though the external material appears sound. Second, members immersed in water or covered by asphalt or concrete are difficult to inspect. However, a newly developed photographic technique could overcome the difficulty of inspecting underwater bridge components (80). The equipment allows divers to obtain clear, underwater photographs under typical inspection conditions.

Sounding

The sonic technique has been used to locate the relatively severe delaminations of concrete bridges by monitoring the audible sounds that result from striking the deck surface of a bridge. Sounding relies on subjective judgments of hollow sound that are produced when concrete is struck with a hammer or when a chain is dragged across the deck surface. Interference from traffic noises may result in an inaccurate judgment (78,81). To overcome this difficulty, the Delamtec, a delamination detector, may be used to monitor the sound. The Delamtec uses piezoelectric hydrophones to characterize vibration waves generated by steel tapping wheels striking delaminated areas. This device is faster than the hammer or chain drag and provides a graphic record of distressed areas.

The use of sonic instruments, which were developed in the 1960s, to inspect piles of timber bridges is still in the experimental stage (82). In the simplest version of the sounding method, an inspector strikes a wood member with a rock pick and listens to the sound produced. A dull or hollow sound may indicate internal decay (79). The approach is economical, but its accuracy is much in doubt. The application of sonic techniques to covered timber decks has not been reported in the literature. One shortcoming of sounding is its inability to identify the areas of debonding (78,83).

Radar

A pilot test using low-power, high-resolution, ground-penetrating radar (GPR) to detect deterioration in concrete bridge decks was performed in 1977 (84). After this pilot test, additional work was done to improve the accuracy of the technique (85-87).

The GPR system consists of a monostatic antenna, a control console that contains a transmitter and receiver, and an oscilloscope. This system transmits impulses of microwave frequency into an overlaid concrete bridge deck and detects the extent of deterioration based on the reflection from the surfaces of the bituminous concrete, the portland cement concrete, and the reinforced concrete slab. When the concrete slab is delaminated, there is an additional reflection from the deteriorated area. The more severe the delamination, the more pronounced is the resulting reflection.

Recent studies indicate that radar can be used to survey the condition of overlay decks and decks that have their original surfaces (78,83). Although the speed and accuracy of the technique need improvement, the experiment of concrete bridges indicates that radar can be a potential tool for inspecting timber bridges. It should be noted, however, that like other nondestructive techniques, radar is not adequately capable of identifying the areas of debonding. Also, the radar technique requires further development to automate data collection and analysis and to improve the interpretation of signals.

Ultrasonic Technique

Ultrasonic technique has been used to detect defects and to measure the strength of concrete for many years (88-93). The technique involves the use of propagated high-frequency sound waves to test materials based on the relationship between the wave velocity in a material and its properties. For example, undamaged wood is an excellent transmitter of these waves, whereas damaged and decayed wood delays transmission. The technique, therefore, requires accurate measurements of the velocity of the propagated stress wave.

A recent study (76) indicated that the velocity of propagation of the waves parallel to the grain V_L and in the radial direction V_N (normal to the grain) in an orthotropic material with a Poisson's ratio for transverse strain in the longitudinal direction, when stress is applied in the radial direction, in the range of 0.01 to 0.04, is approximately

$$V_L = (E_L/p)^{1/2}$$

and

$$V_N = (E_N/p)^{1/2}$$

where E_L and E_N are the dynamic modulus of elasticity in the longitudinal and radial directions, respectively, and p is the material mass density. Factors affecting the velocity of waves include type of wood, effect of treatment, direction of grain, density of wood, degree of decay, and moisture content.

In the late 1960s and 1970s, the Forest Products Laboratory made two efforts to apply the ultrasonic scanning technique to detect defective wood (94,95). Although the results were encouraging, these two applications were merely experimental. It was not until 1984 that an ultrasonic wave propagation method was used to test wood bridge piles in Maryland (76). The results of the Maryland test were satisfactory. The difference between the calculated average crushing strength and the actual measured strength was about 11 percent. The study indicated that the ultrasonic technique can determine if damage is occurring or has occurred and to what extent, thus enabling the inspector to predict the true performance of the pile.

In contrast, the application of ultrasonic technique to detect the deterioration of asphalt-covered concrete decks has yielded no meaningful results because of unknown path lengths in three different materials--the bituminous surfacing, the concrete deck slab, and the reinforcing steel (78). This problem may also exist in an overlaid timber bridge. Therefore, this technique's use on asphalt-covered timber deck inspection requires further research.

Infrared Thermography

The principle of using infrared (IR) thermography as a means of defining delaminated areas caused by corrosion of the reinforcing steel in concrete bridge decks is based on the detection of differences in the surface temperature between the sound concrete and the delaminated concrete. The concrete emits IR radiation generated by the vibration and rotation of its atoms and molecules. From an IR scanner, the temperature can be determined indirectly by measuring the intensity of the emitted IR radiation.

IR thermography has proven to be a faster and more effective method than conventional sounding methods for surveying the extent of delaminations in concrete decks that have not been overlaid with bituminous concrete (96,97). For overlaid concrete bridge decks, the application of an IR scanner is still in the experimental stage. However, two recent studies have shown encouraging results. One study indicated that the scanner detected more than 90 percent of the known delaminations, some of which were less than 6 in. (150 mm) in diameter (78). The results of the other study were not conclusive for quantity estimates, but the applicability of IR equipment to general rehabilitation programming is warranted (98,99).

A number of commercial IR systems have been tested using various configurations of the equipment, including aerial and van-mounted apparatus. Between the two, the van-mounted equipment, which provides simultaneous recordings of the infrared scan and a video of the actual surface appearance, is more practical. The results of recent tests have concluded that the optimum height above the deck for the scanner is in the range of 13 to 20 ft (4 to 6 m) (78,97,98).

Although IR thermography is a promising tool for bridge inspection, it has limitations. First, detecting the point of debonding or the lateral extent of debonding with any degree of accuracy is difficult. Second, there are difficulties associated with isolating and identifying hot spots, such as bituminous patches, crack sealers, and tire marks. The presence of these affects the quality of interpretation and requires appropriate visual records to complete the interpretation. Third, a better definition is needed of the weather conditions under which IR thermography is most effective. Finally, IR thermography requires the development of software to produce a scaled hard copy from videotape.

The application of IR scanners on timber bridges has not been reported on in the literature. However, the successful use of IR scanners to develop a remote sensing technique for tree stress surveys (100) and the satisfactory results of the use of scanners on concrete bridge decks indicate that IR thermography has great potential for detecting the deterioration of timber bridges, including piles and decks.

Summary of Inspection Techniques

The application of nondestructive approaches to timber bridge inspection has been minimal. The review of the use of these methods on concrete bridges shows that the technology in this area has advanced significantly in the last decade. This new technology, particularly in radar and IR thermography, has great potential for use in routine operational procedures for detecting the deterioration of timber bridges. However, at present, the nondestructive techniques are better suited to the rapid assessment of the overall condition of a large number of bridges for developing rehabilitation programs than to the provision of detailed information for the replacement or repair of individual bridges.

RATING

As discussed previously, the field inspection establishes the extent of deterioration in load-carrying members of bridges. After inspection, the information gathered must be rated so that the bridge engineers can make safety decisions about the repair, rehabilitation, posting, closing, or replacement of an existing bridge. The rating of bridge strength generally follows procedures similar to the AASHTO design code, including checks for specified design loads, girder distribution and impact factors, and allowable stresses. Therefore, bridge rating is an important step in producing acceptable safety.

Most bridge rating methods were developed for steel and concrete bridges. Some of these methods use load spectra (101), and others apply the pragmatic approach for rating (102). The pragmatic approach uses a procedure to rate highway bridges for regulation loads without causing yielding of the bridge materials. Federal and state governments have developed several computer systems that inventory the actual conditions of bridges (103,104). These systems may provide a data base for rating and selecting a preferred bridge maintenance program (105-107).

There is not however, a well-developed rating method for timber bridges. The Forest Service has developed a system for bridges and major culverts (BMC) to be used in conducting inventories of forest road structures (108). Implemented several years ago, this national system is progressively being enhanced. Because most Forest Service bridges are timber bridges, the BMC is considered to be a timber bridge inventory system. A major element of this system is composite rating, in which a numerical rating, ranging from 0 to 9, is assigned to the condition of the bridge as a whole. This rating reflects the inspector's evaluation of the more critical features of the bridge affecting safety and cost.

The rating scores rely partly on the mathematical ratings for load capacity and on the inspecting engineer's judgment. Scientific approaches for timber bridge rating have yet to be developed.

MAINTENANCE

Based on the field inspection and the rating, a bridge maintenance program can be developed. Government agencies use numerous methods to develop bridge maintenance programs. For example, the Minnesota Department of Transportation has employed a systems approach as a planning tool (109,110), while the Wisconsin Department of Transportation has used the linear programming method (111) to minimize maintenance costs.

Timber bridge maintenance can be classified into three categories: replacement, repairs, and preventive maintenance. As indicated previously, the age of a structure is a major factor in the deterioration of a bridge. However, a thorough inspection should determine the actual condition of the structure before a decision is made on a maintenance plan.

Replacement

When the load-carrying capacity of a member has decreased by 50 percent or more of its design capacity, the member must be replaced, repaired, or strengthened by adding reinforcements (100,113). Replacement includes the replacement of timber stringers, plank decks, defective piling, and so forth. The construction procedure for replacement is documented elsewhere (114,115). In the case of a partial replacement, the replacement must extend 2 ft beyond the defective area of the member (113). Onsite preservative treatment is recommended for the area surrounding the defect and the newly exposed cutoff area.

Repairs

Repairs aim to extend the service life of a bridge, to increase or maintain the load-carrying capability of the structure, and to improve bridge safety. Repairs range from strengthening existing timber pier caps to fixing cracked or split timber stringers. Depending on the member of the structure and the severity of the damage, methods of repair include reinforcing a member, adding a sister member, stitch bolting, adding protective armors, post-tensioning, using a composite deck, repairing with epoxy, and using preservatives. The application of these methods was discussed in detail in a recent study (112). It is important to point out that after post-tensioning, the load-carrying capacity of a rehabilitated, nail-laminated deck can be increased as much as 100 percent (115). The epoxy repair method was introduced in 1976 (116) and has proved to be highly effective (117). Its applicability to glulam beams also has been demonstrated (118). This approach has great potential for timber bridge repair (119).

Preventive Maintenance

Preventive maintenance can be carried out either by field preservative treatments or by moisture control. Field preservative treatments should be applied at regular intervals to the major structural components of a bridge, such as stringers, piles, caps, and the exposed untreated surface of timber that has resulted from collision damage, delaminations, checks, and splits, to protect them from decay. Forest Service experience indicates that field treatments should be considered when decay is under way on not more than 25 percent of the structural members. Otherwise, when decay has attacked 40 percent or more of the members, a percentage of them will have to be replaced. For instance, when decay has affected 80 percent of the structural members, 14 percent of them will have to be replaced (120).

Moisture control is required at sites where bridge timber is subjected to frequent decay-causing wetting. Proper bridge surface drainage as well as a clear waterway will reduce decay or collision damage. Periodic inspections during periods of minimum moisture to ensure that no glueline separation is developing would be appropriate (121).

RESEARCH AND TECHNOLOGY TRANSFER

The outcome of the future wood utilization research will have significant impact on the research and development of timber bridges. Three major institutional groups in the United States that are involved in research and development concerning the use of wood are the federal government, the forest industry, and academic institutions. The federal government funds and performs research and development, the forest industry develops products and improves manufacturing processes, and academic institutions conduct training and basic research. [The research and development activities of the three groups are reported elsewhere (18).] Two of the groups' major concerns with respect to the timber bridge technology development are research and technology transfer.

Research

As stated in a 1983 report (9), three major research areas are (a) rationalization of limit states design for uniformity in the various structural codes; (b) incorporation of reliability-based formats; and (c) development of a systems approach to the analysis of structures. In the first area, some researchers are applying fracture mechanics theories to define the ultimate strength limit state of wood. The result of research in this area could aid in producing ingrade strength of full-size structural members of a bridge as a basis for establishing allowable design stress in the future.

Significant progress has recently been made in the second area of research. In Canada, the Ontario highway bridge code was developed from probability-based methods of analysis (36). In the United States, a set of reliability indices for single-timber bridges with deck planks and stringers was calculated using probability-based analysis (38). The results of this research are encouraging for the future development of a probability-based design code.

Research in systems approaches to structural analysis also has made headway. Recently, a systematic approach has been developed for the assessment of the reliability in wood structural systems under load (122). The method calculates performance factors in reliability-based design equations and determines reliability levels for "baseline" structures that are accepted as safe and adequate and to which the design of future structures could be calibrated. Further research needs on timber bridges have been documented elsewhere (123, and in communication notes on technical applications to Harold Strickland, Assistant Director of the Engineering Staff, Forest Service, U.S. Department of Agriculture, in 1985).

Technology Transfer

The concept of timber bridge technology transfer was initiated in the Forest Service Road Technology Improvement Program, which was designed to evaluate, develop, and adopt new concepts and technologies to improve road construction, operation, and maintenance, and, consequently, to lower transportation costs. Recently, the Forest Service, in coordination with the American Institute of Timber Construction, established an Industry-Federal Government Cooperative Program on timber bridge technology transfer (124). This cooperative program attempts to encourage the adoption of beam and deck technology and has a target to increase the use of timber bridges tenfold in 5 years (using 1983 as the base year).

It is difficult to transfer new timber bridge technology by means of education and publications. Therefore, the cooperative program devised three specific objectives, as follows: (a) to inform state, county, and township officials, as well as federal agencies, engineers, and contractors about the advantages of timber for new and replacement bridges on local and secondary road systems and federally owned properties; (b) to provide guidance on the rehabilitation of existing timber bridges; and (c) to cooperate with interested professional and industry associations, technical organizations, and government agencies to improve the nation's road systems by providing safe and economical alternatives for bridge replacement needs.

An implementation team developed six tasks for the implementation of the 5-year cooperative program as follows:

1. Develop a timber bridge design and construction manual that would have a larger scope than that developed by Weyerhauser in 1980 (125);
2. Document the economy of timber bridges by providing the requisite initial cost and life-cycle cost information;
3. Study ways to reduce the transverse load-carrying criteria or to design and test a cost-effective bridge-railing detail that meets the current AASHTO requirements (in practice, the load-carrying criteria specified by AASHTO for bridge railings exceed that used by the Forest Service);
4. Disseminate general information through bulletins, leaflets, workshops, seminars, lectures, papers, and demonstrations;
5. Select sites for constructing demonstration projects; and
6. Develop a technology transfer schedule of activities.

CONCLUSIONS

Considerable progress has recently been made on the use of wood as a construction material as follows:

1. Preservative agents may extend the serviceable life of timber considerably. Thus, the life expectancy for glulam-treated timber bridges can increase to 50 years.
2. The evolution of timber bridge design follows the progress of research on timber's load-carrying capacity. Two major approaches for analyzing load-carrying capacity are based on allowable stress, which is used in AASHTO specifications for glulam timber bridges, and on probability methods, which are used in the Ontario Highway Bridge Design Code for prestressed, treated timber bridges. A probability-based design code for glulam timber bridges is still in the development stage.
3. The construction of timber bridges is simple and economical. Procedures for constructing glulam timber bridges are defined in AASHTO specifications and Forest Service reports, while procedures for constructing prestressed bridges has been developed from the Canadian experience in Ontario. Methods for constructing prestressed bridges will require modification as construction experience for this type of bridge progresses. Compared to the costs of steel and concrete bridges, the costs of constructing prestressed treated bridges can be as much as one-third less.
4. Destructive testing approaches are still used for timber bridge inspection. However, there has been considerable research on developing nondestructive inspection methods. The results of studies in this area are encouraging. Both radar and IR thermography have great potential for application to timber bridges.
5. Timber bridge rating mainly relies on the inspecting engineer's judgment. There has been little research in this area. Recently, however, automation systems have been developed for use in conducting inventories on timber bridge conditions and providing information for use in formulating cost-effective bridge maintenance programs.
6. The methods for repairs and preventive maintenance of timber bridges also have been improved considerably. Case studies indicate that a post-tension system of rehabilitation can double a nail-laminated deck's load-carrying capacity. Several studies also have demonstrated that preventive field treatments are cost-effective.
7. The current focus on timber bridges centers on technology transfer. The Forest Service coordinates with other government agencies and the timber industry to develop the Industry-Federal Government Cooperative Program on Timber Bridge Technology Transfer. The goal of this program is to increase the use of timber bridges tenfold within 5 years.
8. The technology associated with various aspects of timber bridges continues to be developed in significant ways. The availability of this technology, along with the advantages of using wood as a construction material and the reduction of government budgets, will make timber bridges more appealing. The initiation of technology transfer by the Forest Service is timely and will have significant impact on the use of timber bridges on low-volume roads in the near future.

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Designing Timber Bridges for Long Life

FRANK W. MUCHMORE

ABSTRACT

Wood is a marvelously adaptable structural building material. When treated with a compatible preservative to prevent early decay deterioration, it is an economical and practical structural material for many short-span bridges (spans in the range of 15 to 60 ft). Timber's inertness to deicing chemicals, as well as some new design developments, such as glued-laminated deck panels and prestressed laminated decks, make it more attractive than ever for use in highway structures. Important factors in assuring long, useful lives for timber bridges include designing to avoid water-trapping details, use of effective and compatible preservative treatment, and following a systematic inspection and maintenance program. Attention to these factors will provide a lifespan that is competitive with other structural materials, such as steel and concrete, and will, in most cases, dramatically increase the useful life of timber bridges.

Timber bridges are remarkable structures--when properly designed, built, and maintained, they can (a) carry heavy loads without showing material fatigue, (b) resist the deteriorating action of deicing chemicals, (c) be constructed by unskilled labor, and (d) last for long periods of time. Timber is particularly adaptable to short-span bridges (spans up to 60 ft) and can be economically designed to carry heavy highway loads. For example, some timber bridges in western Canada routinely carry logging trucks and machinery weighing in excess of 100 t.

Wood was one of man's first building materials. However, as modern technology has yielded its wonders in mass-produced steel, aluminum, concrete, and plastic construction materials, wood has been largely overlooked as a primary structural material for bridges. Wood is still a marvelously adaptable structural material. For example, structural wood

Northern Region, P.O. Box 7669, Forest Service, U.S. Department of Agriculture, Missoula, Mont. 59807.

- Is simple to fabricate,
- Is lightweight and easy to install,
- Has a high strength-to-weight ratio,
- Has excellent sound and thermal insulation properties,
- Has good shock resistance,
- Is immune to deicing chemicals,
- Has unique aesthetic qualities,
- Is a renewable resource,
- Is long-lasting (when properly protected), and
- Has good fire resistance (1).

Note that the unusual claim of good fire resistance of structural timber is well founded. When a large structural timber is exposed to fire, there is some delay as it chars and eventually flames. As burning continues, the charred layer has an insulative effect, and the burning slows to an average rate of about 1/40 in. (0.6 mm) per min [or 1-1/2 in. (38 mm) per hr], for average structural timber species. This slow rate of fire penetration means that timber structural members subjected to fire maintain a high percentage of their original strength for considerable periods of time (2). In contrast, structural steel becomes plastic when exposed to a heat of 1,000°F and it yields almost immediately.

In addition, structural wood is economical. A number of Forest Service bridge construction projects in Montana and Idaho during 1984 and 1985 involved competitive bids for treated timber bridges versus bids for equivalent concrete bridges. At least four timber bridge contracts were awarded.

Many of the covered wood bridges built during the eighteenth and nineteenth centuries have been used for more than 100 years (3). Modern developments and techniques have advanced wood technology significantly since the days of the covered bridges. The widespread use of glued-laminated members, availability of effective preservative treatments, epoxies, and new prestressing techniques are but a few of the developments. Blending the old and new technologies, coupled with knowledge design, construction, inspection, and maintenance practices, is indeed bringing a new popularity to timber bridges (1), and under proper conditions, wood will literally give centuries of service. If it can be protected from organisms that degrade wood, its longevity is remarkable (2). Some techniques and processes to achieve this protection are covered in this paper.

ADVANTAGES OF USING WOOD

As a structural material, wood has the many advantages previously enumerated and comparatively few disadvantages. Dr. John F. Levy of the Imperial College of Science and Technology, London, eloquently states wood's major disadvantage: "As far as a fungus is concerned, wood consists of a large number of conveniently oriented holes surrounded by food" (4). The key to overcoming this major disadvantage of wood as a structural material is to make it inedible to decay fungi and microorganisms and to wood-boring insects and marine organisms. The use of wood preservatives simply forms a protective shell (Figure 1) that renders the wood unpalatable or toxic to all these little "critters." It is important, however, to use compatible woods and treatment methods. Compare the protective shell shown in Figure 2 with that shown in Figure 1. Certain species (and subspecies) do not accept preservative treatment as well as others. Care must be exercised to specify species and treatment methods that will give satisfactory results.

Treatment with the proper preservative process dramatically increase the useful life of wood exposed

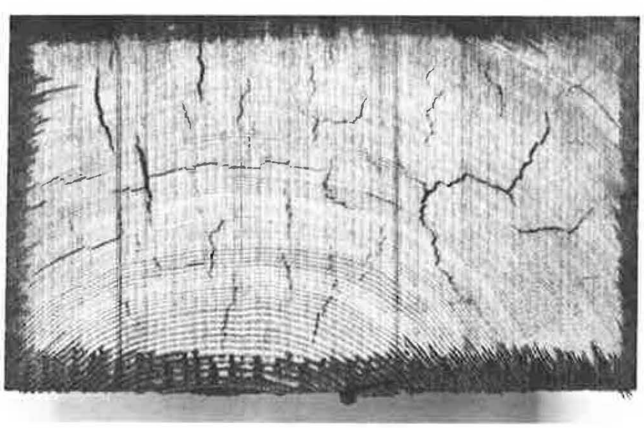

FIGURE 1 Sawed cross section of a pressure-treated coast region Douglas fir bridge stringer.

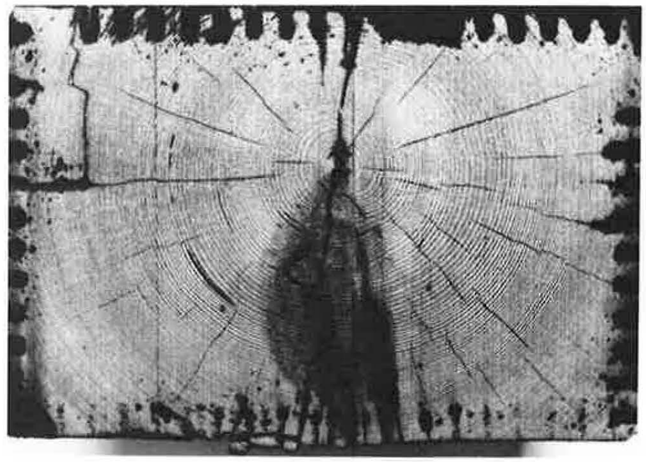

FIGURE 2 Sawed cross section of a pressure-treated inland region Douglas fir bridge stringer.

to the elements. For example, Richardson (4) states that in temperate climates, a normal transmission pole pressure treated with creosote will have a typical life of 45 to 60 years, whereas an untreated identical pole will last only 6 to 12 years. A similarly treated railroad tie can be expected to last more than 35 years, in comparison to 8 to 10 years for untreated wood.

Richardson (4), in discussing the advantages of preserved wood, states:

> Preserved wood must be regarded as an entirely new structural material and must not be considered as just an improved form of wood, as it can be used in entirely different circumstances and certainly in more severe exposure situations. The most obvious advantage of preserved wood is that it can be used with impunity in situations where normal untreated species would inevitably decay, but it may be argued that, in many situations, this is a property that it enjoys together with many competitive materials. In fact, the use of wood has many advantages. It is extremely simple to fabricate structures from wood and, even in the most sophisticated production processes, the tooling costs are relatively low compared with those for competitive materials. Wood is ideal if it is necessary to erect an individual structure for a particular purpose but is equally suitable for small batch or mass production. When these working properties are combined with the other advantages of wood, such as its high strength-to-weight ratio, its excellent thermal insulation and fire resistance, its immunity to deicing chemicals, and the unique aesthetic properties of finished wood, it sometimes becomes difficult to understand why alternative materials have ever been considered! However, there is one feature of wood which is unique amongst all structural materials; it is a crop which can be farmed, whereas its competitors such as stone, brick, metal and plastic are all derived from exhaustible mineral sources.

With all these varied advantages, structural timber exposed to the elements, but properly selected and preserved, can give satisfactory service for 50 years and, in some cases, far longer, and at costs that are competitive with other structural materials, such as steel and concrete. Indeed, there is no reason why well-designed and maintained timber bridges could not survive more than 100 years, as have the previously mentioned covered bridges.

CAUSES OF WOOD DETERIORATION

Decay is by far the most prevalent cause of wood deterioration. However, wood-boring insects and marine organisms will also attack unprotected wood.

Decay in wood is caused by living fungi, which are simple plants that have the capability to break down and use wood cell wall material for food. Infections of decay fungi are often indicated by the presence of "fruiting bodies," mushroom-like, or shelf- or hoof-shaped projections of flat, leathery material commonly found in partially enclosed or sheltered areas, as is shown in Figure 3. Decay fungi are propagated by germination of fungus spores, which are functionally equivalent to the seeds of higher plants. These spores, produced in massive numbers, are microscopic. The spores are distributed so widely by wind, insects, and other means that they are commonly present on most exposed surfaces throughout the world (2).

Most of these spores never germinate for lack of a favorable environment. All of the following conditions must exist in order for decay fungus growth to take place:

1. A sufficient oxygen supply,
2. A favorable temperature range [32°F (0°C) - 90°F (32°C)],
3. An adequate food supply (wood cells), and
4. An available water source [i.e., moisture content must be above the fiber saturation point, or approximately 30 percent for most species (2)].

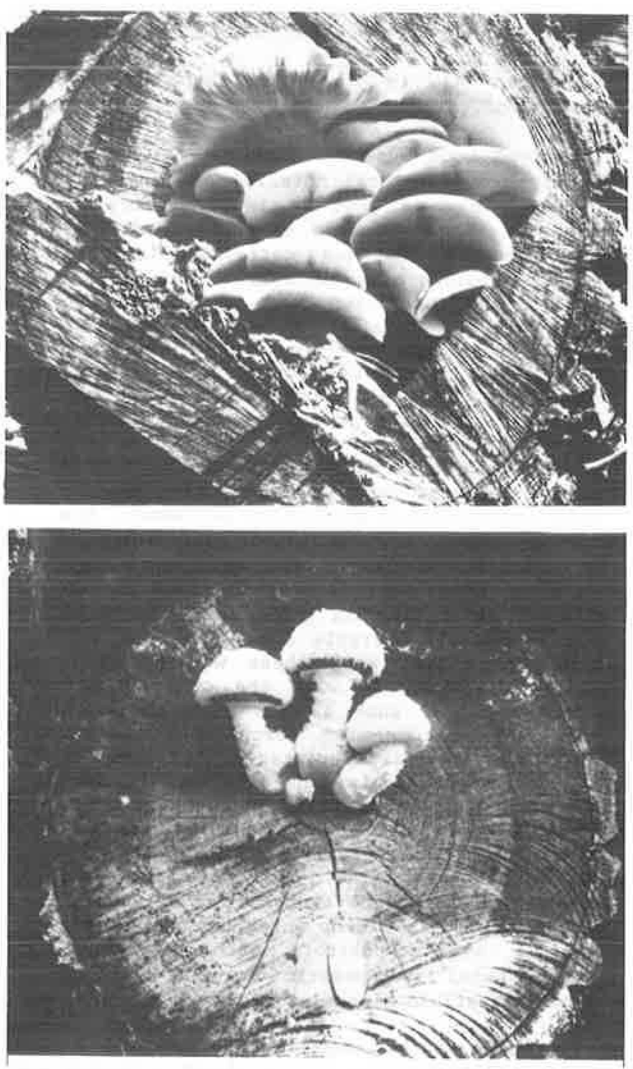

FIGURE 3 Examples of decay fungus fruiting bodies in growing or fresh condition.

Once established, the decay fungi continue to grow at an accelerating rate as long as favorable conditions prevail. Depriving the fungus of any one or more of these required conditions will effectively curtail the spread of decay. Proper preservative treatment effectively provides a toxic barrier to the decay fungi's food supply, thus preventing decay.

Wood that is kept dry (moisture content below the fiber saturation point), or saturated will not support the growth of decay fungi. The unusually long useful life of the covered wood bridges of the eighteenth and nineteenth centuries can be primarily attributed to keeping the wood dry.

When wood is kept at a moisture content below the fiber saturation point, there is insufficient available water to support the growth of the decay fungi. Conversely, wood kept completely and continuously saturated with water will not support the growth of decay fungi, because a sufficient supply of oxygen is not available. However, care must be taken when wood is used in a marine environment. Marine borers, which can quickly perforate wood that is immersed in sea water, become a major hazard unless the wood is properly preserved.

A common misconception is the use of the term "dry rot," which erroneously carries the connotation that dry wood will rot or decay. Even the growth of dry-rot fungi (Merulius lacrymans) requires that the wood have a moisture content above the fiber saturation point. Wood maintained at a moisture content below fiber saturation can be considered safe from decay hazard including dry rot.

TECHNIQUES FOR CONTROLLING MOISTURE

Control of moisture is probably the most cost-effective and practical general technique for extending the service life of new and existing timber bridges. This protective measure undoubtedly contributed significantly to the exceptionally long service life (more than 100 years in some cases) achieved by many of the old, covered wooden bridges. However, in most of the more modern timber bridge designs, preservative treatments have been relied on to control decay, and the older principle of keeping water away from the structural members in the first place is given little (or no) emphasis (1).

Timbers that develop seasoning checks, sometimes after installation, increase the decay hazard significantly. Seasoning checks normally open in the radial direction from the heart (or pith) of the tree. If the heart can be excluded from the structural member, the number of potential seasoning checks is significantly reduced. A specification that boxed-heart members will not be acceptable will accomplish this, although the material price may be increased by 10-15 percent. Glued-laminated members should be used where practical. There will be far less shrinkage and formation of seasoning checks in the individual laminations than in a solid member of the same dimensions. However, it is important to specify the use of waterproof glue for exterior use applications.

Some attempts to control moisture have proven detrimental (1). Examples are

1. Roll roofing material as a water-diverting cover on caps, stringers, and pile or post tops. As the roofing material ages, it dries out and becomes permeable to water from above. More harm than good may be done by using this type of material because it tends to trap moisture and inhibits drying, thus promoting decay.

2. Sheet metal covers between pile or post tops and overlying stringer support caps are also of questionable value. The metal covers are usually installed before the timber cap is placed. When drift pins are driven down through the cap into the pile or post top, the metal cover is punctured and dimpled, creating a funnel for trapped water to enter the pile top and run down the drift pin past the treated top surface into the unpreserved inner wood. Again, the metal also inhibits drying, thus setting up a "double-jeopardy" situation for a high moisture content in untreated wood, which is one of the conditions conducive to the growth of decay fungi. Some results are shown in Figure 4.

FIGURE 4 Sheet metal caps between pile tops and stringer support caps may increase decay hazard.

3. Sheet metal caps on exposed pile or post tops as shown in Figure 5, are also generally regarded as contributing to early decay. Exposed metal caps also attract vandals who cannot resist punching holes in the metal caps with sharp objects or bullets. The net result again is the creation of funnels for water entrance, while inhibiting the drying of the pile or post top. An effective design criterion for the promotion of good drainage is to slope the exposed tops of posts and other water-trapping places.

More effective moisture-proofing requires the use of bituminous or asphaltic mastics in lieu of sheet-metal or roll roofing material as end-grain coatings, joint fillers or seals, and check-filling compound.

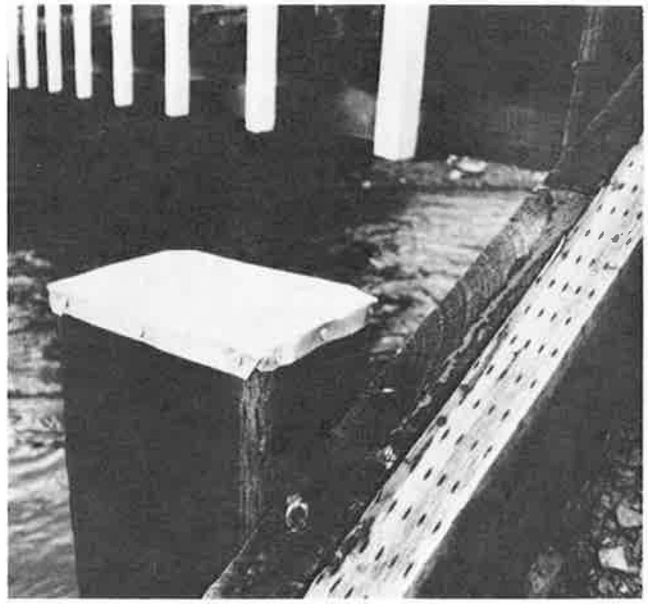

FIGURE 5 Sheet metal caps on exposed pile or post tops may become punctured and funnel water into pile top.

Care must be taken not to form water-entrapment areas under such coatings. It is also important that the end-grain or freshly cut surfaces be protected by a wood preservative before the application of the mastic. Also, since petroleum compounds can dry out, lose their elasticity, and eventually crack, an effective maintenance program will, where feasible, include reapplication (1).

The designer should try to avoid using details that will trap moisture, soil, or other debris, which create conditions conducive to decay. Also, field fabrication of wood members (cuts, holes, daps, etc.) should be held to an absolute minimum because field treatment (flooding, spraying, brushing) is not nearly as effective as in-plant pressure treating after fabrication takes place. The control of moisture access to the structural members should be of primary concern. The principle of the covered bridges should be remembered.

Perhaps the most effective and practical method for controlling moisture on most wood bridge designs is to prevent water passage through the deck. In effect, make the bridge deck an impervious roof over the supporting stringers. If the timber deck is sufficiently rigid and stable, a well-maintained asphalt mat (usually at least 3 in. thick) with a moisture-barrier membrane, crowned to aid drainage, can be quite effective (1).

There are several feasible alternatives to consider in the production of a stable timber bridge deck (1). For example, the use of treated glued-laminated timber panels for decking is quite effective when the laminates are oriented parallel to the bridge ends. Besides providing a more rigid support for asphalt mats than conventional nail-laminated decks, glued-laminated deck panels are produced at a moisture content of about 12 percent and will thus be subject to little shrinkage during service, helping to minimize crack formation in the mat (5). Figure 6 shows glued-laminated panels being installed.

Experience has shown that reflective cracking of the asphalt mat usually occurs over the glued-laminated panel interfaces, particularly when the girder spacing is more than 4 ft (1.2 m). This may not be detrimental to the panels, since in the original development of deck panels, the "treatment" was con-

FIGURE 6 Glued-laminated deck slabs being installed as a replacement deck on a 12-year-old bridge.

sidered the moisture barrier, not the asphalt. However, leakage at the panel interfaces will create undesirable wetting of the girders and probably create unsightly stains on the exposed girder faces.

The placement of a geotextile fabric mat as an underlayment for the asphalt mat may help prevent the reflective cracking. The fabric must be tacked down to the deck surface with liquid asphalt, then the asphalt mat is placed by normal methods. The Forest Service has placed fabric experimentally on several bridges (6), but no conclusive results have been documented to date.

Another product that may have promise as a wearing surface and roof is elastomeric concrete. Developed in France for installing expansion joints in concrete decks, it is also used as bridge deck surfacing for orthotropic steel deck bridges. It is expensive [$5.00 to $10.00 per square foot ($54.00 to $108.00 per square meter)] and somewhat difficult to install in the field, as it requires a heat vulcanizing process. It may be more practical if it is installed in the plant, on glued-laminated deck panels, for instance.

Another technique for reducing moisture penetration and producing a stable deck surface is to "prestress" (or "post-tension") a laminated timber deck (7). The prestressing force is applied by installing the prestressing bars through predrilled holes at mid-depth of the laminates. Steel channel bulkheads and anchorage plates are used to anchor the prestressing bars (see Figure 7).

This system allows prestressing (posttensioning) of either longitudinally laminated or transverse laminated decks. Transverse laminated panels can be prefabricated in the shop with steel plant bulkheads at the joining ends of the panels and can be made as large as practical for handling and transportation. When installed, the panels are prestressed together by using high-strength bar couplers between panels and jacking against the steel plate bulkheads in a manner similar to segmental concrete construction. Prestressed transverse laminated decks as long as 150 ft (46 m) and longitudinally laminated decks as long as 400 ft (122 m) have been continuously tensioned by this method in Canada.

For rehabilitation of an existing nail-laminated deck, the prestressing force is applied perpendicular to the laminates by a system of high-strength bars located above and below the deck. These bars are attached to heavy steel anchorage plates bearing against a steel channel to form a strong, flexible clamping system (see Figure 8). Then the top bars

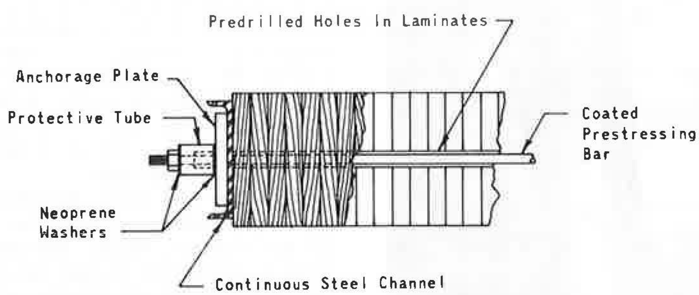

FIGURE 7 Prestressing system for new construction (courtesy of Ministry of Transportation and Communications, Ontario, Canada).

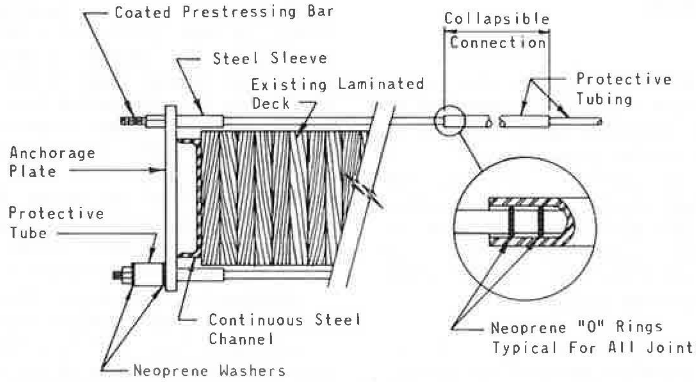

FIGURE 8 Prestressing system for rehabilitation (courtesy of Ministry of Transportation and Communications, Ontario, Canada).

are embedded in the asphalt mat. This rehabilitation system is most adaptable to longitudinally laminated decks (laminates running parallel to the roadway centerline) and perhaps short transverse laminated decks (laminates perpendicular to roadway centerline).

The prestressed timber deck system was developed and tested in Canada by the Research and Development Branch of the Ontario Ministry of Transportation and Communications, and has been successfully used for a number of rehabilitation projects and new bridges there, as well. More detailed information can be found in Taylor et al. (7).

DESIGN CRITERIA FOR MAXIMIZING SERVICE LIFE

Designing a timber bridge for the longest practical life demands attention to details. The designer must be constantly aware that wood is in the "danger zone" for decay hazard whenever the moisture content is between fiber saturation (approximately 30 percent for most species) and fully saturated (100 percent). Details that do not trap moisture, and that allow free drainage of rainwater are highly desirable.

The following is a summary of some of the more important items to be considered by the designer:

1. Only wood species (or subspecies) that are sufficiently permeable to permit the penetration and retention of preservative required by an accepted standard should be used, such as American Wood Preservers Association (AWPA) standards C1, C2, and C28. Avoid species or subspecies that are difficult to treat. Guidance for species treatability is avail- able from the Forest Products Laboratory, P.O. Box 5130, Madison, Wisconsin 53705.

2. Oil-borne pressure preservative treatments (pentachlorophenol, creosote) are generally more satisfactory for bridge timbers than waterborne treatments. There are several reasons for this. Among them are

• Material treated with waterborne preservatives frequently have a higher internal moisture content as a result of the treatment process. Thus, they are subject to more drying shrinkage, checking, and splitting.

• Oil-borne treatments leave a residual of oil on and near the surface, which provides a measure of waterproofing (depending on the oil used). The moisture content, after treating, is typically lower than for waterborne-treated material, which tends to minimize drying shrinkage, checking, and so forth.

3. Fabricate all timber completely and accurately (holes, cuts, daps, etc.) before treatment with preservative to assure a complete protective envelope (see Figure 9). Avoid field fabrication (drilling and cutting) after treatment. When field fabrication or repair of damaged treated members is necessary, field treatment should be done in accordance with accepted standards, such as AWPA M-4. Remember that field treatment is not nearly as effective as in-plant pressure treatment.

4. Avoid design details that trap water, dirt, or other debris (see Figure 9).

5. To minimize the development of seasoning checks, which can expose untreated wood to decay hazard, avoid using timbers that have "boxed-heart," that is the heart (or pith) of the tree is contained within the member (see Figure 9).

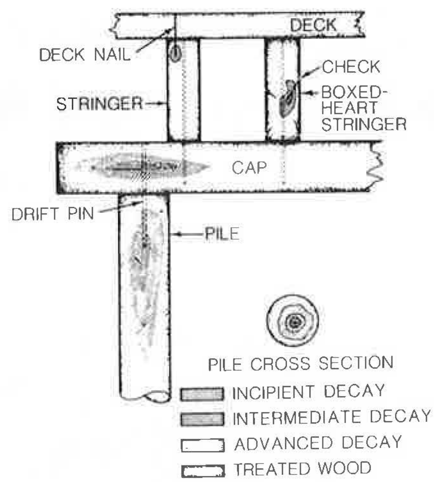

FIGURE 9 Schematic diagram of a portion of a pile bent, stringers, and deck showing locations where decay is most likely to occur.

6. Use glued-laminated members (waterproof glue) where practical, which are produced from kiln-dried lumber, typically at moisture contents of around 12 percent. Drying shrinkage, checking, and splitting are minimized.

7. Avoid using roll roofing or sheet metal caps on stringers, pile, or post tops. Their use can increase decay hazard by inhibiting the drying of the wood.

8. Made the bridge deck an impervious "roof" over the supporting stringers. Asphalt mats (with membrane), glued-laminated panels, or prestressed timber decks can be effectively utilized.

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Lateral Stability Considerations of Timber Beams in Old Bridges

BRUCE A. SUPRENANT, FRED VIDEON, RICHARD EHLERT, and ALAN JACKSON

ABSTRACT

Some timber bridges built in the early 1900s do not have the roadway deck attached to the beams. These laterally unsupported beams are not capable of supporting current truck loads. County and state engineers who have inherited the responsibilities of maintaining these bridges must use considerable skill in developing a lateral bracing scheme for the beams. County engineers are usually working within a definite financial constraint such as the county budget. This paper was written to help engineers understand the history, theory, and practical solutions required for bracing timber beams in old bridges. A history of the early requirements for the design and construction of bridges is reported. The 5-ton loading requirement of the early 1900s does not compare with the current truck loads of 15 tons or higher. A brief discussion of the theory of lateral buckling of beams is presented along with methods to calculate the forces in the bracing. The current AASHTO design practice for laterally unsupported beams is presented. The effect on the allowable stresses resulting from lateral bracing of beams is shown. Some deck-to-beam and beam-to-beam connections are presented as possible solutions to the beam instability problem. The deck-to-beam connections discussed are nailing, bolting, reverse bolting, angles and bolts, and friction. The beam-to-beam connections presented are blocking, cross-bridging, and tension side bracing. The advantages and disadvantages of each type of connection is discussed. Tension side bracing of beams, while perhaps being economically feasible, has not been verified either analytically or experimentally. There are no current design guidelines for tension side bracing.

The ingenuity and skill of an engineer can be tested when developing a rehabilitation scheme for an existing bridge. In addition, if that bridge was built between 1900 and 1920, the engineers' problem is even further complicated. County and state highway engineers have inherited the responsibility of maintaining many old bridges. A common problem of timber bridges built in the early 1900s is the lack of lateral support provided to the beams. In other words, the deck planks were not attached to the timber beams and no other lateral bracing was provided. These laterally unsupported beams may have been strong enough for the loading requirements of the early 1900s, but buckle laterally under current truck loads.

Television reporters always seem to find it highly amusing to show pictures of a school bus stopped at a bridge while the children walk across. Figure 1 shows such a bridge located in Montana. The capacity of this bridge was limited by the moment capacity of the beams. The beams were laterally unsupported for the full length. To increase the moment capacity of the beams, county personnel worked for two weeks to add blocking between the beams. This blocking, which proved to be an expensive operation, is shown in Figure 2. However, after the beams were braced, the load capacity of the bridge was increased. Figure 3 shows the posted load limits for the rehabilitated bridge.

Typically, county engineers encounter a technical and financial problem in that most counties do not usually have sufficient funds to maintain all of the bridges in their jurisdiction. The topics specifically addressed herein, which concern laterally unsupported beams, are (a) their lack of strength, (b) their theory and practice, and (c) rehabilitation techniques for increasing the bridge capacity by decreasing their length. The material presented is not intended to be theoretical, but rather oriented for the practicing engineer.

FIGURE 1 Old County Bridge.

HISTORY

In the early 1900s, some companies had prepared general specifications for the design and construction of bridges. It appears that the American Bridge Company set the standard for the era. This company

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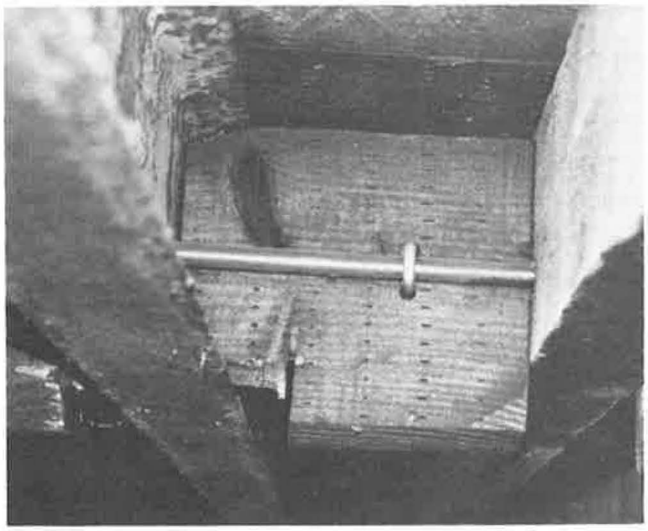

FIGURE 2 New blocking placed between existing steel and timber beams.

FIGURE 3 Posted weight limits for rehabilitated bridge.

not only had general specifications for the design of bridges but had also published a book in 1901 entitled, Standards for Structural Details (1). The specifications prepared by the American Bridge Company and J.A.L. Waddell, a consulting engineer, will be reviewed to illustrate the standard design of the early 1900s. A comparison of the design requirements of early timber bridges and current requirements will show the necessity for bracing timber bridge beams.

The American Bridge Company, in their specifications, divided highway bridges into six classes (1). These classes, with their descriptions and loading information, are given in Table 1.

J.A.L. Waddell divided highway bridges into three classes (2). These classes, with their description and loading information, are given in Table 2. Waddell's specifications (1) require that the designer assess the loading that is due to electric trains. Waddell also requires that a uniformly distributed live load be considered on the entire roadway, including footwalks. This uniformly distributed design load is determined from Figure 4, provided in the specifications. Waddell states,

"In addition to these (uniformly distributed) loads, the floor, joists, floor-beams, beam hangers, and primary truss members are to be proportioned for the following concentrated loads (shown in Table 2), which are, however, supposed to occupy a whole panel length of the main roadway to the exclusion of the other live loads there (excepting only the electric-railway live load)" (1).

The specifications provide a good overview of the loading requirements of bridges. However, the design and construction requirements of the beams, specifically those concerning bracing and attachment of the deck, were difficult to assess.

The specifications of this era, 1900-1910, provided significant information regarding the stability of beams, through attachment to the deck, for railroad bridges. For instance, the American Bridge Company (2) required 8-in. x 8-in. deck timbers with a center-to-center spacing of 14 in. or less. The timbers were required to be notched 1/2 in. and have a full and even bearing over every beam. Every fifth deck tie was to be fastened to the beam by a 3/4-in. bolt. These typical requirements for railroad bridges assured the lateral stability of the beam by the attachment of the deck. Figure 5 shows the attachment of the deck to the beams for a railroad bridge. This is a typical floor system used by the New York Central and Hudson River Railroad Companies (NYC and HRRR).

The specifications are not as precise for bridges that would be categorized as Class D (American Bridge Company) or Class C (Waddell). For instance, the American Bridge Company specifications (1) indicate the following for roadway planks and stringers:

1. For roadway planks, "For single thickness, the roadway planks shall not be less than 3 inches thick, nor less than one twelfth of the distance between stringers, and shall be laid transversely with 1/4 inch openings.

"When an additional wearing surface is specified for the roadway, it shall be 1 1/2 inches thick, and the lower planks, of a minimum thickness of 2 1/2 inches, shall be laid diagonally and with 1/2 inch openings."

2. For stringers, "Wooden joists shall not be less than 3 inches thick, shall be spaced not more than 2 1/2 feet between centers and shall be dapped over the seat angles or floor beams to exact level. In the latter case, they shall lap by each other over the full width of the floor beam, and shall be separated 1/2 inch for free circulation of air."

As is evident, the specifications do not mention any attachment of the roadway planks (deck) to the beams or bracing of the stringers (beams).

It has been the authors' experience that county engineers are encountering bridge structures which, in 1900, would have been classified as Class D (American Bridge Company) or Class C (Waddell). The bridge structures typically do not have braced beams, either through attachment to the deck or by extra bracing. The necessity for braced beams has been dictated by the increased loading requirements of new vehicles. A comparison will be made between existing design requirements and those for which the bridge was originally designed.

The American Association of State Highway and Transportation Officials (AASHTO) has indicated the loading requirements in the 1983 Standard Specifications for Highway Bridges (3). AASHTO provides four standard classes of highway loadings: H 20, H 15, HS 20, and HS 15. For comparison, H 15 (the smallest loading) will be considered as the design requirement for a rural county road.

TABLE 1 **American Bridge Company Specifications for Live Load** *(1)*

Type	Description	Loading (floors & supports)
Class A	City traffic	A concentrated load of 24 tons[a,b,c]
Class B	Suburban or interurban traffic with heavy electric cars	A concentrated load of 12 tons[a,c], or on each street car track, 24 tons[b,c]
Class C	Country roads with light electric cars or heavy highway traffic	A concentrated load of 12 tons[a,c], or on each street car track, 18 tons[b,c]
Class D	Country roads with ordinary highway traffic	A concentrated load of 6 tons[a] or 80 psf
Class E_1	Heavy electric street railways only	A concentrated load on each street car track of 24 tons[b]
Class E_2	Light electric street railways only	A concentrated load on each street car track of 18 tons[b]

[a] All concentrated truck loads are on two axles, 10 ft apart, and a 5-ft gauge. The truck is assumed to occupy a width of 12 ft.
[b] All concentrated street car track loads are on two axles, 10 ft apart.
[c] Classes A, B, and C, in addition to the concentrated load, must have a uniformly distributed live load of 100 psf on any remaining portion of the deck.

TABLE 2 **Waddell Specifications for Live Load**[a] *(1)*

Type	Description	Loading
Class A	Continued application of heavy loads (densely populated cities)	A concentrated load of 15 tons[b], 6 tons on front axle, 9 tons on back axle, 11 ft between axles
Class B	Occasional application of heavy loads (smaller cities and manufacturing districts)	A concentrated load of 8 tons, equally distributed on two axles, 8 ft apart. Wheel spacing is 6 ft
Class C	Ordinary light traffic (country road bridges)	A concentrated load of 5 tons, distributed as described in Class B

[a] These loads do not include electric trains.
[b] For Class A, the vehicle is considered to be a road roller. The width of the front roller is designated as 4 ft and the width of the rear wheel is to be 1 ft, 8 in. The two rear wheels have a spacing of 5 ft.

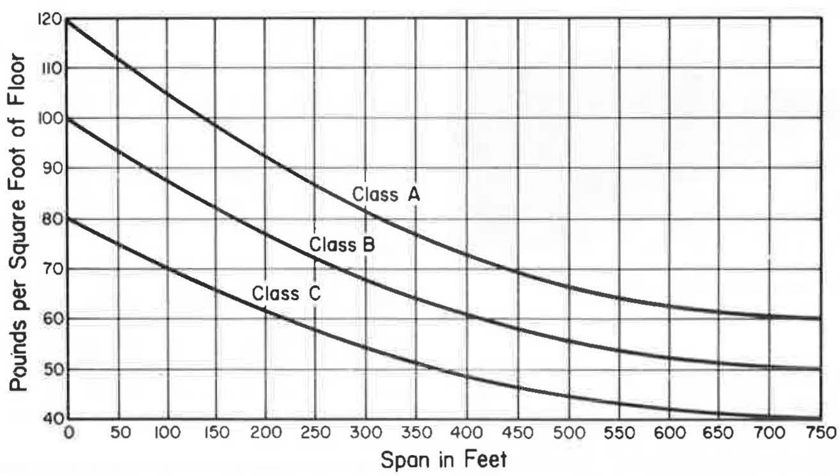

FIGURE 4 **Waddell's live loads for highway bridges** *(1)*.

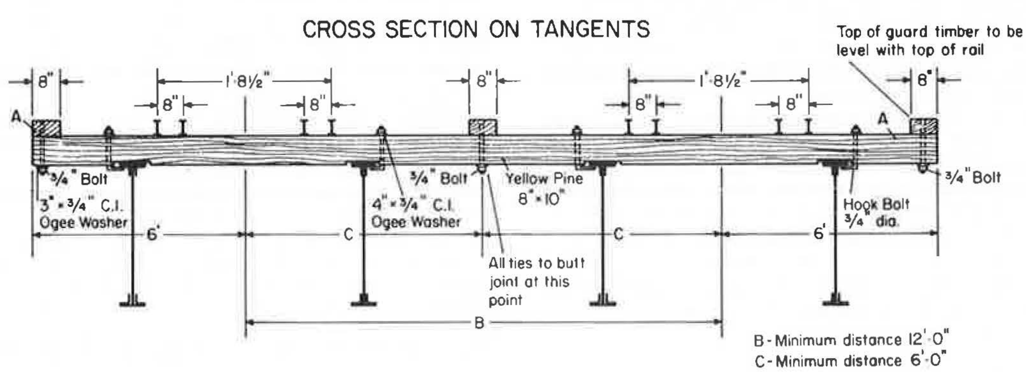

FIGURE 5 **Standard details of NYC and HRRR** *(1)*.

AASHTO Specifications (3) are, unfortunately, not as simple to interpret as the early specifications (1). The H 15 loading consists of a total truck weight of 15 tons. The distribution of the weight to the two axles spaced 14 ft apart is 3 tons to the front axle and 12 tons to the rear axle. The truck is assumed to occupy a lane width of 10 ft. The spacing of the wheels on an axle is assumed to be 6 ft. However, an equivalent lane loading is also provided for H 15 loading. The equivalent lane loading requires a uniform load of 480 lb per linear foot of loaded lane and, in addition, a single concentrated moving load of 13,500 lb for moment calculations and a single concentrated moving load of 19,500 lb for shear calculations.

To compare the current AASHTO H 15 loading with those required by early specifications, a typical 20-ft bridge structure is investigated. Table 3 gives the maximum moment and shear calculated from the three different loadings given in the specifications. For the AASHTO loading, the truck load was

TABLE 3 Comparison of Maximum Moment and Shear

Loading	Maximum Moment (ft-lb)	Maximum Shear (lb)
AASHTO–H 15 (15 tons, 14-ft axle spacing)	120,000	25,800
American Bridge Company–Class D (6 tons, 10-ft axle spacing)	33,750	9,000
Waddell–Class C (5 tons, 8-ft axle spacing)	40,000	8,000

used rather than the equivalent lane loading. The data in Table 3 indicate the dilemma faced by the county bridge engineer. The increase in moment requirements is approximately a factor of 3. Certainly one way to increase the load capacity of a long slender beam is to brace the beam. The desire is to prevent a stability failure (buckling) and ensure a material failure. The possibility of increasing the failure stress by a factor of 3 will be considered in the next section.

This example clearly shows the predicament an engineer encounters when trying to upgrade an old bridge to meet current truck loads. The engineers' ability to solve this problem may be further hampered by the designation of the bridge as a historic structure; thus, considerable ingenuity and skill would be required in restoring and strengthening (without changing the style and appearance of) the structure.

THEORY

The allowable moment capacity of a beam in bending is determined by multiplying its section modulus by the allowable stress. The allowable stress may be based on a material failure or a stability failure (buckling). An allowable stress based on a material failure will give the highest beam moment capacity. The allowable stress for buckling is always less than that for a material failure. Economy would seem to dictate a beam braced to prevent buckling, so that the highest beam moment capacity could be obtained.

It is well known that long rectangular beams with no lateral restraint may buckle. The lateral buckling of a beam is a function of the beam's torsional and flexural rigidity, loading, beam length, and bracing. Lateral buckling in timber beams may occur as a result of the compressive stress introduced in the top portion of the beam. This compressive stress is due to flexural bending of the member. This bending compressive stress can be thought of as creating an equivalent column-buckling problem in the compressive half of the beam. If buckling occurs, the beam will deflect laterally between points of bracing or support. Figure 6 shows the buckled configuration of a narrow rectangular beam. In practice, most beams are braced to prevent lateral buckling, thereby ensuring a material failure.

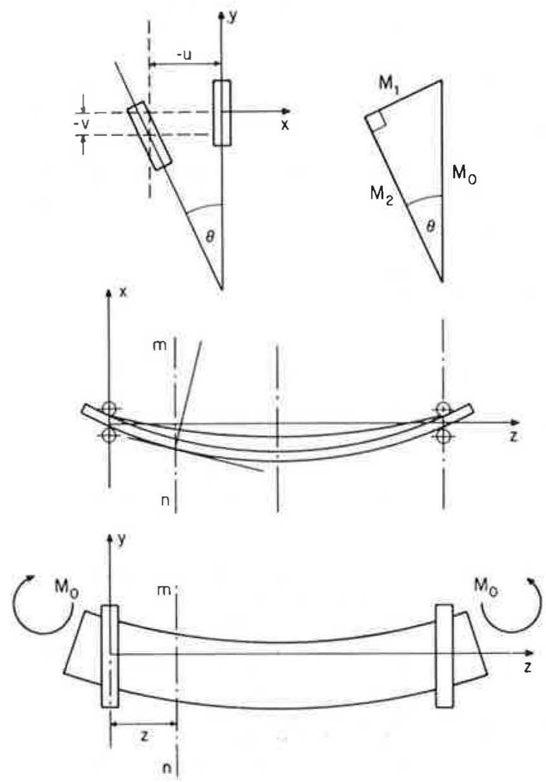

FIGURE 6 Buckled configuration of long narrow rectangular beam *(4)*.

In most cases, the question of lateral instability is eliminated by providing lateral support to the compression side of the beam at close intervals. If the deck members can be attached to the beams, lateral instability is prevented by continuous lateral support. If the deck cannot be attached, cross-bridging at regular intervals will also prevent the beam from buckling laterally.

Although it would seem reasonable to attach the deck to the beams, this is not always possible. Also, placing cross-bridging on an existing bridge may not be feasible, either economically or technically. The discussion of preventing lateral buckling has indicated a requirement of bracing the compression flange of the beam. Many times it is difficult, if not impossible, to connect bracing to the compression flange of a beam in an existing bridge. What about the possibility of tension flange bracing? Tension flange bracing is not as effective as a compression flange bracing. Depending on the beam properties, bracing the tension flange may be of little value. However, with long rectangular timber beams, tension flange bracing may increase the stress at which lateral buckling would occur. Unfortunately, tension flange bracing is not covered in any design code. The reason for this omission is the difficulty encountered when solving the differential equation for beam buckling supported at its tension flange. Bleich

(5) and Winter (6) have solved the problem for continuous tension side bracing. Roeder and Assadi (7) have verified experimentally the potential beneficial effect of continuous tension flange bracing for steel beams. They have also proposed a design equation for continuous tension flange bracing. Milner and Rao (8) have proposed a procedure for incorporating the point support bracing of the tension flange of steel beams in design codes. At present, there does not appear to be any solution to the effectiveness of tension side bracing for timber beams.

Effective bracing on the compression side of the beam requires the brace to be designed for strength and rigidity. The design requirements for the brace are necessary to force the beam to buckle in a higher mode of instability. If enough bracing is provided, instability will not occur and the beam will reach a material failure. Zahn (9), Zuk (10), and Winter (11) have proposed methods of designing braces for strength and rigidity. One of the easiest and most common is that proposed by Winter. Winter suggests treating the compression side of the beam as an equivalent column. He proposed that bracing requirements for beams be computed according to the following two steps (11):

1. "Determine the total compression force of the fully braced beam when the allowable stress multiplied by the safety factor is reached in the outer fiber."
2. "Determine the required characteristics for bracing against column buckling perpendicular to the plane of the loads of the compression portion alone."

In his book (12) McGuire presents examples of this method used to design braces for steel beams.

Another common practice in design is to require each lateral support to provide for 2 percent of the total compressive force that exists in the compression side of the beam (13). Zuk (10) has confirmed this practice to be satisfactory. He also believes that braces designed for such forces will have sufficient rigidity to prevent the beam from buckling in a lower mode.

Although the behavior of lateral buckling of beams and braces has been discussed, the design codes (3, 14) also provide some insight into the problem. In the designing of new timber beams or rating existing timber beams, AASHTO (3) provides design recommendations for laterally unsupported beams. The behavior of the beam is divided into three categories: short, intermediate, and long. A short beam will have a material failure, an intermediate beam will fail by inelastic buckling, and a long beam by elastic buckling. AASHTO (3) determines the allowable stress of intermediate and long beams by reducing the allowable material failure stress. This reduction is accomplished by the use of a slenderness ratio, C_s, which takes into account the dimensions of the member and the unsupported length. The beginning of the elastic buckling beam failure is denoted by C_k, which accounts for the modulus of elasticity and the allowable material stress in bending.

Figure 7, modified from Gurfinkel (15), shows how the allowable stress decreases in relation to the unsupported length of the member. F_b represents the allowable material stress and F'_b represents the allowable stress reduced for a stability failure. Note the classification of the beam on the right side of Figure 7. AASHTO (3) limits the slenderness factor to less than 50. For a $C_s \leq 10$, the allowable stress is equal to the allowable material stress. This requires a small vertical step at $C_s = 10$, as shown in Figure 7.

It is interesting to note the possibility of increasing the beam's allowable stress by a factor of

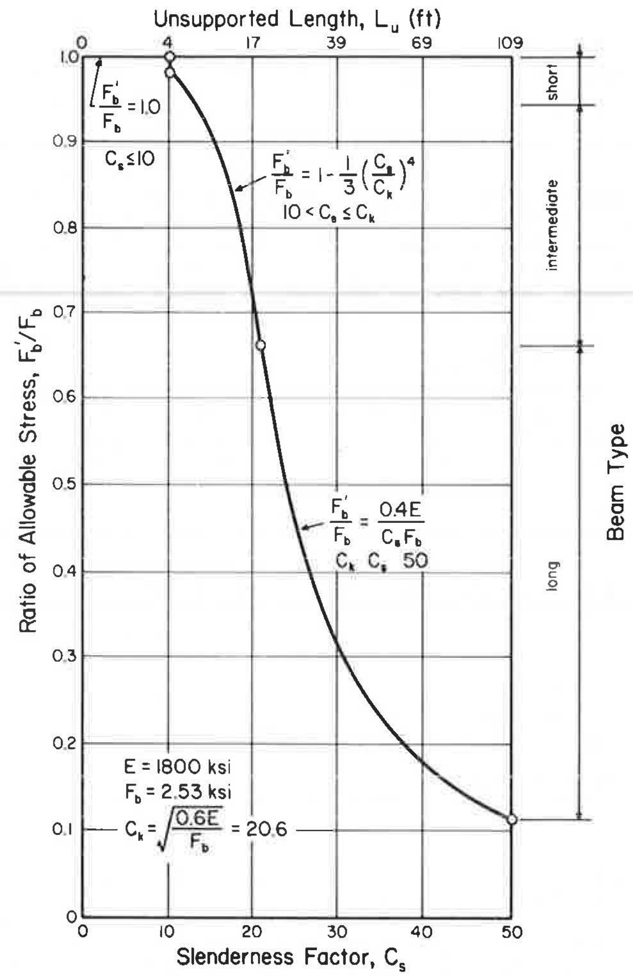

FIGURE 7 Variation in allowable stresses caused by laterally unsupported beam length (*15*).

10, at $C_s = 50$, by introducing continuous lateral support. It is evident from Figure 7 that providing lateral support to an existing laterally unsupported beam can greatly enhance the allowable stress and thereby increase the beam's moment capacity.

From this discussion, the benefits that lateral bracing can provide are obvious. A laterally unsupported beam supporting its design load of 5 tons in 1910 may be changed to a laterally supported beam capable of supporting 15 tons today. However, in practice, the attachment of lateral bracing, including connection of the deck to the beams, may not always be easily accomplished. The next section contains a discussion on practical solutions for accomplishing the lateral bracing of unsupported existing timber beams.

SOLUTIONS

To reiterate, the problem to which solutions are indicated is the lateral support of existing unsupported timber beams in old bridges. Because the beams are currently in use, the engineer may have to exercise considerable skill and ingenuity in proposing a lateral bracing scheme for the beams. The repair work may be hampered by construction difficulties, associated with a lack of working space (no room to swing a hammer or use a staple gun) and working over water, railroad tracks, or roads.

Most solutions of bracing the beams fall into two

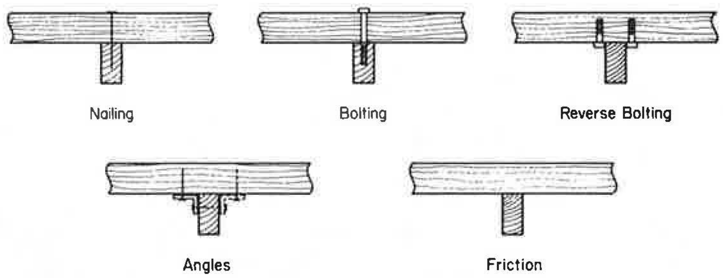

FIGURE 8 Deck-to-beam connections.

categories: deck-to-beam connections and beam-to-beam connections. Deck-to-beam connections include nailing, bolting, clip angles, and possibly a consideration of friction. Beam-to-beam connections include blocking, cross-bridging, and tension side bracing. Figures 8 and 9 show the possible connection techniques. All of these possible connections are discussed in the next two sections. No matter which connection technique is selected, the engineer should be careful to apply preservative treatment to any member that is penetrated. It does not make sense to solve one problem while creating another.

Deck-to-Beam Connections

Attaching the deck to the beams by nailing is analogous to attaching plywood floors to joists in house construction. This procedure can be an effective method of providing continuous lateral support. However, unlike houses, bridges experience vibration caused by vehicle traffic. It may be possible for the nails to "work out" from the beams. (This process has been witnessed by the senior author.) This is due to traffic moving across planks of different sizes and shapes, and unlevel deck bearing on the beams. This movement of the roadway deck tends to pry out the nails, making them more effective in puncturing tires than in providing lateral support. This effect can be even more pronounced if the roadway deck consists of two or more layers of different size and shape planks placed on top of each other. In one case, a 3-in. asphalt layer was placed over 2 layers of planks for a wearing surface. Because of the tilting and movement of the planks, the asphalt surface was broken up.

Bolting of the deck to the beam provides a greater resistance to uplift than nailing. However, depending on the degree of movement of the planks, the bolts may still be pried out.

Although bolting and nailing the deck to the beams is simple and may provide continuous lateral support, problems other than the "prying out" of the connectors may be encountered. It is not always an easy task to make sure the nail or bolt enters the beam below, much less near the center of the beam. Also, deterioration or rotting of the deck and beam may reduce the effectiveness of the connection. Although it would be easy to distinguish deck deterioration, it would be difficult to check the condition of the beam-deck interface. It is likely, however, that the three problems associated with nailing and bolting the deck to the beams (prying out, missing the beam, and deck or beam deterioration) could be overcome by using an abundance of connectors. It is not a highly scientific solution, but it is economical and practical.

An interesting deck-to-beam connection may be a form of reverse bolting. The connectors would be bolted to the underside of the deck as shown in Figure 8. The connectors should slide past the beam during movement. This requires that the unembedded portion of the bolt be long enough to maintain contact if the beam moves laterally. This type of connection does eliminate the three problems discussed for nailing and bolting.

Because it is possible to see where the bolt is being placed, there should be no problems with the connector not being adjacent to the beam. Also, it would be possible to check for deterioration of the deck at the proposed location of bolting. Beam deterioration would have little effect on the connection. As long as the connector slides past the beam for any upward deck movement, the aforementioned prying out of the bolt cannot occur. Also, the bolt (unless it protrudes through the deck surface) would not have any adverse effect on vehicle operating characteristics.

Certainly, the biggest construction problem associated with this connection is the cost of working underneath the bridge. A hanging scaffold may be required. Another problem is that of existing satisfactory performance. None of the authors have seen this type of connection used before and do not have any test results as to its actual field performance.

Another underside deck-to-beam connection is that which encompasses clip angles and bolts. This has all the advantages and disadvantages indicated for reverse bolting. This connection is more time consuming than reverse bolting, but it also provides a more positive connection. If the bridge was experiencing not only a lateral support problem, but also a problem of distributing wheel loads to beams, this connection would be better than reverse bolting.

The discussion of a friction connection has been saved for last. It is doubtful that the friction between the deck and beam could account for the necessary lateral force to brace the beam. According to Zuk (10), the friction force would have to be 2 percent of the total compressive force in the beam. This friction force would also have to be consistent over a long period of time. The bridge vibrations introduced by traffic would be enough to break any chemical adhesion or bonding between the deck and beam. Many designers have recommended against using

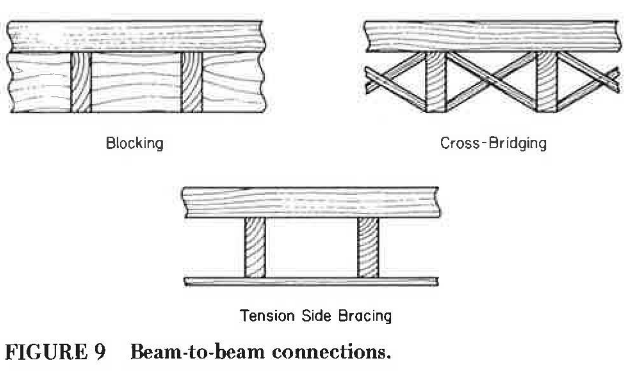

FIGURE 9 Beam-to-beam connections.

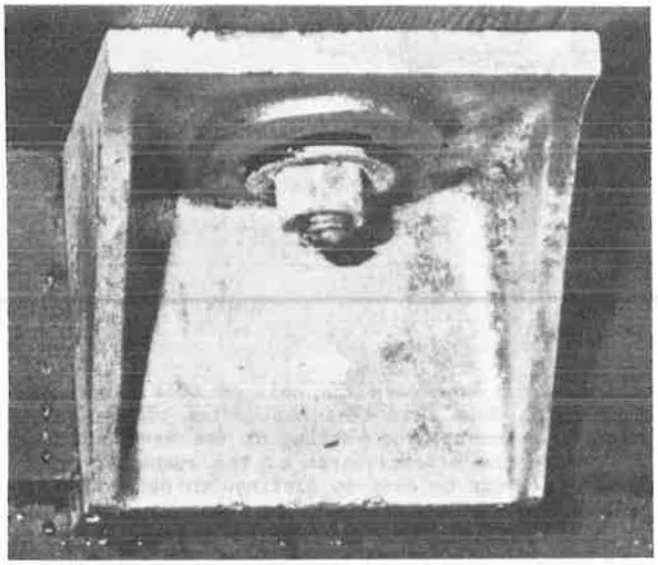

FIGURE 10 Weyerhaeuser deck bracket *(16)*.

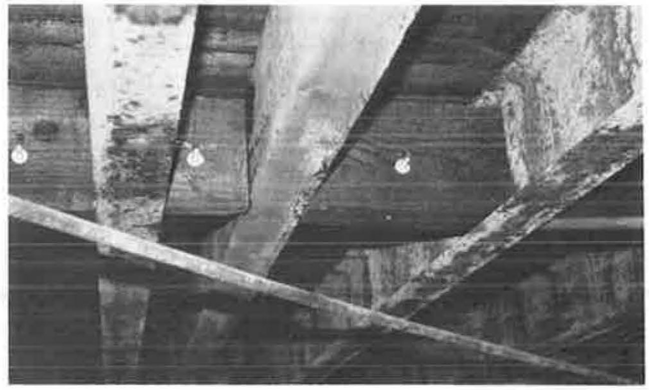

FIGURE 11 Blocking between beams.

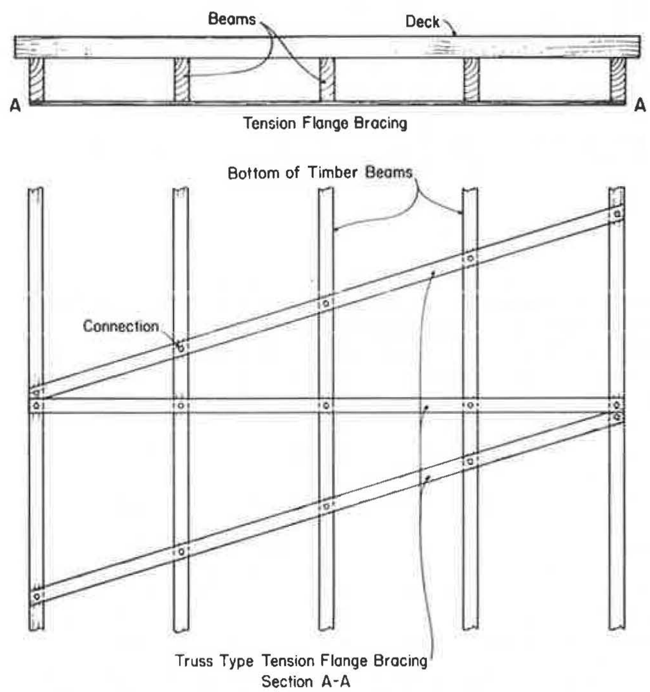

FIGURE 12 Tension side bracing.

friction as lateral support between a concrete slab placed on steel beams for buildings. This situation would have a higher normal force (dead weight of concrete), a wider bearing area (concrete against steel flange), and presumably no vibration. Therefore, if friction is not considered as lateral support for beams in buildings, it definitely should not be considered as useful in bridge structures.

Another type of deck-to-beam connection is a deck bracket (Figure 10) manufactured and marketed by Weyerhaeuser (16). This deck bracket, which is intended for use in panelized glued-laminated deck systems, could also be used to support a beam. This deck bracket, while bolted through the deck, is fitted into a routed groove on the beam (17). The formed teeth in the deck bracket firmly grip the beam and deck when the bolt is tightened. The deck bracket was developed to minimize deterioration resulting from connectors breaking through the treated envelope of members. Although it is not known whether this deck bracket has been used to provide lateral support of beams in existing bridges, it is a possibility. The biggest problem would be routing the groove in the side of the beam.

Beam-to-Beam Connections

AASHTO (3) currently requires timber beams in new bridges to have cross-bridging. Figure 9 shows both cross-bridging and blocking, which can be used effectively to support beams. Blocking and cross-bridging both must be applied under the bridge from hanging scaffolds. The biggest construction problem associated with both blocking and cross-bridging is fitting the new bracing between existing beams. Figure 11 shows the difficulty associated with blocking between timber and steel beams. (Note that this figure is for the bridge shown in Figures 1-3.) It took county personnel two weeks to install the blocking for these beams. It would be interesting to guess how effective this blocking is, due to the noticeable and unavoidable gaps between the blocking and beams. Note also how the distance varies between the beams, making field cuts of all blocking from the hanging scaffold a necessity.

Tension flange bracing as shown in Figure 12 would certainly speed up the construction process. The braces could be bolted into the bottom of the timber members. A truss type configuration could be used as shown. This system could work well for a bridge that combines steel and timber beams. The tension side bracing could be bolted to the bottom of the timber beams and L-shaped bolts could be used to clamp on to the bottom flange of the beam. (See Figure 5, which shows this type of clamp connection at the top of the steel flange.) Tension side bracing would be the easiest of the beam-to-beam type connectors to construct.

Unfortunately, the effectiveness of tension side bracing for timber beams has not yet been established technically. Therefore, it would be difficult to suggest a brace spacing to satisfy the lateral support requirements of the beam. Also, tension flange bracing could, during extremely high water, entangle objects floating in the water as they passed under the bridge.

CONCLUSION

Older bridges designed with long, laterally unsupported beams are not capable of carrying the truck loads currently imposed. The beams must be laterally braced to increase the load carrying capacity. There are methods available to calculate the strength and rigidity of lateral bracing. Current design codes enable the engineer to select the lateral brace spacing to achieve a given level of allowable stress.

The advantages and disadvantages of deck-to-beam connections and beam-to-beam connections are presented. Tension side bracing of beams may be an economical approach in the future, but no practical solution currently exists to determine the effectiveness of that bracing technique.

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Ultimate Strength of Timber-Deck Bridges

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ABSTRACT

Contained in this paper is a discussion on the procedures for evaluating the ultimate strength of timber deck bridges. Three structural systems are considered: sawed timber stringers, nailed laminated decks, and prestressed laminated decks. The load and resistance are treated as random variables. Their parameters are determined on the basis of material tests, load surveys, and analysis. The bridge performance is measured in terms of the reliability index. Various safety analysis methods are discussed. The procedures used in calculations were selected on the basis of accuracy, requirements for input data, and simplicity of use. System reliability models were used to include load sharing between deck components. Reliability indices were calculated for three structures. It has been observed that the degree of load sharing determines the safety level. Reliability is highest for the prestressed laminates. Sawed stringers can be considered as a series system in the system-reliability sense, with a rather limited redundancy. Therefore, the corresponding reliability indices are relatively low. Prestressed timber structures are recommended. The major advantages are a better load distribution, a better control of wood deterioration, and a relatively low cost of application.

The use of wood in constructing bridges (note that timber bridges are common in many parts of North America) has various advantages and disadvantages. For example, the wood is generally available (especially in the northern and northwestern parts of the continent), is renewable (cut trees may be replaced by new ones), is relatively light-weight, and has good dynamic damping properties. On the other hand (a) wood has a high degree of variation of strength (modulus of rupture), (b) its performance strongly depends on the environment, (c) it deteriorates and drastically loses its strength under unfavorable conditions, (d) its load-carrying capacity decreases under prolonged load duration, and (e) it is vulnerable to fire. Therefore, it is desirable to control the strength and deterioration of timber structures. The new improved technology allows for a better control of the structural performance.

The strength of timber is usually controlled by a visual inspection; however, visual (or even mechanical) inspection does not allow for elimination of the variation in strength. Special structural systems may allow for a better load distribution so that weaker members take smaller loads and stronger members take larger loads. Deterioration can be controlled by chemical treatment, however, which may also serve as a protection against fire. However, bridge loading and load-carrying capacity (resistance) are random variables. Therefore, the bridge behavior cannot be predicted without uncertainty.

Safety analysis methods have been developed in the last 15 years (1). The accuracy of the available procedures depends on the required data on load and resistance, computer capacity, and the level of effort. Application of the system-reliability theory reveals some hidden safety reserve in the structure. This is particularly useful in the evaluation of existing bridges. In this paper, safety analysis methods are applied to evaluate the ultimate strength of timber deck bridges. The deck structure is treated as a system of interacting, partially correlated (in the statistical sense) members. Material properties are modeled using the available test data. The analysis is demonstrated on the examples of typical structures.

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TIMBER DECK STRUCTURES

Three typical deck structures are considered in this paper: timber sawed stringers, nailed laminated deck, and prestressed laminated deck.

In the first system (Figure 1), the laminated deck is supported by solid section stringers. Sawed timber stringers are usually spaced between 15 and 30 in. with respective spans of 10 to 24 ft and a deck thickness of from 3.5 to 5.5 in. The ultimate strength is reached when either the deck planks or any of the stringers rupture. The contact area between the truck wheel tire and the pavement is large enough to ensure a uniform deflection of several adjacent deck planks (over a width of about 2 ft). Therefore, the load per plank can be considered proportional to the corresponding modulus of elasticity (MOE). Planks transfer the load to stringers. Load per stringer depends on the stiffness and spacing of the stringers as well as stiffness of the deck planks.

A typical laminated deck is shown in Figure 2. The laminates are usually made of boards sized 2 x 8 in. to 3 x 10 in., with spans of 10 to 20 ft. Laminations may be parallel or perpendicular to traffic direction. The boards are interconnected by nails, which allow for the transfer of load to adjacent boards. However, the nails often become loose and ineffective after several years of operation.

Observations indicate that the degree of load sharing by the adjacent boards is limited. The deck deflection is uniform under the contact area between the tire and pavement, and it is almost zero outside this area. The ultimate strength is reached when the deck unit (a number of boards immediately under the truck tire) ruptures. The failure mechanism and probability are discussed later in this paper.

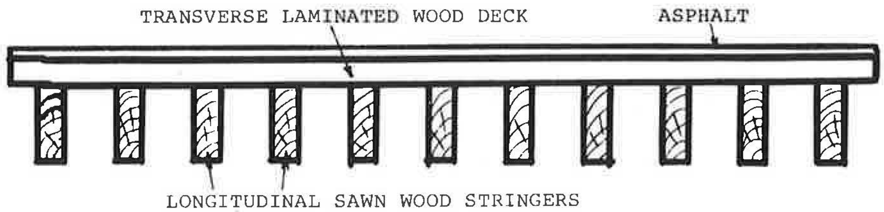

FIGURE 1 Sawed timber stringers.

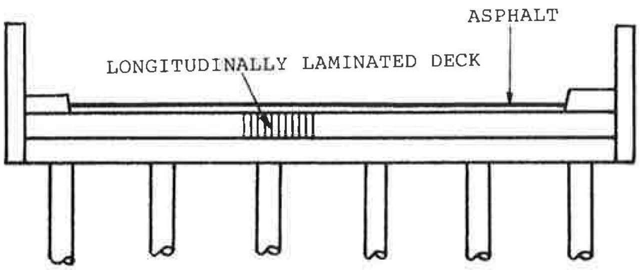

FIGURE 2 Nailed laminated deck.

Prestressed laminated decks were first introduced in Ontario over 8 years ago (2). The prestressing force was applied to an existing laminated deck that showed signs of severe deterioration due to loosening of the nails holding the laminates together. This transverse pressure made the deck tight and significantly improved the overall performance of the bridge. In the last several years, this prestressing method has also been used in many newly designed bridges. The details of prestressing bars for existing and new bridges are shown in Figures 3 and 4, respectively.

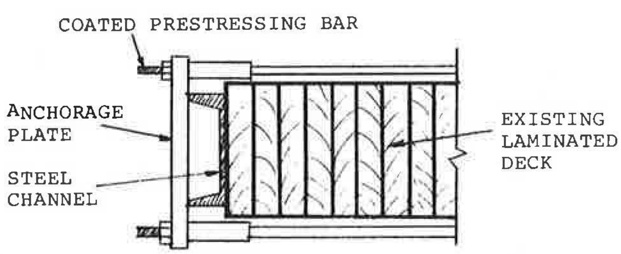

FIGURE 3 Details of prestressing for an existing laminated deck.

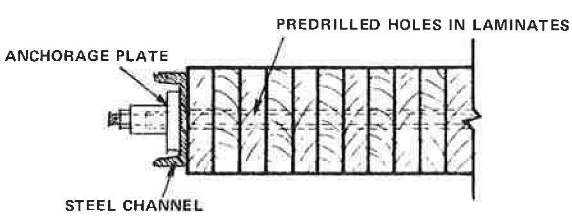

FIGURE 4 Details of prestressing for new construction.

Prestressing considerably improves the shear force transfer between laminates (load sharing). The deflection of a prestressed deck is much more uniform than the nailed decks. Because of a considerable load-sharing effect, the failure mechanism is rather complicated. In the analysis, the deck is divided into units, each with a width of about 2 to 3 ft. The ultimate strength of a unit is modeled on the basis of the available test data.

BRIDGE LOADING

The major components of bridge loading include dead load, live load, and environmental effects.

In timber bridges, dead load (D) constitutes about 10 to 20 percent of the total load. The weight of timber is assumed to be 50 lb/ft^3 and, for the purposes of this paper, a square foot of the deck typically is assumed to weigh about 100 lb for sawed stringers with laminated decks, and about 50 lb for a laminated deck. Asphalt weight depends on the applied thickness, which may vary from almost zero to 9 in. (3). An average thickness of 3 in. was assumed, which is equivalent to about 30 lb/ft^2.

Statistical distribution of dead load is well established for buildings (4). In the case of bridges, the distribution is somewhat different. The material (timber in this case) is the same as in buildings; however, the dimensions are often much larger for bridges. Also bridge construction is usually handled by experienced firms, so that a higher quality level of workmanship can be maintained. This also means a smaller variation of the weights.

In the probabilistic analysis, it is assumed that dead load is normally distributed. The ratio of mean-to-nominal is taken as 1.05 and the coefficient of variation as 10 percent. Live load is the most important load component for timber bridges. Static and dynamic parts of live load are considered separately.

The static effect (L) depends on wheel force, wheel geometry (configuration), vehicle position on the bridge, number of vehicles on the bridge (multiple presence), stiffness of the deck and stiffness of the stringers. Because of complexity of the model, the variation in vehicle load and the variation in load distribution properties of the bridge are considered separately.

Truck weight is modeled on the basis of a truck survey carried out in Ontario (5). About 10,000 vehicles were measured including axle weights and interaxle spacings. For these trucks, midspan moments were calculated for various simple spans (3). Moments per stringer or per laminated deck unit were determined using a specially developed computer program (6).

Typical span lengths in timber decks are 10 to 20 ft, and they essentially do not exceed 30 ft. Therefore, only one or two closely spaced axles can be on the bridge simultaneously. The mean maximum 50-year live load was modeled using exponential distribution (3). It turned out to be equivalent to a single axle of 44 kips or two axles of 30 kips each spaced at 4 ft. The coefficient of variation of L was taken as 11 percent.

The dynamic portion of live load, I, depends on various parameters such as dynamic properties of the bridge (natural frequency of vibration), surface conditions, and mechanics of the vehicle. Natural frequency is a function of dead weight and span, both of which are rather small in case of a timber deck. However, presence of potholes or bumps (especially on the approach) drastically increases the dynamic load. In colder regions, this may be caused by an uneven accumulation of snow and ice.

Observations carried out by the Ontario Ministry of Transportation and Communications indicate that dynamic load cannot be ignored in timber bridges (unpublished study). Even though AASHTO does not require the use of impact in the design, the new Ontario Highway Bridge Design Code (7) specifies dynamic load allowance at 70 percent of the value specified for other materials (steel or concrete). In this study, it is assumed that the mean maximum static live load (as described above) occurs simultaneously with dynamic load equal to 10 percent of L for spans up to 10 ft, and 15 percent of L for spans up to about 30 ft. The coefficient of variation of I is assumed as 45 percent (3).

Other loads may be critical in some design cases. However, a simultaneous occurrence of a heavy truck with an extreme value of some environmental load (wind, earthquake, snow, ice, temperature) is unlikely. There are several load combination models available. Good results were obtained by use of "Turkstra's rule" (3). Turkstra observed that the maximum of a load combination occurs when one load component takes on its maximum value while the other components are equal to their average values. In this paper, only dead load and live load combinations are considered.

PROPERTIES OF MATERIAL

The behavior of a bridge deck depends on mechanical properties of its components such as stringers, deck planks, nails, and prestressing rods. The most important characteristics of timber are the modulus of rupture (MOR) and the modulus of elasticity (MOE). The statistical models were developed on the basis of available test data including flexural tests of beams and prestressed units (8). There is a need for more data about the performance of nailed laminates, in particular as a function of time (deterioration). [Note that flexural tests were performed by Madsen at the University of British Columbia (9). The statistical models for MOR and MOE were developed by Nowak (10).]

It has been observed that MOR is featured with a considerable scatter. Tests were performed for thousands of specimens, 200 or 300 at a time, and each time, a different distribution function was obtained. Therefore, for each species and grade, MOR is described by a family of distribution functions. Models were developed to represent each family by a function (average function) and a coefficient of variation (of the mean value). The coefficient of variation serves as a measure of the scatter within each family.

A function representative for a family can be approximated by three lognormal distributions; one for the lower range of values, one for the center, and one for the upper tail. The parameters of representative functions (mean and coefficient of variation), and the corresponding coefficients of variation of the mean, are given in Table 1 for several species and grades.

The distribution functions of MOE were modeled in a similar way. It has been observed that, even though a considerable scatter is involved, the distribution function seems to be independent of the size, and depends on the species and grade only. This function can be represented by a lognormal distribution and the derived parameters are given in Table 2.

MOR and MOE are partly correlated in the statistical sense. In the analysis, the correlation is simulated using the Monte Carlo technique. MOR is treated as an independent variable. For a given value of MOR, the corresponding MOE is assumed to be lognormally distributed with the coefficient of variation equal to 20 percent and with the mean (m_{MOE}) expressed as

$$m_{MOE} = a_1 \text{ MOR} + a_2 \tag{1}$$

where a_1 and a_2 are constants.

If MOR is expressed in kips per in.², then the best fit for testing data is obtained for $a_1 = 150$ and $a_2 = 700$ ksi, for all sizes, species, and grades considered. The resulting MOE is also in kips per in.².

The prestressed laminates were tested at the Western Forest Products Laboratory (11). Sawed timbers of 2 x 10 x 16 ft were made into units with widths of 1, 2, and 3 ft. The corresponding numbers of boards were 8, 16, and 24, respectively. Three species were considered: hem-fir, white pine, and red pine. The units were transversely prestressed by threaded reinforcing bars, spaced at 28 in.

The results of the ultimate strength tests indicate a relatively small scatter for each species. The distribution function of MOR for the unit can be approximated by a normal distribution with the coefficient of variation ranging from about 15 percent

TABLE 1 Parameters of MOR Distribution for Selected Species and Grades

Species, Grade, Size (in.)		Average Distribution Function: Mean (psi)	Average Distribution Function: Coefficient of Variation (%)	Coefficient of Variation of the Mean (%)
Douglas-Fir				
Select	2 x 8	5,500	25	28
	2 x 10	5,200	27	30
	3 x 8	6,200	22.5	25
	6 x 16	6,200	22.5	25
Grade 1 and 2	2 x 8	4,100	32	28
	2 x 10	3,850	32.5	30
	3 x 8	4,800	29	25
	6 x 16	4,800	29	25
Hem-Fir				
Select	2 x 8	6,600	30.5	28
	2 x 10	6,200	31.5	30
Grade 1 and 2	2 x 8	4,750	37.5	28
	2 x 10	4,350	38	30
Spruce-Pine-Fir				
Select	2 x 8	5,400	29.5	28
	2 x 10	4,950	27.5	30
Grade 1 and 2	2 x 8	4,800	32.5	28
	2 x 10	4,500	33.5	30

TABLE 2 Parameters of MOE Distribution for Selected Species and Grades

Species and Grades	Average Distribution Function: Mean (10^3 ksi)	Average Distribution Function: Coefficient of Variation (%)	Coefficient of Variation of the Mean (%)
Douglas-Fir			
Select	1.50	20	20
Grade 1 and 2	1.29	21	20
Hem-Fir			
Select	1.65	18	20
Grade 1 and 2	1.50	20	20
Spruce-Pine-Fir			
Select	1.50	17	20
Grade 1 and 2	1.375	20.5	20

for 1-ft width to 8 percent for 3-ft width. The mean MOR is practically the same for all unit widths.

The theoretical model for MOR of prestressed units has been developed by Nowak and Taylor (8). The unit can be considered as a system of interconnected boards (elements). The system is parallel, because the unit fails only after all its components fail. It has been observed that individual boards retain their ultimate strength after the maximum stress is reached, and the strain can be increased by about 10 percent to 20 percent before the final failure occurs. Furthermore, rupture of a board is a very localized phenomenon, and its occurrence reduces the load-carrying capacity of the board only in the immediate neighborhood of the defect.

It can be assumed that the load-carrying capacity of the unit is equal to the sum of capacities of its components (boards). Flexural tests indicated that strengths of boards in the deck can be treated as independent variables. The distribution of the capacity for the unit can be derived from the strength distributions of the components. The number of boards in a unit varies from 8 to 24. Therefore, by using the central limit theorem of the theory of probability, the distribution of the unit capacity becomes normal, regardless of the distribution of components. The mean unit capacity is equal to the sum of means of components, and the coefficient of variation, V_u, is equal to

$$V_u = V_1/n^{1/2} \tag{2}$$

where V_1 = the coefficient of variation of MOR for a component (single board), and n = the number of boards in a unit.

The comparison of the theoretical and observed parameters (means and coefficients of variation) for the prestressed units shows a good agreement. The results are presented in Table 3.

TABLE 3 Comparison of Theoretical and Observed Parameters of Prestressed Deck Units

Unit Width (ft)	Species	Mean, Ratio of Theoretical to Observed Value	Coefficient and Variation (%)	
1	Hem-Fir	.98	13	16
	White Pine	.82	15	15
	Red Pine	1.02	15	21
2	Hem-Fir	1.05	9	9
	White Pine	.75	10	7
	Red Pine	.96	10	14
3	Hem-Fir	1.02	9	8
	White Pine	.87	8	8
	Red Pine	.97	8	7

STRUCTURAL RELIABILITY ANALYSIS

Reliability of the structure is the probability of performance without failure. There are various forms of failure: from minor limitations of use (e.g., cracking), local failures (exterior girder twisted by high trucks), to overall collapse. This paper deals with failures that directly affect the load-carrying capacity. The reliability models are developed for three systems of timber-deck bridges.

Probability of failure (P_F) can be expressed as the probability of load effect (Q) being larger than load-carrying capacity or resistance (R)

$$P_F = \text{Prob}\ (Q > R) \tag{3}$$

Direct use of Equation 1 is not possible in practical situations because of numerical difficulties involved in computation of convolution functions. It is convenient to measure structural safety in terms of a reliability index (B) defined as

$$B = -F_N^{-1}(P_F) \tag{4}$$

where F_N^{-1} = the inverse of the standard normal distribution function (1).

Cornell developed the following formula for B,

$$B = (m_R - m_Q)/(s_R^2 + s_Q^2)^{1/2} \tag{5}$$

where

m_R = mean value of R,
m_Q = mean value of Q,
s_R = standard deviation of R, and
s_Q = standard deviation of Q.

If R and Q are normally distributed, then

$$P_F = F_N(-B) \tag{6}$$

Rosenblueth and Esteva developed a simple logarithmic formula for B, which is expressed as

$$B = \ln(m_R/m_Q)/(V_R^2 + V_Q^2)^{1/2} \tag{7}$$

where V_R = the coefficient of variation of R and V_Q = the coefficient of variation of Q. If both R and Q are lognormally distributed then P_F can be calculated using Equation 6. However, in most practical cases, neither R nor Q are normal or lognormal.

There are various methods available for calculating B in a general case. Good results are obtained by using a procedure developed by Rackwitz and Fiessler (1). The method is based on normal approximations of nonnormal functions at the design point. An easy-to-use graphical variant of the procedure was developed by Nowak and Regupathy (12).

The reliability index can also be calculated by Monte Carlo simulations. This is particularly efficient in cases of complicated distribution functions and partially correlated variables.

The aforementioned methods were used in the reliability analysis of bridge decks. Load distributions were generated by Monte Carlo simulations and then further calculations followed the Rackwitz and Fiessler procedure. In case of normal or lognormal distributions, either Equation 5 or 7 was used.

RELIABILITY OF BRIDGE DECKS

In the reliability analysis, it is assumed that the bridge fails when the whole span, or a large portion of a span, loses its load-carrying capacity. In practice, the deck supported by sawed stringers will fail after the rupture of a single stringer. It is impossible to transfer the load from the overloaded stringer to other ones. The degree of load sharing is considerably higher in deck planks or laminates where one broken element does not necessarily result in an overall failure. The situation is even better in the case of prestressed units. The load sharing results in a more uniform deflection of the deck. Failure occurs only after a large number of single boards reach their ultimate loads. In the analysis, it is assumed that the ultimate strength of a stringer bridge is reached when any of the stringers ruptures. This assumption was supported by the analysis performed by Nowak and Boutros (6).

The bridge structure was modeled as a system of finite strips (stringers with transverse deck planks). The mechanical properties (MOR and MOE, partially correlated) of stringers were generated using the Monte Carlo technique. Stresses were calculated for mean maximum 50-year live load. For each stringer, the ratio of calculated stress to MOR was determined. The minimum ratio, r_{min}, for the bridge determines the safety level for the considered run. The runs were repeated to develop the distribution function for the minimum ratio. The probability of failure is equal to the probability of r_{min} being larger than 1.

The capacity of a nailed laminated deck is determined by the strength of a deck unit with a width corresponding to the truck tire contact area. Measurements of deflections performed by Csagoly and Taylor (2) indicate a limited degree of load sharing in such bridges after several years of operation. Therefore, the probability of failure of the deck is determined by the behavior of the deck units.

Performance of a deck unit is determined by the mechanical properties of its components. The distribution of MOR for a unit is developed by Monte Carlo simulations as follows. Values of MOR and MOE are generated first, as in the case of stringers. All components of one unit (2-ft wide) are subject to the same deflection under loading (this is forced by the tire width). The first element (board) to reach its MOR is identified. It is assumed that in each board, the stress-strain relationship is linear up to MOR, then stress remains constant until strain is increased by 10 percent, and then rupture occurs. This behavior model is idealized on the basis of flexural tests. In the analysis of a deck unit, it is also assumed that any broken board retains its strength until the deflection is increased by 10 percent. After the first board is eliminated, all the load is redistributed to the remaining boards, next board reaching its MOR is identified, and so on. The ultimate strength of the unit is the largest moment applied in the elimination process. Typically, up to 3 boards were eliminated before the maximum strength was reached. The unit properties were generated repeatedly so that the distribution of ultimate strength could be developed.

The load per deck unit was taken as one-half of truck (wheel) load. The mean maximum 50-year wheel load is either 22 kips or 2 15-kip forces spaced at 4-ft intervals. Reliability indices were calculated using the Rackwitz and Fiessler procedure.

To determine the ultimate strength of a prestressed deck, the structure is modeled as a system of units, each with a width of 2 to 3 ft. Observations of deflections of several bridges in Ontario indicate a high degree of load sharing between the units. The maximum load per unit is about 60 percent of the wheel load. The distribution of ultimate strength of a unit was derived from test data (11). Live load was taken as 60 percent of maximum 50-year truck load. As in the case of laminated nailed decks, the reliability indices were calculated using the Rackwitz and Fiessler procedure.

NUMERICAL EXAMPLES

Three typical timber-deck bridges were selected. The structures were designed using AASHTO specifications (13). In these specifications, there are no special provisions for prestressed decks. Therefore, a nailed laminated deck was designed for both the prestressed and nonprestressed case.

The sawed-timber cross section is shown in Figure 1. The stringers, which are 6 x 16 in., and deck planks, which are 2 x 6 in., are made of Douglas-Fir Select Structural. There are 18 stringers, spaced at 18 in. (center-to-center). The simply supported span is 16 ft and the reliability index for the bridge is B = 4.2.

The nailed laminated deck (Figure 2) is made of 3 x 8-in. Douglas-Fir Select Structural. The deck width is 27.5 ft, the span is 9.5 ft, and the reliability index is B = 3.0.

The prestressed deck is designed identically to the nailed laminated structure described previously. The prestressing rebars are added and they are spaced at 28 in. The reliability index for the bridge is B = 7.5. The reliability index for the prestressed unit is much higher than B for the other two designs. This indicates a potential safety reserve and advantage of using the prestressing technique.

CONCLUSIONS

The ultimate strength of three timber bridge systems has been evaluated with safety considered as the measure of structural performance. It was determined that system reliability theory allows for a more realistic analysis of the load-carrying capacity.

Reliability indices were calculated for a sawed timber bridge, a nailed laminated deck, and a prestressed laminated deck. The safety reserve is proportional to the degree of load sharing. The tests and calculations indicated that prestressing considerably improved structural performance.

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Load Distribution Criteria for Glued-Laminated Longitudinal Timber Deck Highway Bridges

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ABSTRACT

In recent years, changes in the load distribution criteria for highway bridges have been limited primarily to steel and concrete bridges, and only a few changes have occurred in timber bridge criteria. Several years ago, the AASHTO Bridge Committee approved the inclusion of criteria for glued-laminated stringer bridges. However, the glued-laminated, longitudinal, timber-deck bridge developed in recent years has been subject to code specifications that do not reflect the favorable load distribution characteristics of the bridge. Sufficient test data now exist to verify the distribution behavior. The purpose of this study was to develop criteria that more accurately represent the bridge behavior. The study was conducted in three phases. Phase One was a literature review of both analytical and experimental investigations of load distribution in timber-deck bridges. Phase Two analytically investigated the distribution characteristics of a broad range of glued-laminated, longitudinal, timber-deck bridges. The analytical procedure used was verified by comparison of theoretical results with results from actual field tests. In Phase Three, criteria for inclusion in the appropriate sections of the AASHTO Bridge Specifications were developed and have been approved by the AASHTO Bridge Committee. The new criteria more adequately reflect the load distribution of the longitudinal deck bridge.

During the last 25 years, many studies have been conducted to develop new or improved criteria for the distribution of wheel loads on highway bridges. These studies have been used to provide new criteria for improved and different types of construction and have resulted in a number of changes in existing criteria for steel and concrete bridges. For many years, the design criteria for timber bridges remained unchanged in the AASHTO Standard Specifications for Highway Bridges (1). Research in the area of timber bridge structures has led to recent changes in some sections of the specifications. Studies such as the one conducted by Iowa State University (ISU) for the American Institute of Timber Construction (AITC) in 1979 showed the benefits derived from glued-laminated bridge members. In this study, which was conducted on glued-laminated timber bridges with longitudinal stringers, load distribution criteria for the stringers were developed that showed improved distribution behavior over previous criteria. A study by McCutcheon et al. (2) described a design technique for the glued-laminated panels supported by stringers. As happened with the study by Sanders, AASHTO adopted McCutcheon's design procedure and thus included it in the bridge specifications.

The recent development of another type of timber bridge has created the need for additional design criteria. This glued-laminated longitudinal timber-deck panel bridge requires no stringers and can span distances approaching 40 ft. The system consists of panels approximately 48 in. wide, connected by stiffener beams placed transverse to the direction of the span (see Figure 1). Multiple panels are arranged to provide the required roadway width. No positive shear transfer device (e.g., dowels) except

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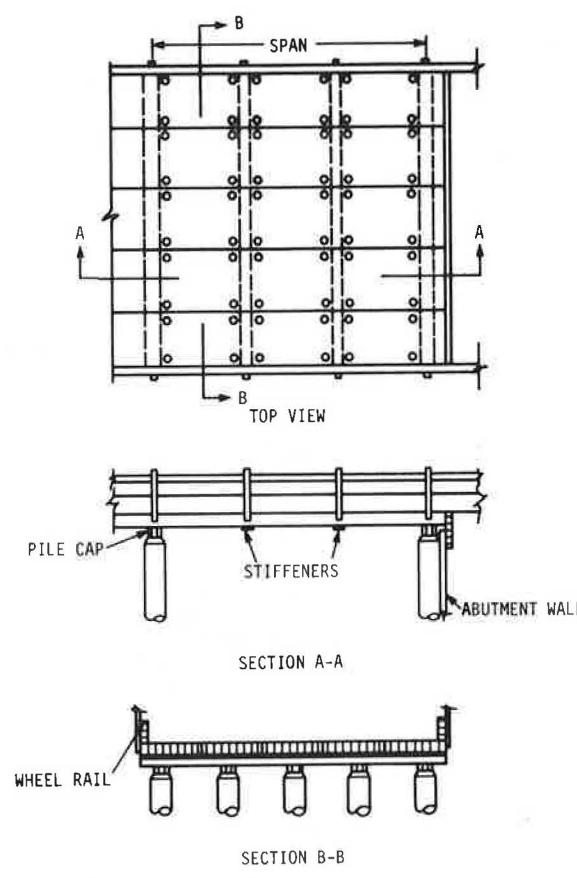


FIGURE 1 Glued laminated longitudinal timber deck highway bridge.

the stiffener beam is normally provided between panels. Presently, there are no clear design criteria for this type of bridge. Over the past few years, research by private industry, government laboratories, and universities has been conducted on this bridge type. With these experimental data, Iowa State University (ISU), under the sponsorship of AITC, conducted a study to develop design criteria for live-load distribution that more realistically reflected the behavior of this type of bridge and to standardize the design procedure.

BACKGROUND

Interest in timber construction for short-to medium-span bridges has increased in recent years and has brought new ideas for more economical designs for timber bridges. One such bridge type was developed in the 1970s by Stone at the Weyerhaeuser Company in Tacoma, Washington (unpublished data). The bridge has no stringers and, therefore, is useful in situations where adequate vertical clearance is a problem. Although many of these bridges have been constructed since their development, lack of clear and concise design criteria has probably precluded their use in some instances. The need for such design criteria has been widely known within the timber industry for some time.

The previously mentioned work by Stone involved full-scale bridge tests as well as tests on various components of the bridge to provide insight into bridge behavior. An analytical study was subsequently performed by Evans (unpublished data) using experimental data from the previously described tests to provide insight into a design technique for the bridge.

Although design criteria that are applicable to this bridge type currently appear in the AASHTO specifications, they are not clearly defined. Recent research efforts and experience show that these criteria do not seem to describe the bridge behavior realistically.

OBJECTIVE AND SCOPE

The research program was conducted in three phases. Phase One involved a thorough literature review of all documented research on the behavior of timber bridges in general and, more specifically, on the behavior of timber bridges composed of the glued-laminated longitudinal deck panels. Both analytical studies and field test studies were included in this review. As a result, a method of analysis was selected for the development of the analytical model to be used to study the bridge behavior.

Phase Two involved a survey of standard highway bridges that incorporated the glued-laminated longitudinal deck panels. The survey included discussions with design engineers and manufacturers who had experience in the design and construction of this type of bridge. The intent of the survey was to provide information on the sizes and structural configurations of the longitudinal bridges typically in use. From this information, it was possible to determine ranges of values of the key variables for use in the model study.

Phase Three used information from Phase Two for the development of an analytical model to study load distribution behavior. From the analytical study, design criteria were developed. The criteria use current distribution criteria formats and have been approved for inclusion in the AASHTO specifications (1).

LITERATURE REVIEW

Before an analytical model for studying the load distribution behavior of the bridge was developed, a review of standard highway bridges using glued-laminated longitudinal deck panels was performed. This review determined the practical limitations of the bridge geometrics and quantified the parameters that describe the bridge so that an accurate model of the bridge could be developed.

Glued-Laminated Longitudinal Timber Deck Studies

From the literature review, it was found that typical span lengths range from 9 to 36 ft for both single and multiple spans. The widths of the glued-laminated panels used in the bridges ranged from 44 to 62 in. However, the most common panel widths were between 48 and 52 in. Roadway widths varied from 16 to 36 ft and most commonly had two lanes. Designs for location and size of stiffener beam varied. In most cases, the arrangements and sizes were similar to the recommended values found in the Weyerhaeuser Design Tables (3) for longitudinal deck bridges. Four types (or variations) of stiffener beam connectors are available: aluminum brackets, thru-bolts, steel plates, and C-clips (Figure 2). The most common types of connectors being used are the aluminum brackets and thru-bolts. Four deck thicknesses are typically used: 6 3/4, 8 3/4, 10 3/4, and 12 1/4 in.

In the late 1970s, several studies provided valuable information about longitudinal deck bridge behavior. Evans conducted theoretical studies (unpublished) of single-span, low-profile, glued-laminated decks and stiffened deck panel systems. The studies involved the development of finite element models for use in predicting the behavior of the bridge. The analytical models were validated by comparison with test bridges 24 ft in length having various stiffener beam spacings. The model results compared well with the bridge test results. Two noteworthy conclusions from the study were that "for all cases investigated, a conservative design procedure results when a single standard wheel line is applied to a single panel and that panel is designed as a beam; i.e., the load is assumed to cause symmetric bending" and "present AASHTO code design requirements are overly conservative." In addition to providing information that illustrated the favorable behavior of the bridge with respect to load distribution, the study supported the validity of the finite element approach for modeling this bridge.

Hale performed field tests on both single-span and three-span stiffened longitudinal deck bridges (4-6). The tests determined the behavioral characteristics of various hardware and connectors that are typically used with the bridges. The tests also provided insight into the mechanism of the load transfer that occurs between the adjacent panels through the combined action of the stiffener beams and connectors.

Analytical Studies

Plate theory has been used most often in the modeling of timber bridges. Sanders (7) used orthotropic plate theory in a study of load distribution behavior in bridge stringers. The stringers were composed of steel and timber that supported timber decks. The complete system, including deck and stringers, was assigned appropriate orthotropic properties in the analysis.

McCutcheon (2) also utilized orthotropic plate theory to represent glued-laminated deck panels that

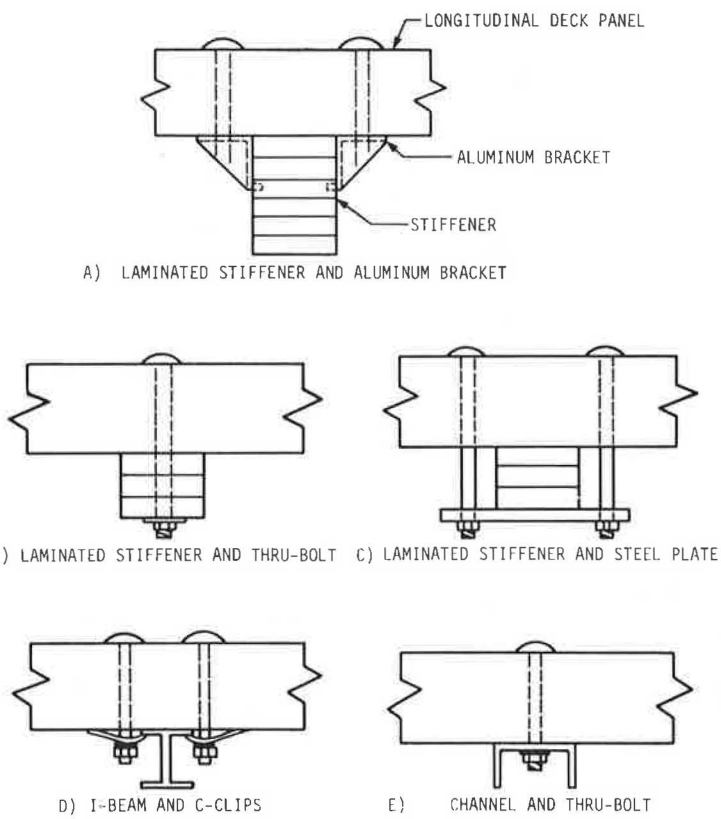


FIGURE 2 Types of transverse stiffener beam connections.

were supported by stringers. The panels were placed transverse to the stringers and connected to each other by dowels. The deck panels were analyzed as infinite strips under uniform rectangular load, where the load represented the wheel of the vehicle.

A third study, by Taylor et al. (8), utilized the same theory for another type of timber deck bridge. The bridge consisted of individual laminates that act together through a system of lateral post-tensioning. Laboratory tests were conducted on specimens that were post-tensioned to different levels of force, and orthotropic properties were determined. It was found that the plate properties were dependent on the level of post-tensioning force.

BEHAVIOR OF GLUED-LAMINATED BRIDGES

Glued-Laminated Bridge Idealization

On the basis of information obtained during Phase Two of this study, an analytical model was developed. This section discusses the model and the investigation of its validity as well as its sensitivity to selected parameters.

Finite element analysis was used in the development of the model for this study; plate theory was used to idealize the glued-laminated deck. A structural analysis program, SAP IV (9), commonly used for analysis of simple and complex structures, was used. For the model, plate and beam elements were used to simulate the various bridge members. Figure 3 shows a sketch of a model used to simulate a four-panel bridge with a single midspan stiffener. The deck panels were modeled as thin-plate elements having both longitudinal and transverse flexural rigidity as well as torsional rigidity. These plates are connected laterally to each other through a system of vertical elements that have axial stiffness only and simulate the deck connectors. The horizontal flexural elements represent the stiffener beams; the panels are connected only at the stiffener beams through the vertical elements. In this manner, lateral shear is transferred to adjacent panels through the interaction of the panel connectors and stiffener beams.

The longitudinal timber deck was analyzed assuming orthotropic plate behavior. The appropriate material matrix coefficients that define the timber panels were based on a report by Bodig et al. (10).

The remaining major components of the glued-laminated longitudinal deck highway bridge are the connectors and stiffener beams. Obviously, in the case of a connector in compression, load would be transferred primarily through the action of bearing between the panel and the stiffener beam. Previous analytical refinement of this interaction by Evans (unpublished data) produced no significant improvement over the model used in this study. Therefore, no attempt was made to refine the connector interaction model previously described.

Validation of Computer Model

The computer model's sensitivity to different magnitudes of the connector stiffness and type of stiffener beam was investigated and results were compared to full-scale test data obtained by Weyerhaeuser on 4-panel bridges made up of 24-ft simple spans (6). The panels were made of Douglas fir.

Two connector types (or variations of) are typically used for the type of bridge in question. In a study by Hale (4), five connector types shown in Figure 2 were tested in the laboratory to determine their structural characteristics. From the test

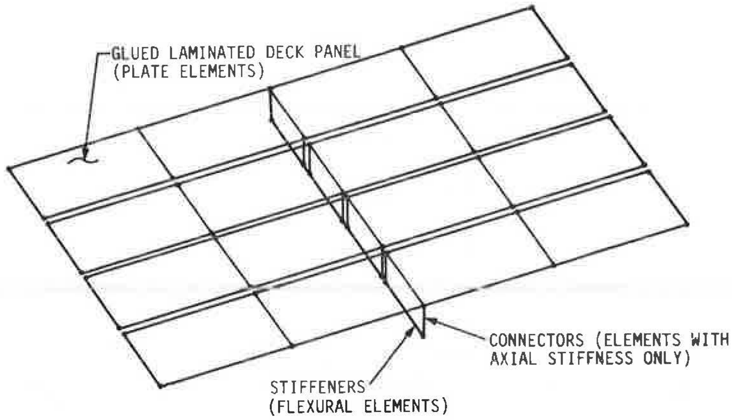


FIGURE 3 Finite element computer model of longitudinal glued-laminated bridge.

data, connector stiffness (K) was determined and subsequently used in the bridge model. The load-deflection data from the study indicated a linear relationship for the thru-bolt connection; its calculated stiffness was 667,000 lb per in. The aluminum bracket connection behaved in a nonlinear manner. In the interest of simplifying the behavior for use in modeling, Evans, in an earlier analytical study (unpublished), estimated the stiffness as 130,000 lb per in. The connector stiffness was varied within a wide range of values, and the model results were compared to data from selected full-scale tests. A typical result of this investigation is shown in Table 1, which shows deflections for a four-panel bridge for different connector stiffness. As may be seen, there is a significant behavioral difference in the bridge for the smaller stiffnesses considered. As noted, the smaller connector stiffness allows greater relative panel movement. In studying the deflection of the panels, it appears reasonable that the smaller stiffnesses also create less load sharing between adjacent panels. In the analytical study performed to determine load distribution behavior, the stiffness corresponding to the aluminum bracket connection was used, thus assuring that the developed design criteria would be conservative.

TABLE 1 Model Sensitivity Study for Variation of Connector Stiffness (K)—Weyerhaeuser Test No. 20[a]

Connector Stiffness (kips/in.)	Midspan Deflections (in.)							
	Panel 1		Panel 2		Panel 3		Panel 4	
	A	B	C	D	E	F	G	H
Experimental value[b]	.56	.45	.41	.37	.27	.16	.12	.04
1.3	.62	.54	.45	.64	.05	.04	.02	0
50	.56	.44	.40	.38	.26	.15	.13	.04
100	.54	.43	.40	.35	.28	.16	.14	.05
130	.54	.42	.40	.35	.29	.16	.15	.05
1000	.53	.41	.40	.32	.31	.17	.16	.06

[a]See Figure 5 for description of test.
[b]This value is based on the actual test; other values are from theoretical study.

The effect of the size (i.e., measure of stiffness) of the stiffener beam on model results was investigated. In the field tests performed by Hale (6), stiffener beams of various sizes and materials were tested including glued-laminated and steel members. However, the literature survey conducted earlier indicated only the use of rectangular glued-laminated stiffener beams, in which the sizes of the beams varied within a small range (in most cases they are approximately 5 to 7 in. deep).

The stiffener moment of inertia (I) was varied to determine the model sensitivity to this variable. Results were compared with data from the Weyerhaeuser full-scale bridge tests. Table 2 shows a typical result of the comparisons and indicates that the relatively large I's have a relatively small effect on the bridge behavior and that the range of I's selected did not have nearly as pronounced an effect on behavior as did the connector stiffnesses that were selected. These results were consistent with those found in an earlier study by Evans (unpublished data). In the analytical study, the use of a rectangular, glued-laminated, 5 x 7 in. stiffener beam was assumed.

TABLE 2 Model Sensitivity Study for Variation of Stiffener Beam Inertia (I)—Weyerhaeuser Test No. 20[a]

Stiffener Beam Inertia ($in.^4$)	Midspan Deflections (in.)							
	Panel 1		Panel 2		Panel 3		Panel 4	
	A	B	C	D	E	F	G	H
Experimental value[b]	.56	.45	.41	.37	.27	.16	.12	.04
20	.53	.40	.39	.35	.34	.14	.14	.07
1,000	.53	.41	.41	.31	.30	.17	.17	.06
10,000	.53	.41	.41	.31	.29	.18	.17	.06

[a]See Figure 5 for description of test.
[b]This value is based on actual test; other values are from theoretical study.

After developing the finite element model and studying the sensitivity of various bridge parameters on its performance, a validation of the model using full-scale test data was performed. Although the sensitivity study discussed in the previous section also served to validate the model through comparisons made with full-scale test data, this section involved validations made using only the parameter values that were selected for use in the analytical study for load distribution. A typical comparison of results is shown in Figure 4. The results were equally good in all comparisons.

Although analytical model validation was made with bridges constructed of Douglas fir, the distri-

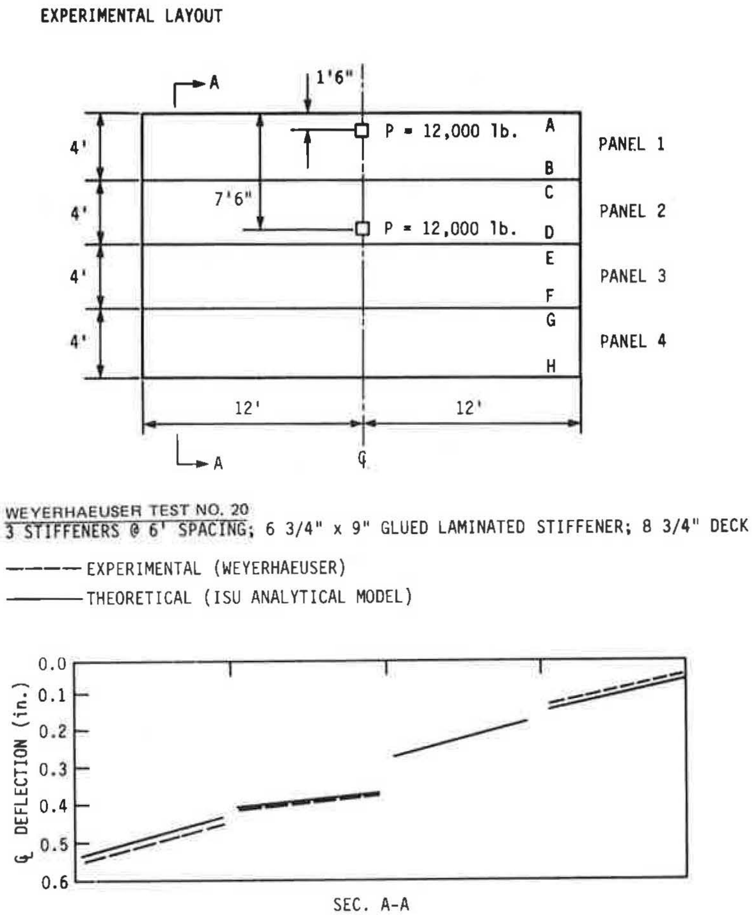


FIGURE 4 Comparison of theoretical data with experimental data.

bution criteria should be equally applicable to bridges constructed of southern pine. On the basis of results of the sensitivity studies, it is anticipated that load distribution behavior will be slightly more favorable for the bridge constructed of southern pine. Correspondingly, the panel deflections would be slightly greater.

ANALYTICAL STUDY

Deflection of Parameters

Once the finite element model was validated, Phase Three was initiated. This phase involved the use of an analytical study to quantify the load distribution behavior of the longitudinal deck bridges. On the basis of previous survey information of typical bridge sizes, a parameter program was set up that covered the range of values for the critical variables that describe the bridge behavior. Initially, a much fuller complement of simulations considering additional span lengths was considered for this study. However, after studying the results from the simulations of the lower and upper ranges of span length, and noting the similarity to the results obtained by Evans (unpublished data), it was decided that considerably fewer simulations were required to determine the bridge behavior.

The parametric program setup consisted of bridges with span lengths from 9 to 33 ft, roadway widths from 16 to 40 ft, deck thicknesses from 6 3/4 to 12 1/4 in., and various stiffener arrangements (see Table 3). These span lengths were initially applied to the study of single spans only; however, multiple spans were later considered. The loadings were placed for maximum effects for both the concentric and eccentric loading conditions. One-, two-, and three-lane roadways that were investigated are illustrated in Figure 5. The loadings were based on AASHTO standard specifications for critical positioning of traffic lanes.

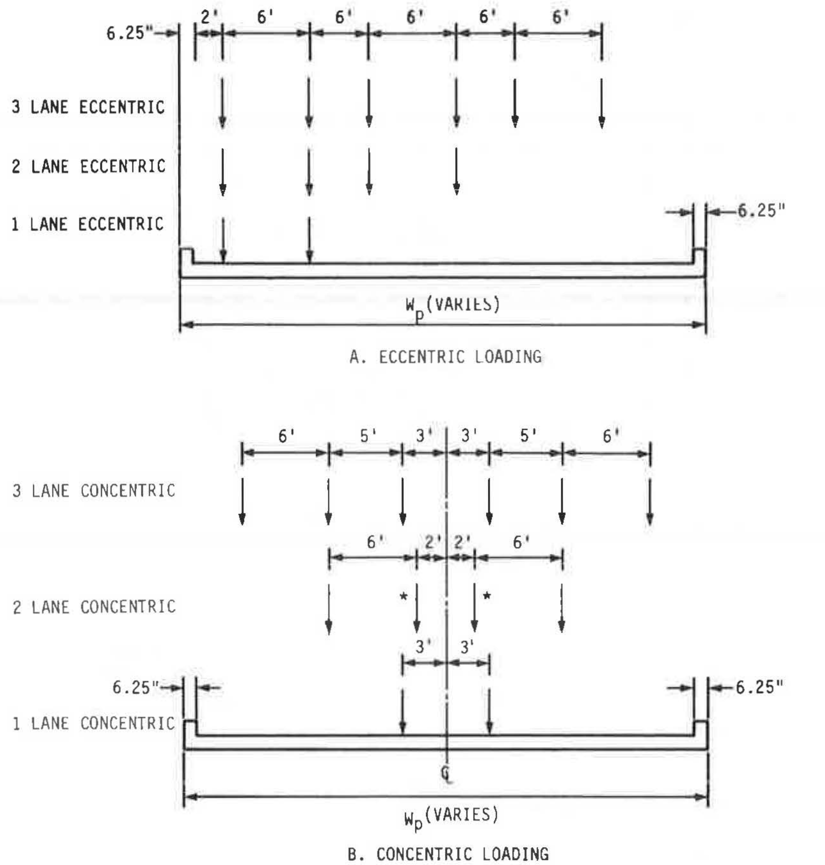
TABLE 3 Parameter Program for Single-Span Bridges

Bridge Span Length (ft)	Number of Stiffener Beams	Panel Thickness (in.)	Number of 48-in. Wide Panels[a]
9	1	6 3/4	4,7,8,9,10
15	2	8 3/4	4,7,8,9,10
21	3, 2	8 3/4, 10 3/4	4,7,8,9,10
27	3, 2	10 3/4, 12 1/4	4,7,8,9,10
33	5	12 1/4	4,7,8,9,10

[a]Bridges with 4 panels simulate single-lane roadways. Bridges with 7,8,9 panels simulate two-lane roadways. Bridges with 10 panels simulate three-lane roadways.

Results of Analytical Study

The current AASHTO format for load distribution, which uses a distribution factor (D) to represent load distribution behavior, was selected for this study because of its simplicity for use by the designer as well as for its convenience in representing distribution characteristics. A wheel-load fraction (W_p/D) was computed for each load case and



*When this load was located at a panel joint, the load was assumed to be carried equally by adjacent panels.

FIGURE 5 Loading cases considered.

bridge configuration where the term W_p represents the panel width. The distribution factor (D) is determined by comparison of the panel moment in the most heavily loaded panel (maximum moment) for a given loading condition with the average moment in the panels.

Thus,

D = bridge width/(M)(number of wheels for loading considered)

where M is the maximum panel moment/average moment in panels. A heavy concentration of moment in a panel would yield a large value for M and a corresponding small value for D. Therefore, a small D value represents poorer load distribution behavior.

The results of the analytical study with regard to single-span distribution of moment are shown in Table 4. As noted in the table, for single-lane bridges (4 panels), the critical loading case was obtained when eccentric loading was acting, whereas concentric loading gave critical values for D for multiple-lane bridges (7, 8, 9, and 10 panels). It is further noted that the width of the roadway had little effect on the load distribution behavior for multiple-lane roadways. The tendency to transfer lateral load was dissipated within the same distance regardless of the number of panels making up the bridge.

A limited number of two equal-span configurations were also investigated. The comprehensive study conducted for single-span distribution behavior provided adequate information to enable this limited study to predict multiple-span behavior. The lateral load distribution tendencies were similar to those for the single spans as shown in Table 5, with the multiple-span systems showing slightly worse behavior in most cases. The stiffer system caused by the continuity is the apparent cause of this behavior.

TABLE 4 Distribution Factors for Single-Span Moment (values shown are D in the Wheel Fraction W_p/D)

Number of Panels	Load Cases	Span Length in ft. (No. of Stiffeners) 9 (1)	15 (2)	21 (3)	21 (2)	27 (3)	27 (2)	33 (5)
4	1C[a]	5.21	6.07	7.04	6.36	–[c]	–	6.93
	1E[b]	4.84	5.18	5.59	5.38	5.69	5.58	6.00
7	1C	5.83	7.22	8.60	7.78	–	–	9.90
	2C	4.21	4.56	5.12	4.82	5.20	5.07	5.55
	1E	4.84	5.24	–	5.40	5.70	5.62	6.14
	2E	4.91	5.28	5.53	5.35	–	5.41	5.58
8	1C	5.33	6.40	–	6.92	–	–	–
	2C	4.64	4.92	5.45	5.07	5.30	5.27	5.91
	1E	4.85	5.23	5.69	5.40	5.59	5.62	6.15
	2E	4.91	5.28	5.52	5.35	–	5.40	5.57
9	1C	5.79	7.25	–	7.81	–	–	–
	2C	4.21	4.56	5.17	4.83	5.10	5.12	5.76
	1E	4.84	5.22	5.67	5.41	5.82	–	6.16
	2E	4.91	5.29	5.52	5.35	–	5.41	5.57
10	1C	5.33	6.41	–	6.92	–	–	–
	2C	4.65	4.92	5.48	5.08	5.22	5.29	6.02
	3C	4.56	4.95	–	5.10	–	–	–
	1E	4.84	5.23	–	5.41	5.65	5.62	6.20
	2E	4.91	5.28	5.53	5.36	5.52	5.41	–
	3E	4.91	5.29	5.36	5.36	–	5.36	5.60

[a]C = concentric load case.
[b]E = eccentric load case.
[c]Load case not reviewed.

The effect of distribution on shear was also con-

TABLE 5 Summary of Critical Distribution Factors for Moment and Shear

Span Type (Internal Force)	Span Length (ft) 9	15	21	27	33
Single Span (Moment)	4.24	4.56	4.82	5.07	5.55
Multiple Span (Moment)	4.19	4.50	4.90	5.13	5.30
Single Span (Shear)	4.24	4.68	5.08	5.42	6.13
Multiple Span (Shear)	4.13	4.27	4.54	4.90	5.05

sidered by calculating the reaction at the supports. The load cases considered were the same as those used for the lateral load moment distribution study. Values of D for shear were determined in a manner similar to those for moment (i.e., by comparing the maximum shear in the most heavily loaded panel for a given loading condition with the average shear in the panels). As noted for the longer-span lengths, the behavior was considerably better for shear than for moment when considering the single-span bridges. However, shear in the multiple span cases exhibited different behavior than this as shown in Table 5. Because these distribution factors were similar to those found for moment, and in the interest of simplifying the shear criteria, a decision was made to apply the moment distribution factors in the development of all shear criteria.

Figures 6 and 7 contain plots of the critical distribution values (D) versus the span length (L). As noted in the figures, the distribution behavior improves as span length increases. This behavior may be explained by considering the mechanics of load transfer performed by the various components of the longitudinal bridges. As the span length increases, the panel flexural stiffness decreases. This decreased stiffness causes a corresponding increase in the panel deflections, which causes increased interaction between the stiffener beams and adjacent panels. The cause of poorer distribution for the multiple span may also be attributed to the same factors, in that the multiple-span system is stiffer than the singlespan system of corresponding length. Although the study by Evans (unpublished data) attempted to quantify the bridge behavior in a different manner than that presented here, the basic results were similar to those found in this study. In Evans' study, the same trend toward improved distribution behavior with corresponding increase in span length was found.

DEVELOPMENT OF LOAD DISTRIBUTION CRITERIA

The proposed distribution criteria submitted to AASHTO, which appear in an appendix to this paper, were based on the results of the analytical study

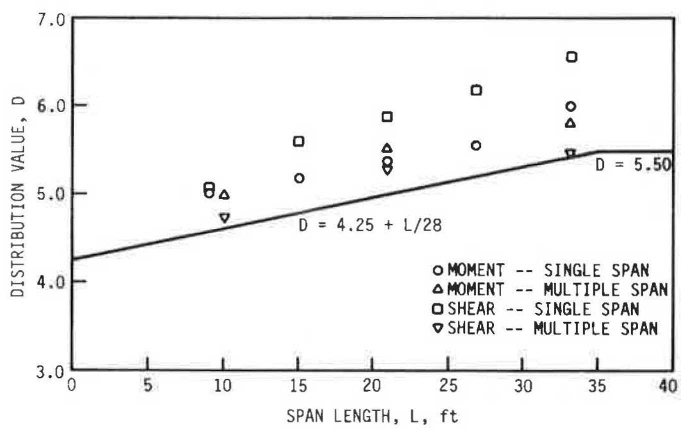


FIGURE 6 Plot of critical D values for all cases considered in parameter study for single-lane bridges.

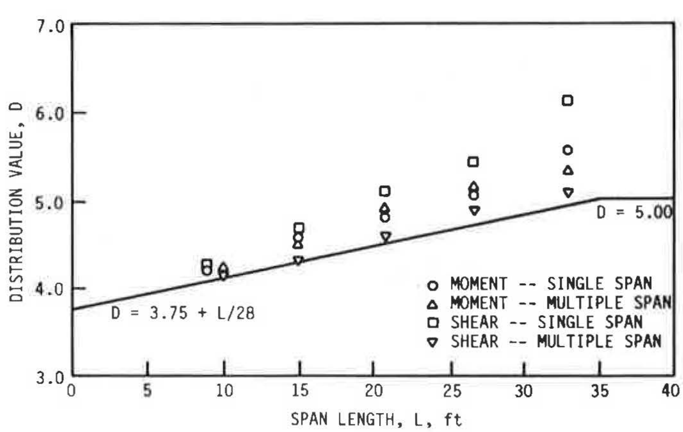


FIGURE 7 Plot of critical D values for all cases considered in parameter study for multiple-lane bridges.

just summarized. In this section, the previously mentioned criteria are developed and the effect of various parameters on their prediction performance are discussed. A design example may be found in a report by Sanders et al. (11).

Proposed Design Equation for Panels

As previously mentioned, the present AASHTO format for load distribution that incorporates a wheel fraction, W_p/D, to indicate wheel load percentage acting on a load carrying member, was also adopted for this study. As noted in the design equation in the Appendix, the distribution factor (D) is represented by a linear equation that includes the span length of the bridge. The equation was obtained from a plot of all of the critical D's for both shear and moment as a function of span length (see Figures 6 and 7). A somewhat arbitrary line (in that no set confidence limit was used considering all data points) was then drawn to represent a conservative relationship for D. This line represented adequate conservatism for design in lieu of the conservatism of the model and because the values plotted in Figures 6 and 7 were the critical values. The numerator term in the design equation (W_p) is the actual panel width and may vary. A discussion of the effect of varying panel widths is found in a later section of this paper.

Once the wheel fraction is determined, the panel is designed for the appropriate partial wheel load based on symmetrical bending. The full panel dimensions are used in determining the section properties.

Additional Design Considerations

In the proposed criteria (see Appendix), two additional design considerations are presented: deflection prediction and stiffener beam arrangement. The analytical study revealed that the maximum deflection in the bridge can be predicted by applying to a panel the moment wheel load fraction. The desired deflection is then calculated using conventional elastic analysis, assuming a symmetrically loaded beam. The deflections determined by this method agreed closely with the analytical results; comparisons of these deflections are shown in Tables 6 and 7. It should be noted that the values shown for the longer spans do not necessarily represent the absolute maximum deflection in the bridge. The deflections shown are due to the loads used only in the distribution study, which, for single spans, represent loads placed at midspan.

An empirical approach is used in the stiffener beam design. From the distribution study based on the beam spacings shown in Table 3, an investigation was made into the effects of varying the spacing. From these results, it was concluded that the addition of beams over those shown in Table 3 had an insignificant effect on the distribution. In many cases, one stiffener beam at midspan gave results as favorable for midspan lateral distribution as for placing beams at the one-quarter points. It was definitely advantageous, from a lateral distribution standpoint, to have at least a midspan stiffener rather than stiffener beams at one-third points. On the basis of this information, the criteria specify that a beam shall be placed at midspan, with any additional beams spaced at intervals not to exceed 10 ft. Consideration of the relative panel deflection is left to the discretion of the design engineer.

It is also recommended that the stiffness factor, EI, of the stiffener beam shall be no less than 80,000 kips per in.^{2}. This value is representative of the stiffness of the most commonly used stiffener beams found in the literature survey. Because the analytical study showed that the size of the stiffener beam had insignificant effects on distribution of beams within a practical size range, however, this criterion is appropriate.

TABLE 6 Comparison of Theoretical Deflections with Calculated Deflections—Single Span

Span Length (ft)	Panel Thickness (in.)	Wheel Load Fraction	Theoretical Deflection (in.)	Deflection Based on Proposed Criteria[a] (in.)
9	6.75	0.982	0.16	0.16
15	8.75	0.933	0.31	0.33
21	10.75	0.889	0.43	0.47
27	12.25	0.848	0.59	0.64
33	12.25	0.812	0.97	1.11

[a]Deflection based on $\Delta = PL^3/48EI$
where E = 2050 ksi, P = 16^k, and Panel Width (W_p) = 4 ft.

TABLE 7 Comparison of Theoretical Deflections with Calculated Deflections—Multiple Span

Span Length (ft)	Panel Thickness (in.)	Wheel Load Fraction	Theoretical Deflection (in.)	Deflection Based on Proposed Criteria[a] (in.)
10	6.75	0.974	0.16	0.16
15	8.75	0.933	0.23	0.24
21	8.75	0.899	0.57	0.62
27	10.75	0.848	0.62	0.68
33	12.25	0.812	0.74	0.80

[a]Deflection based on $\Delta = 3\ PL^3/200EI$
where E = 2050 ksi, P = 16^k, and Panel Width (W_p) = 4 ft.

Effect of Panel Width and Stiffener Beam Spacing

After the design criteria were developed, an investigation was made of the effect that a practical range of parameter values might have on the design criteria prediction. Two parameters were considered: panel width and stiffener beam spacing. Initially, the panel width was thought to be a key factor in load distribution behavior because, as found in the model study, a major reason for differences in distribution was the change in relative position of wheel loads with respect to the panel edge. In other words, the lateral load position on a panel affects the interaction that the loaded panel has with the stiffener beams, which consequently affects the distribution of the load to adjacent panels. Subsequent sensitivity studies considering varying panel widths within practical ranges, however, showed only slight changes in distribution. The range of panel widths considered for the analytical study was 42 to 54 in. It was found that the distribution factor (D) was slightly larger for 42-in. panel bridges than for 48-in. panel bridges. Also, the D's for the 54-in. panel bridges were slightly smaller than for the 48-in. panel bridges. As a result of this study, the proposed criteria include the consideration of a panel width within the ranges mentioned above as part of the wheel fraction expression.

As may be seen in the proposed design equation for lateral load distribution, the panel width (W_p) is a variable. The panel is designed for symmetrical bending using the full panel width properties. The wider the panel, the larger the wheel fraction, W_p/D. There is compensation, however, for the design of the wide panel relative to the narrow panel. This is because of the wider panel's larger section modulus (S) and corresponding smaller stress. The result is that even though the two preceding cases result in different wheel fractions, similar flexural stresses are derived.

The stiffener beam arrangement and its effect on distribution behavior was also investigated. The stiffener beam spacing had a definite effect on relative panel displacements. For this reason, before deciding on beam spacing, the designer would certainly want to determine how much differential movement could be tolerated without damage to the deck surfacing. As far as distribution behavior is concerned, a stiffener beam placed at midspan of a single span results in the most favorable behavior. The primary effect of additional stiffener beams is that of limiting the differential movement between panels and, for this reason, the proposed criteria address stiffener-spacing limitations. The addition of stiffener beams beyond limits recommended in the criteria would not significantly improve distribution.

SUMMARY

There has been renewed interest in the use of timber bridges for secondary roadways. A recent development in the timber industry has been the glued-laminated, longitudinal, deck highway bridge. Present design criteria for this bridge are not clearly defined, nor do they truly represent the distribution behavior.

Full-scale tests were performed in the 1970s on the longitudinal deck bridge to study its behavior. Subsequent analytical studies were performed using this test data to study the bridge further in order to provide insight into a possible design technique.

The study presented here was performed in three phases, the ultimate goal of which was the development of load distribution design criteria that would be applicable to the longitudinal deck bridge. The design criteria that have been developed have been submitted to the AASHTO Bridge Subcommittee and approved for inclusion in the Bridge Design Specifications.

The first two phases of the study involved a review of literature and a survey of bridge parameters related to the longitudinal bridge. Parameters were defined from the results of the two phases, and in Phase 3, an analytical model was developed to study the load distribution behavior. The parameters identified in the first two phases of the study were used in the model. The sensitivity of various parameters (connector stiffness, stiffener beam size, etc.) on the bridge behavior was determined by using the analytical model. The analytical model was validated by comparison of its results to full-scale test data obtained by others.

The analytical study considered single- and multiple-lane bridges loaded with standard AASHTO loading. The placement of load was based on critical positioning of traffic lanes defined by AASHTO. The model results indicate that the load distribution behavior is primarily dependent on span length. As the span length increases, the load distribution behavior improves. The criteria contain a design equation that allows a determination to be made of a wheel load fraction that acts on the panel. The panel is then designed for this load on the basis of symmetrical bending.

CONCLUSIONS

Based on the results of this study, the following conclusions can be made:

1. The research contained herein supports the improved distribution criteria over the existing criteria.
2. The size of the stiffener beam within practical size ranges did not significantly affect distribution behavior.
3. The type of connectors typically used have an effect on relative panel displacements, and the thru-bolt connector is recommended.
4. Load distribution behavior at the midspan of single-span bridges is best when at least one stiffener beam is placed at midspan.
5. The lateral load distribution behavior of the bridge improves as span length increases.

ACKNOWLEDGMENTS

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APPENDIX--Proposed Distribution Criteria

The following proposed specifications have been submitted to and approved by the AASHTO Operating Subcommittee on Bridges and Structures:

Change Section 3.25 on "DISTRIBUTION ON WHEEL LOADS ON TIMBER FLOORING" in AASHTO Standard Specifications for Highway Bridges as follows:

1. Add new Section 3.25.3 shown below;
2. Delete reference to glued laminated panels in 3.25.2.2;
3. Change title of 3.25.2 to read "PLANK AND NAIL LAMINATED LONGITUDINAL FLOORING;"
4. Change 3.25.3 to 3.25.4.

3.25.3 LONGITUDINAL GLUED LAMINATED TIMBER DECKS

3.25.3.1 BENDING MOMENT

In calculating bending moments in glued-laminated timber longitudinal decks, no longitudinal distribution of wheel loads shall be assumed. The lateral distribution shall be determined as follows.

The live load bending moment for each panel shall be determined by applying to the panel the fraction of a wheel load determined from the following equations:

TWO OR MORE TRAFFIC LANES

Load Fraction = $W_p/[3.75 + (L/28)]$ or $W_p/5.00$, whichever is greater.

ONE TRAFFIC LANE

Load Fraction = $W_p/[4.25 + (L/28)]$ or $W_p/5.50$, whichever is greater.

where W_p = Width of Panel in ft ($3.5 \leq W_p \leq 4.5$) and L = Length of span for simple-span bridges and the length of the shortest span for continuous bridges in ft.

3.25.3.2 SHEAR

When calculating the end shears and end reactions for each panel, no longitudinal distribution of the wheel loads shall be assumed. The lateral distribution of the wheel load at the supports shall be that determined by the equation:

Wheel Load Fraction per Panel = $W_p/4.00$ but not less than 1.

For wheel loads in other positions on the span, the lateral distribution for shear shall be determined by the method prescribed for moment.

3.25.3.3 DEFLECTIONS

The maximum deflection may be calculated by applying to the panel the wheel load fraction determined by the method prescribed for moment.

3.25.3.4 STIFFENER ARRANGEMENT

The transverse stiffeners shall be adequately attached to each panel, at points near the panel edges, with either steel plates, thru-bolts, C-clips, or aluminum brackets. The stiffener spacing required will depend upon the spacing needed in order to prevent differential panel movement; however, a stiffener shall be placed at midspan with additional stiffeners placed at intervals not to exceed 10 ft. The stiffness factor EI of the stiffeners shall not be less than 80,000 kips per in.[2]

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Probabilistic Assessment of the Failure of Laminated Timber Bridges

LESLIE G. JAEGER and BAIDAR BAKHT

ABSTRACT

For a given loading, the ratio of the expected failure load of a laminated timber bridge and the analytical failure load obtained by assuming the modulus of rupture (MOR) and modulus of elasticity (E_L) to obtain their respective mean values, is called the reduction factor. The value of this factor is always less than 1.0, and depends on the transverse deflection profile of the bridge. A procedure is presented in this paper by which a realistic assessment of the reduction factor can be obtained by using test data for the MOR and E_L for any species of wood.

There are two properties of timber beams that have a marked effect on the distribution of live loads in laminated timber bridges and on the ultimate load-carrying capacity of such bridges; these are the modulus of elasticity (E_L) and the modulus of rupture (MOR).

The E_L and MOR vary widely from one timber beam to another. Figure 1 shows test values of E_L and MOR on 70 Red Pine specimens. These values were originally given in an unpublished report of the Ontario Ministry of Transportation and Communications, and were used recently in Bakht and Jaeger (1). As will be seen from Figure 1, E_L and MOR are strongly correlated, but not perfectly so. If they were perfectly correlated, all points in Figure 1 would appear on a single line. The variability in E_L and MOR, and, more particularly, the variability in E_L/MOR, has the effect of significantly reducing the ultimate load-carrying capacity of the bridge, as compared to what it would be if all beams had the same E_L and MOR, equal to their mean values for the species concerned. This significant reduction can be related to the amount of scatter that is shown in Figure 1. The reduction factor is also dependent on the deflection profile of a cross-section of the bridge. In this paper, only the estimation of the expected load capacity is dealt with; estimation of the variance of this capacity is dealt with elsewhere.

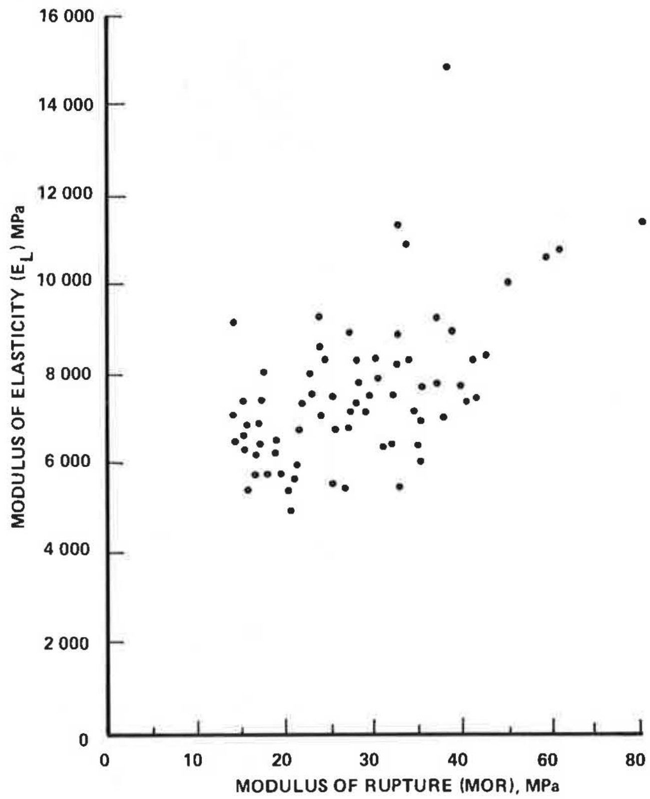

FIGURE 1 Test data on 70 Red Pine specimens.

DERIVATION OF THE METHOD OF ANALYSIS

As a first step, the values of E_L and MOR given in Figure 1 are replotted on axes of $E_L/(E_L)_{mean}$ and $MOR/(MOR)_{mean}$ as shown in Figure 2. Then, for a given species of timber, the scatter of points can be approximated as lying within an area bounded by $r = r_1$ and $r = r_2$ and by $\theta = \alpha$ and $\theta = (90°-\alpha)$, where r is a radial distance from the origin and θ is an angle measured clockwise from the $E_L/(E_L)_{mean}$ axis, as shown in Figure 2. For Red Pine, the angle α is about 30°. It will be shown later that the limiting values r_1 and r_2 do not need to be estimated.

The behavior depicted in Figure 2 may be contrasted with that of Figure 3, which shows what happens if the values of E_L and MOR are assumed to be perfectly correlated (i.e., if E_L/MOR is assumed to be constant). It will be seen that the imperfect nature of the correlation that occurs with Red Pine results in a "fan" of points rather than a single line.

The scatter of points over the area of Figure 2 is represented by a probability density function $p(r,\theta)$, which satisfies the relationship

$$\int_{\alpha}^{(\pi/2)-\alpha} \int_{r_1}^{r_2} p\,(r,\,\theta)\; r\,dr\,d\theta = 1 \qquad (1)$$

For simplicity, two further assumptions are made about the nature of the points in Figure 2. These

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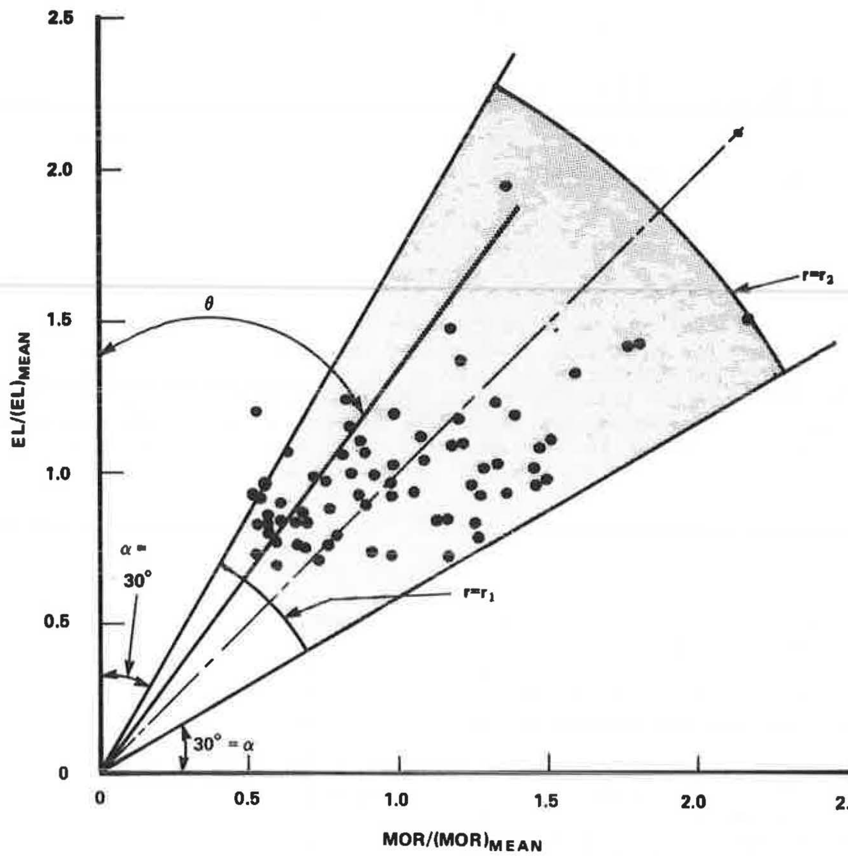

FIGURE 2 Data on Red Pine wood.

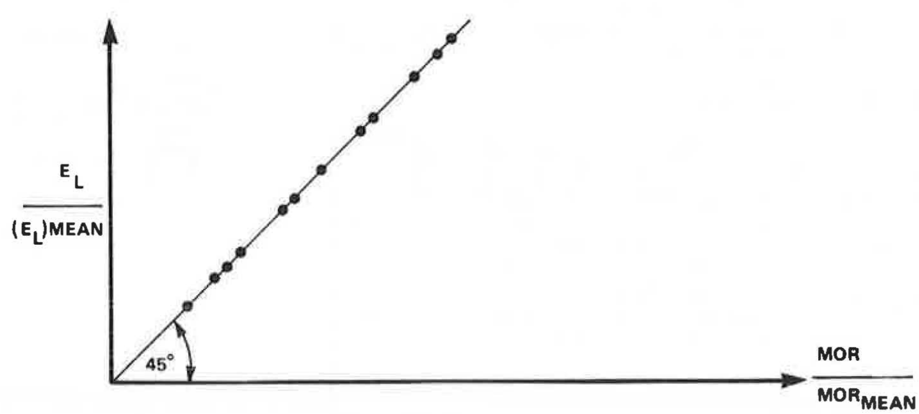

FIGURE 3 Perfect correlation of E_L and MOR.

are that the probability distribution can be split up into "variation with θ" and "variation with r" and that the "variation with θ" is symmetrical about the line θ = 45°. Both of these assumptions are reasonable in light of the observed test values for different species of timber.

It is fortunate for the purposes of estimation of failure load that the analysis is not sensitive to the precise form of the probability function $p(r,\theta)$ that is chosen. (This point will be returned to later.) The general nature of the probability distribution $p(r,\theta)$ is shown in Figure 4. Then,

$$p(r,\theta) = f(r)\phi(\theta) \tag{2}$$

where

$$\int_{r_1}^{r_2} f(r)r\,dr = 1 \tag{3}$$

and

$$\int_{\alpha}^{(\pi/2)-\alpha} \phi(\theta)\,d\theta = 1 \tag{4}$$

Figure 5 shows the general nature of the probability functions $\phi\theta$. This function must be zero outside the range of values α and (90°−α) and be symmetrical about θ = 45°. A suitable form for the purpose of analysis is

$$\phi(\theta) = c(\sin 2\theta - \sin 2\alpha) \tag{5}$$

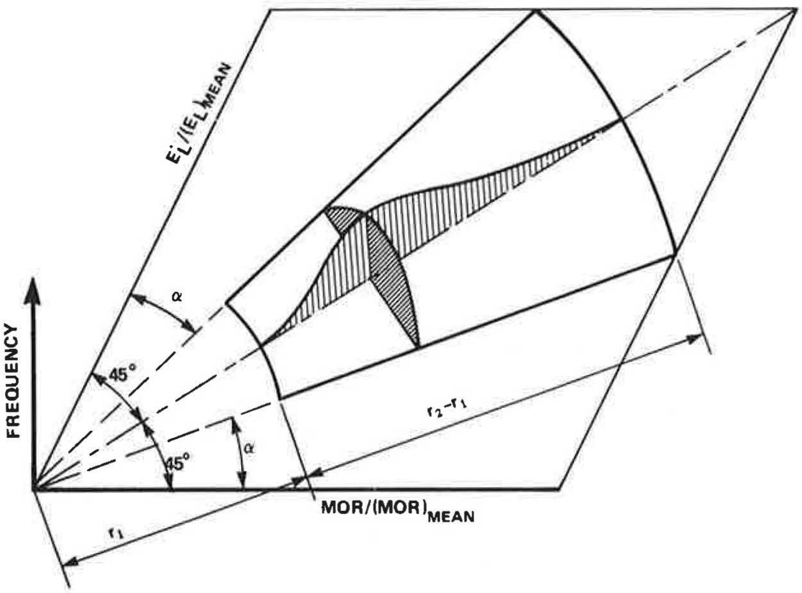

FIGURE 4 The probability function p(r,θ).

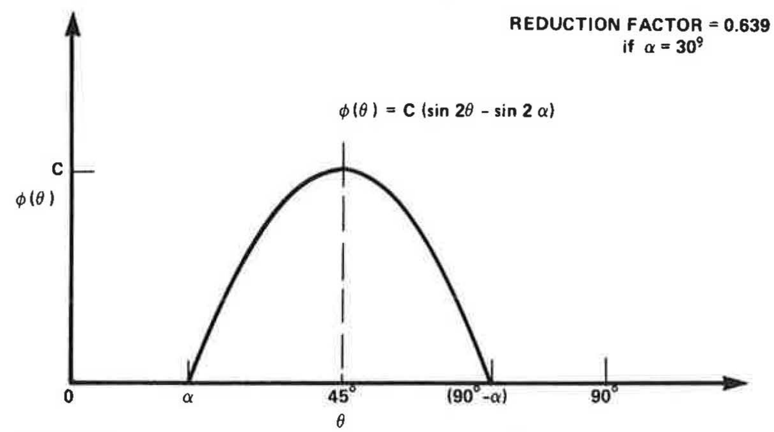

FIGURE 5 Plot of φ(θ) against θ.

for

$\alpha \leq \theta \leq 90° - \alpha$

The constant c in Equation 5 is readily found by using Equation 4. In the case of Red Pine, taking α = 30° the following is obtained:

$$\phi(\theta) = 21.488[\sin 2\theta - (3^{1/2}/2)] \quad (5a)$$

Having derived the necessary properties of the scatter of pairs of values of E_L and MOR, an investigation of ultimate failure loads for various patterns of transverse deflection can now be undertaken. For each pattern of deflection, the actual expected value of ultimate failure load is related to the deterministic failure load that is obtained by taking all beams to have the same E_L and MOR, being the mean value for the timber species concerned. As a preliminary, it may be noted that the ultimate failure load, M_{ult}, of a beam is directly proportional to its MOR. Hence, in Figure 2, the horizontal axis can be $M_{ult}/(M_{ult})_{mean}$ as well as $MOR/(MOR)_{mean}$.

THE CASE OF UNIFORM DEFLECTION

The first pattern of transverse deflection that is examined is that of uniform deflection, shown in Figure 6. If all beams are taken to be identical, then as the uniform deflection increases, the bending moments accepted by the beams increase until all beams simultaneously reach $(M_{ult})_{mean}$. Hence, in the deterministic approach, the total ultimate bending moment capacity of the bridge is simply

$$M_T = N(M_{ult})_{mean} \quad (6)$$

When the scatter of values of E_L and MOR is taken into account, a different behavior emerges. The beams do not fail simultaneously. Because all beams deflect equally, the moment accepted by a beam is proportional to its own E_L, and its ability to withstand moment is proportional to its own value of MOR (or, alternatively, its own value of M_{ult}). The first beams to fail, therefore, are those with the lowest value of MOR/E_L. Figure 7 is a repetition of Figure 2 with the addition of a failure boundary at $\theta = \beta$. As the uniform deflection shown in Figure 6 increases, a

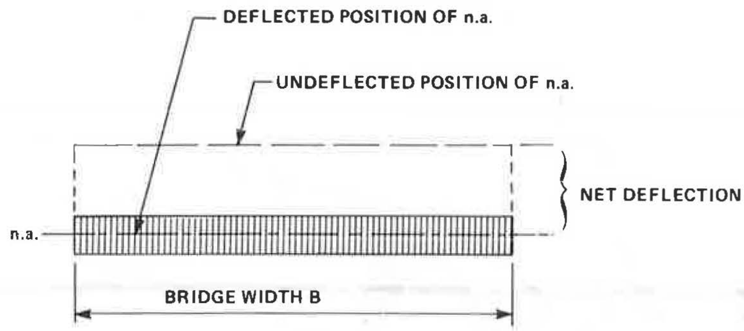

FIGURE 6 Uniform deflection across a bridge cross-section.

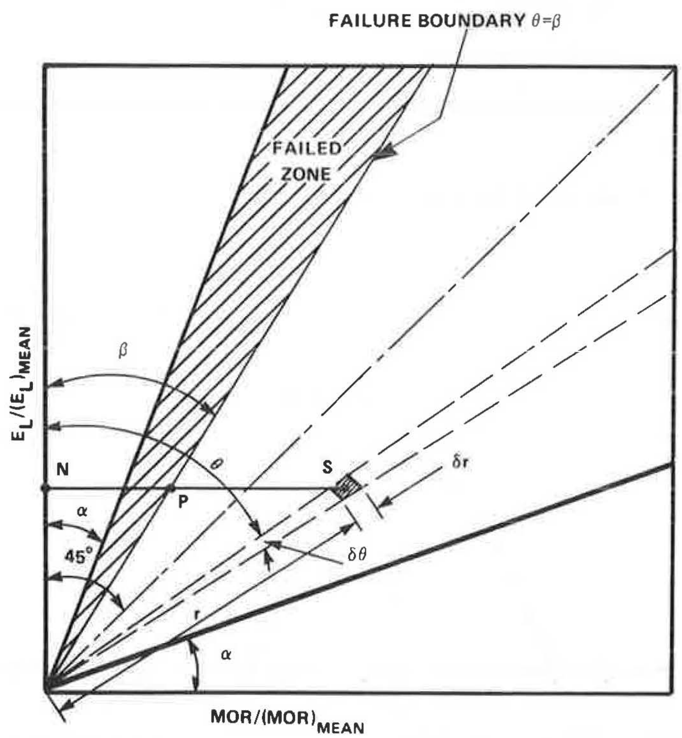

FIGURE 7 Failed zone.

radial line (starting at the vertical axis) rotates clockwise in Figure 7. For a certain deflection, all of those beams to the left of the boundary in Figure 7 have already failed, although those to the right have not. Similarly, in Csagoly and Taylor (2), it is assumed that when a beam's ultimate moment capacity has been reached, the beam fails and its moment carrying capacity falls to zero.

The total bending moment being accepted by those beams that have not yet failed is readily found. The small element $r\delta r\delta\theta$ shown at the point S in Figure 7 is considered. The number of beams expected in this area is $NP(r,\theta)r\delta r\delta\theta$. These beams are not yet at their failure moments, however; they are, in fact, taking a fraction (NP/NS) of their failure moments, with the notation of Figure 7. This follows because all beams having the same value of E_L take the same moments as one another until failure. Using the notation shown in Figure 7,

$$NP = r\cos\theta\tan\beta \tag{7}$$

The bending moment accepted by the beams in the shaded area $r\delta r\delta\theta$ is

$$\delta M = \{Np(r,\theta)r\delta r\delta\theta\}r\cos\theta\tan\beta(M_{ult})_{mean} \tag{8}$$

Using Equations 2-4, and 6, and integrating Equation 8

$$M = M_T\tan\beta \int_{r_1}^{r_2} f(r)r^2dr \int_{\beta}^{(\pi/2)-\alpha} \cos\theta\phi(\theta)d\theta \tag{9a}$$

which is conveniently written as

$$M = k(\beta)M_T \tag{9b}$$

where

$$k(\beta) = \tan\beta \int_{r_1}^{r_2} f(r)r^2dr \int_{\beta}^{(\pi/2)-\alpha} \cos\theta\phi(\theta)d\theta \tag{9c}$$

The calculation of $k(\beta)$ using Equation 9c is straightforward and is greatly simplified by the fact that the function $f(r)$ does not need to be found explicitly. In fact, the value $\int_{r_1}^{r_2} f(r)r^2dr$ can be

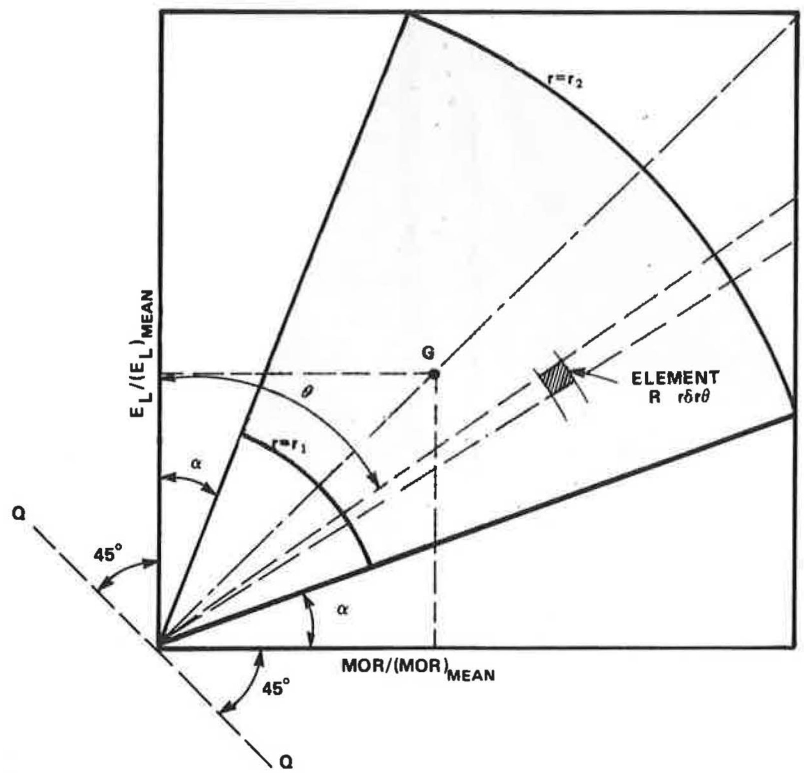

FIGURE 8 Evaluation of $\int_{r_1}^{r_2} f(r)r^2 dr$.

determined from knowing the position of the center of gravity of the probability distribution. (Details of the evaluation of Equation 9c are given in Jaeger and Bakht (3) and an evaluation is shown in Figure 8.

It is also shown in Jaeger and Bakht (3) that the expression for $k(\beta)$ can be simplified, on eliminating the integral with respect to r, to

$$k(\beta) = \{2 \tan \beta \int_{\beta}^{[(\pi/2)-\alpha]} \cos \theta \, \phi(\theta) \, d\theta$$

$$\div \int_{\alpha}^{[(\pi/2)-\alpha]} (\sin \theta + \cos \theta) \, \phi(\theta) \, d\theta\} \qquad (9d)$$

where $k(\beta$ is readily evaluated from) Equation 9d for any assumed $\phi(\theta)$.

Although the details of the mathematics may not be of much interest to engineers, the underlying physics of how the collapse builds up certainly are. Referring again to Figures 6 and 7, for small values of deflection, the boundary $\theta = \beta$ does not go through the area defined by the scatter of points. As deflection continues to increase, and the radial line continues its motion, a stage is reached at which the radial line reaches the angular position α of Figure 7. From that point on, the most vulnerable beams (i.e., those with the smallest values of MOR/E_L) begin to fail. For a time, increasing values of deflection, and the corresponding increases in the angle β, are accompanied by further increases in the load-carrying capacity, but the rate of increase decreases steadily. Eventually, a value of β is reached for which $k(\beta)$ is a maximum, after which it reduces. The maximum value of $k(\beta)$ gives the failure state of the bridge, in accordance with Equation 6.

The equation for $k(\beta)$ is given as

$$k(\beta) = 30.588 \tan\beta (2/3 \cos^3\beta + 3^{1/2}/2 \sin\beta - 5/6) \qquad (10)$$

for values of β greater than 30° for Red Pine (3).

Figure 9 is a graph of ultimate load factor $k(\beta)$ plotted against the angle β. The maximum is reached when β is 0.62 radians (i.e., 35.52°), at which point $k(\beta)$ has the value 0.639. Hence, for Red Pine in uniform deflection,

$$M = 0.639 \, M_T$$

$$= N \{0.639 (M_{ult})_{mean}\} \qquad (11)$$

From Equation 11, the ultimate failure load of the bridge is reduced (because of the scatter of values) to 0.639 of what it would have been if all beams had had the same M_{ult}--that is, if all had had $(M_{ult})_{mean}$, and the same value of E_L--that is, $(E_L)_{mean}$. It should be emphasized that the reduction in the ultimate failure load as estimated by Equation 11 is not brought about by having beams that are, on the average, weaker than usual. The beams are taken to have an average strength that is entirely normal, and it is the scatter of (MOR/E_L) above and below the mean that gives rise to the progressive failure of the bridge and the reduction in its ultimate load-carrying capacity as compared with the "no scatter" situation.

The reduction factor of 0.639 that appears in Equation 11 is derived from the probability distribution of the quantity (MOR/E_L), and the ultimate failure moment M given by Equation 11 is, in fact, the mean value (i.e., the expectation) of a random variable. The derivation of a failure load for design purposes also requires the determination of the standard deviation of this random variable. That determination is given in Jaeger and Bakht (3). It

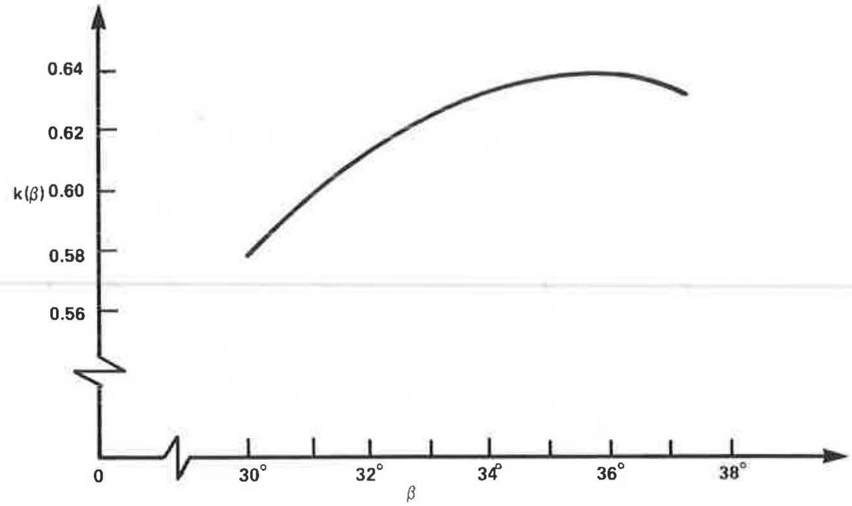

FIGURE 9 Plot of k (β) against β.

is sufficient to note here that for laminated timber bridges, the Number (N) of beams is large and, hence, the variability of the ultimate failure load about the mean value given by Equation 11 is small. It is instructive to repeat the preceding analysis for other assumed probability distributions. Fortunately, the reduction factor is found to be not sensitive to such changes; for example, if the function $\phi(\theta)$ is assumed to be constant for values of θ between α and $(90°-\alpha)$ as shown in Figure 10, even this quite marked change only has the effect of lowering the reduction factor to about 0.577 when α = 30°.

Figures 5, 10, and 11 clearly bring out one important point. This is that the shift from the perfect correlation case of Figure 11 to either Figures 5 or 10 gives a change in reduction factor from 1.00 to 0.639 or 0.577, which is much larger than the difference between the latter two. This confirms that a

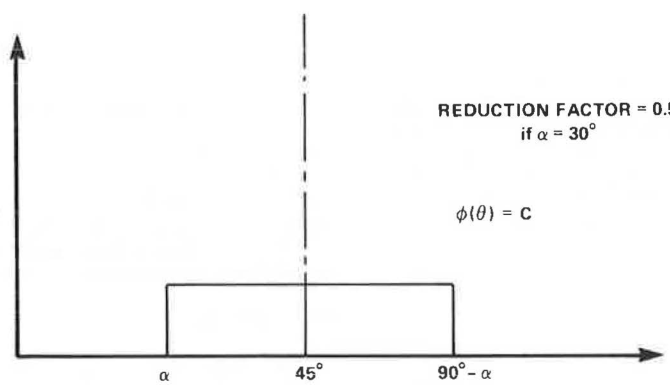

FIGURE 10 An extreme assumption for $\phi(\theta)$.

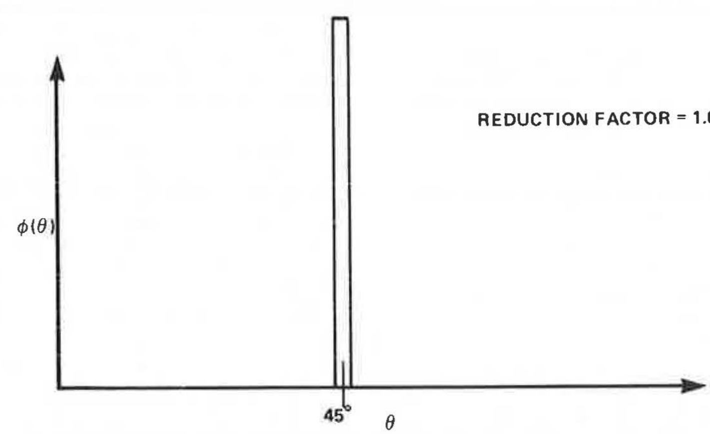

FIGURE 11 The plot of $\phi(\theta)$ for perfect correlation of E_L and MOR showing an indefinitely high spike of vanishing width—Dirac Delta Function.

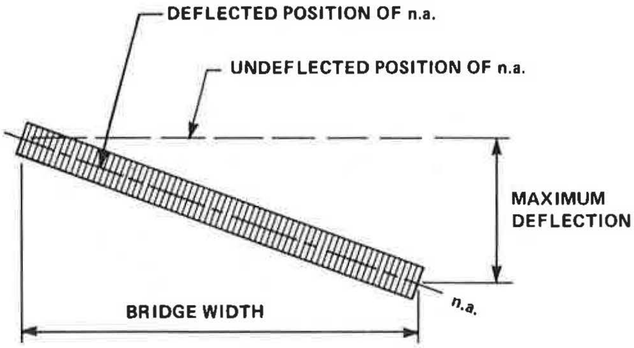

FIGURE 12 Linear deflection across a bridge cross section.

fairly good estimate of the reduction factor can be obtained by taking a reasonable representation of the scatter with respect to θ.

THE CASE OF LINEARLY VARYING DEFLECTION

The adoption of a completely uniform deflection pattern across a transverse cross-section is, of course, a highly idealized case, which can be regarded as one extreme. For purposes of comparison, the second deflection pattern considered is that of Figure 12, in which the deflection is assumed to vary in a straight-line manner from zero at one side of the bridge to a maximum value at the other. If all beams are taken to be identical with $(E_L)_{mean}$ and ultimate moment $(M_{ult})_{mean}$, then failure is reached when the deflection of the right-hand end reaches the value that gives $(M_{ult})_{mean}$ at that end.

Again, in making the assumption that the moment of resistance of a failed beam falls immediately to zero, it is readily seen that attainment of $(M_{ult})_{mean}$ at the right-hand end leads at once to the collapse of the bridge and that

$$M_T = 0.5N\ (M_{ult})_{mean} \qquad (12)$$

In Equation 12, the factor 0.5 arises from the linear variation of deflection across the cross-section. The estimation of total failure moment for this deflection pattern [taking into account the scatter in values of (MOR/E_L)] follows the same general line as that for the uniform deflection cases, except that now each element of the cross-section has its own deflection and, hence, its own radial line on the plot of Figures 2 and 3. Figure 13 shows the situation in which the deflection of the right-hand end gives the radial line $\theta = \beta$. Then, a width $B(\tan\alpha/\tan\beta)$ of the cross-section is known to be still unfailed, although the right-hand portion of the cross-section is composed of elements that have increasing probabilities of failure as one moves from left to right. [Details of the estimation of the ultimate failure moment are given in Jaeger and Bakht (3).] In the case of Red Pine, the result is

$$M = 0.415\ N\ (M_{ult})mean \qquad (13)$$

In this case, it is seen from Equations 12 and 13 that the reduction factor in ultimate load-carrying capacity is 0.830, as compared to 0.639 in the case of a uniform deflection. This relatively better performance is entirely to be expected because with the pattern of deflection now assumed, a fairly large fraction of the cross-section is known to be not involved in the initiation of failure, so that the

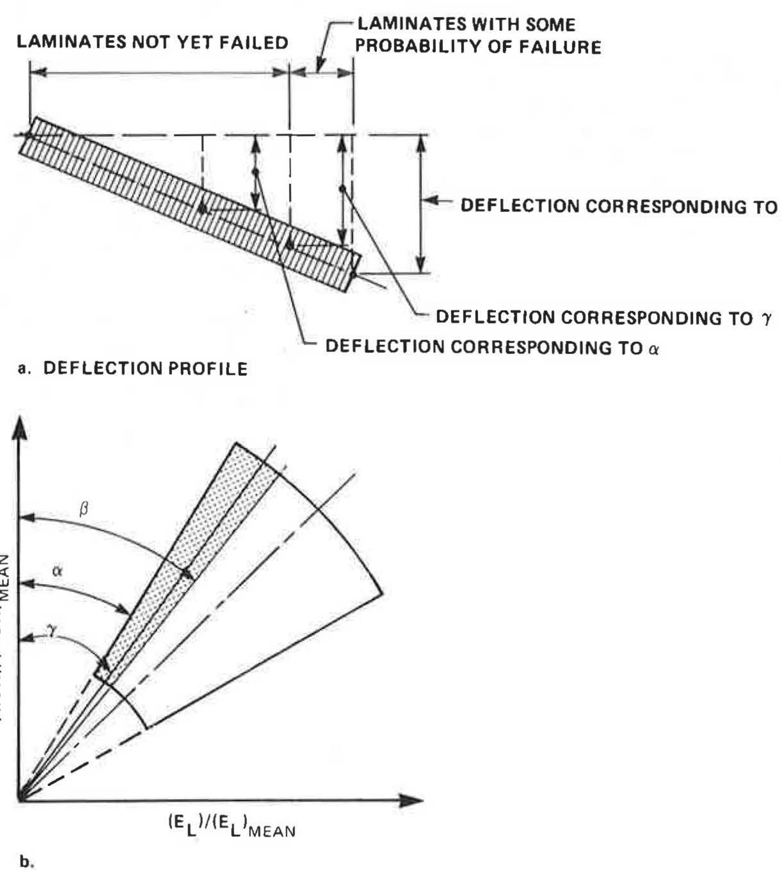

FIGURE 13 Identification of failure zone.

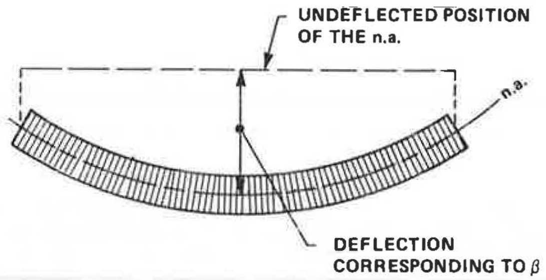

FIGURE 14 A nonuniform deflection pile.

influence of the scatter of values of (MOR/E_L) is much reduced.

The procedure for estimating ultimate moment capacity for any general deflection pattern, such as that shown in Figure 14, follows the same steps. The maximum deflection of the cross-section corresponds to a radial line $\theta = \beta$ in the plot of Figure 7. Any other point on the cross-section has a radial line $\theta = \lambda$ where $(\tan\lambda/\tan\beta)$ is the ratio of deflections for the two points concerned. Estimation of the ultimate moment capacity then follows using the method given in Jaeger and Bakht (3). In practice, the reduction factor for a uniform deflection, which is 0.639 in the case of Red Pine, is found to be the lowest reduction factor that occurs.

CONCLUSIONS

The failure loads of laminated timber bridges cannot be safely estimated by treating all of the beams as having the same (average) E_L and MOR. For Red Pine, a safe estimate of failure load is about 60 percent of the failure load obtained in that way. The estimate of failure load is not sensitive to the probability distribution that is assumed; provided that this distribution is fairly near the truth, the reduction factor that is due to the progressive nature of collapse can be predicted with reasonable accuracy. It is noted that this paper deals with estimation of the expectations of failure load (i.e., with estimation of the mean value of a random variable). The estimation of the standard derivation of this random variable is outlined elsewhere (3).

NOTATION

Following is a list of definitions for the variables used in this paper.

E_L = modulus of elasticity of a timber beam in the longitudinal direction;

$(E_L)_{mean}$ = mean value of E_L for the species of timber concerned;

MOR = modulus of rupture of a timber beam;

$(MOR)_{mean}$ = mean value of MOR for the species of timber concerned;

$E_L/(E_L)_{mean}$ = coordinate axes for representation of probability distribution of a given species of timber;

$MOR/(MOR)_{mean}$ = coordinate axes for representation of probability distribution of a given species of timber;

(r,θ) = polar coordinates in the plane of $E_L/(E_L)_{mean}$ and $MOR/(MOR)_{mean}$, with θ being $E_L/(E_L)_{mean}$;

$p(r,\theta)$ = a probability distribution;

α = minimum value of θ for which the probability function $p(r,\theta)$ has non-zero values;

$f(r)$, $\phi(\theta)$ = variations of $p(r,\theta)$ in the r and θ directions, respectively;

β = the value of θ corresponding to the maximum deflection in the cross-section of a bridge; that is the maximum value of λ; and

λ = the value of θ corresponding to the deflection of a representative point on the cross-section of a bridge.

ACKNOWLEDGMENTS

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A Prefabricated Modular Timber Bridge

J. D. PARRY

ABSTRACT

Part of the research program of the Transport and Road Research Laboratory in the United Kingdom is the study of road transport problems in developing countries. The unit that specializes in this work has carried out an assessment of a new design of timber bridge. It is a truss type made from prefabricated panels and is appropriate for use in countries that need low-cost bridges, but have only a supply of relatively small-section timber. The design is now being developed and promoted in Honduras, Madagascar, and other countries by the Timber Research and Development Association under sponsorship by the United Kingdom and United Nations Industrial Development Organization.

There are few areas of the world that are not now open to road transport. Its importance in the developing world has increased to the point where the economic and the social order are dependent on it, as they are in the western world. The creation and maintenance of a road network is extremely costly for countries that have little to export, so they are obliged to use as much as their own indigenous resources as possible, both in the road pavements and in the bridges. This has led to a variety of different bridge designs using different materials around the world. The Chinese are known for their concrete arch bridges. Several Bailey-type steel frame bridges still exist in areas where they were used as temporary bridging by armies in times of war. Timber bridges are found everywhere, but the type of timber available and the ability of the constructors have led to the development of different types of timber structure.

The most primitive (and still the most common) is the simple beam, which can take the form of a pedestrian bridge or may be used in multiple forms as a highway bridge.

The cantilever, although not as old as the beam, has also been in use for many centuries, as a bridge for pedestrians and pack animals, and, more recently, for motor vehicles.

Timber trusses, although they played such an important role in the United States during the last century, are not well known in developing countries today. This seems strange at first because the pioneering work is complete. However, Howe, Warren, Town, and others developed sound designs and proved their worth by building bridges, some of which have lasted more than a century. Each bridge, however, required the personal attention of a competent engineer. In developing countries today, trained men are usually responsible for all the engineering projects in a large district. It is understandable that they prefer the simpler, if more expensive, technology using concrete and steel prefabricated parts for their bridges.

It is rare today in developing countries to find a timber bridge on a trunk road. Even where the vertical alignment and waterway permit short spans, more permanent materials are preferred for the high investment roads. This is not always justified, however, because many of the roads are designed for a 20- to 25-year life, after which time extensive rehabilitation would be required or a new road would be built. A timber bridge could serve without extensive maintenance for at least that time.

New timber bridges are generally confined to the rural access roads. These roads are usually surfaced with gravel and built to a basic standard for low-volume traffic. They provide the link between grower and market and promote many forms of contact between town and country, improving communications for purposes of education, administration, mobility of labor, and tourism. These roads serve groups of villages or large, sparsely populated areas, whose economy has now changed and come to depend on them. When an emergency occurs, such as an epidemic or a famine, they can literally become the lifeline of the people.

Bridges are built on these roads only if drifts, causeways, and culverts are not able to provide water crossings for most of the year. Reinforced concrete is used if there is no local timber. Spans are short to keep the technology simple, even if this means that piers are required. All too often they are built to cope with the normal flow, and fail when high water carries away the deck or damages the abutments. Consequently, timber bridges tend to be found on the less trafficked roads, away from the centers of civilization--on the type of road that can make up 80 percent or more of a developing nation's network. The bridges are frequently uncounted and almost never maintained in a systematic manner. Described in this paper is a new design of timber bridge suitable for use on these rural access roads.

A PREFABRICATED MODULAR TIMBER BRIDGE

There is still a great need for low-cost bridges in many countries, and today there is far less large-section timber available, especially of the hard woods, to make long beams. Bridges are required that use small-section timber and the faster growing soft woods are more commonly and cheaply available now--even from areas that traditionally provided hard woods. Truss-type structures answer the requirement and many fine truss designs came from Europe and particularly the United States in times past, but they have not proved altogether suitable for developing-country conditions.

Because of this, a new design of modular truss form was developed in Nairobi, Kenya. The early work was sponsored by the United Nations Industrial De-

Overseas Unit, Transport and Road Research Laboratory, Crowthorne, Berkshire, England.

FIGURE 1 The 4-truss bridge at Nyeri.

velopment Organization (UNIDO) and a few of these bridges were built in Kenya in the 1970s. A report on the design was published in 1981 by the Transport and Road Research Laboratory (1) in the United Kingdom, after extensive examination and some development of the original work. Then, the Timber Research and Development Association (TRADA) took it up (2) and has promoted its development in several countries, under sponsorship from the British government and UNIDO.

The purpose of this project was to provide relatively cheap bridges to carry light commercial traffic in rural areas. The design that emerged requires largely semi- and unskilled labor and has the advantage that the bridges can be erected quickly and, if necessary, can be dismantled and reerected elsewhere. As with the Bailey-type bridge, there is one basic unit, which is reproduced and stored in readiness for use and can be used to build bridges of various spans and load-carrying capacities.

GENERAL DESCRIPTION

The bridge is a truss type, with the road deck carried on top of the trusses (Figure 1). The upper chords of the trusses, the verticals, diagonals, bracings, and deck are all constructed from timber. The bottom chords and the joints were designed to be made from mild steel.

Each truss is assembled from a number of identical frames (Figure 2), and is prefabricated and transported to the site together with the bottom chords. The frames are made of rough, sawed boards that are 50 mm thick, and are dowelled and nailed together to form an inverted triangle 3-m long with a vertical brace. They weigh about 140 kg each and are all made in the same manner. The frames fit end-to-end with steel or timber bottom chords (Figure 3) to form the trusses. An even number of parallel trusses is required, depending on the load capacity, the span of the bridge, and the quality of the timber. Design tables have been prepared using British and American loading standards and a large variety of timber types. As an example, the following table gives the number of trusses required when using East African cypress wood. Practical considerations limit the number of frames in a truss from 3 to 10.

	Span (m)					
Loading Duty	12	15	18	21	24	27
HA	6	8	-	-	-	-
H20-44	4	4	6	6	8	-
H10-44	2	2	4	4	4	6

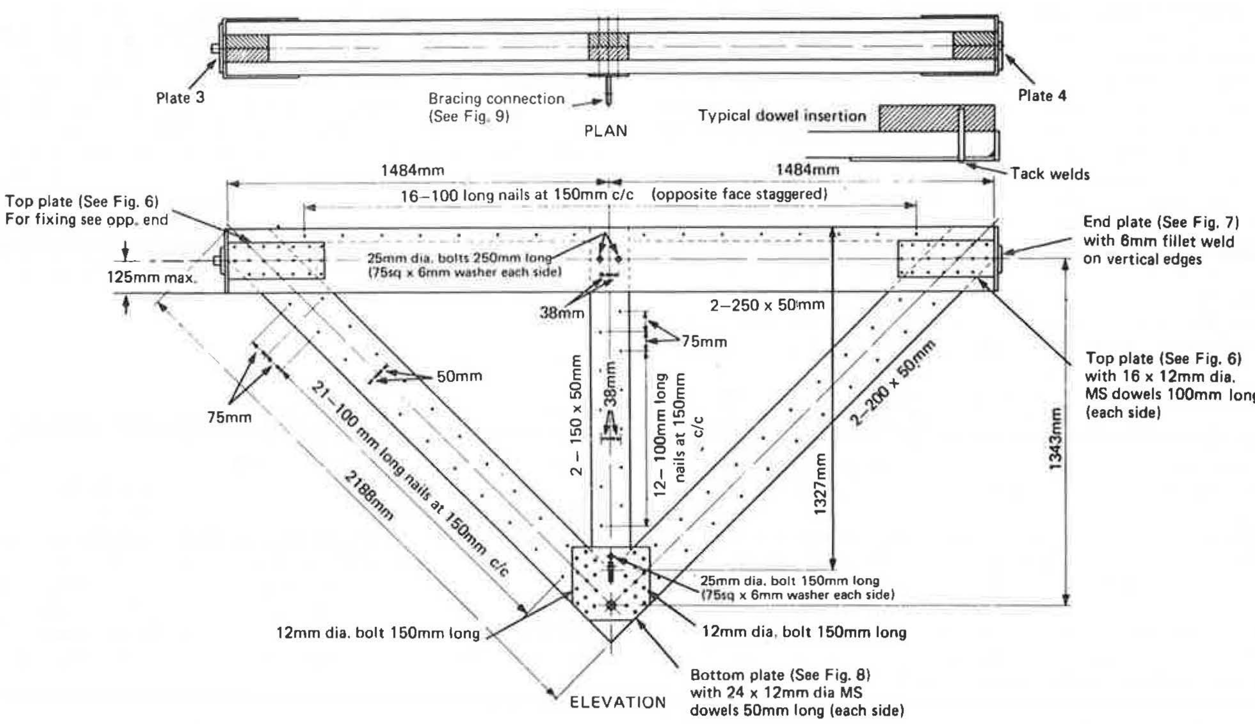

FIGURE 2 Frame assembly.

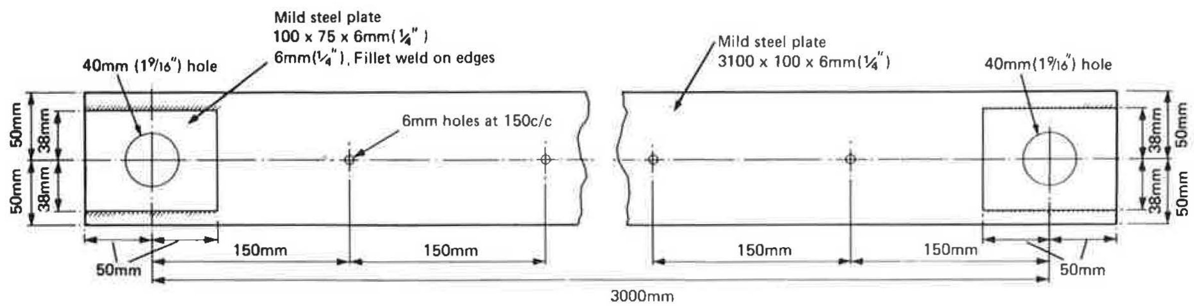

FIGURE 3 Bottom chord.

The trusses are connected by the timber deck (or transoms) and diagonal bracing members (both vertical and horizontal) between the trusses (Figure 4). Longitudinal running boards are nailed to the transoms. At their ends, the trusses are supported by angle brackets (Figure 5), which act as the bridge bearings. Stone, concrete, brick, or timber abutments may be used to support the brackets.

The main features of the design are as follows:

1. Local timber can be used;
2. The design is easy to fabricate;
3. The largest component measures 3 x 1.5 m and is light enough to be manhandled;
4. All the frames are identical and so may be made on a jig in a workshop, where inspection of the quality and finish is easier;
5. The cost is much lower than that of a steel or concrete bridge with similar loading capacity; and
6. It is usually limited to spans from 9 to 24 m.

Steel dowels are used to join the timber members at each corner of the frame. The two top joints are identical, each consisting of two steel plates (Figure 6), one on each side, through which the dowels pass, penetrating through the horizontal timber member and then into the diagonal (see Figure 2). There is no connection through the joint but the two top steel plates are joined when the end plate (Figure 7) is welded across them (a male end plate at one end of the frame and a female at the other). At the ends of the trusses, these end plates mate with the bridge bearings.

In the bottom joint, the two diagonal members are not joined directly by the dowels but each is dowelled to a bottom plate (Figure 8). Again, similar plates are used on each side of the joint and dowels are driven through them and into the timber from each side (see Figure 1).

In service, the horizontal top chords (A) (in Figure 9) are always in compression because of the residual weight and the applied load, whereas most of the diagonal members (B) are in compression or tension according to the position of the load. On the end frames of each truss, one diagonal member (C) is in permanent compression--and the other (D)

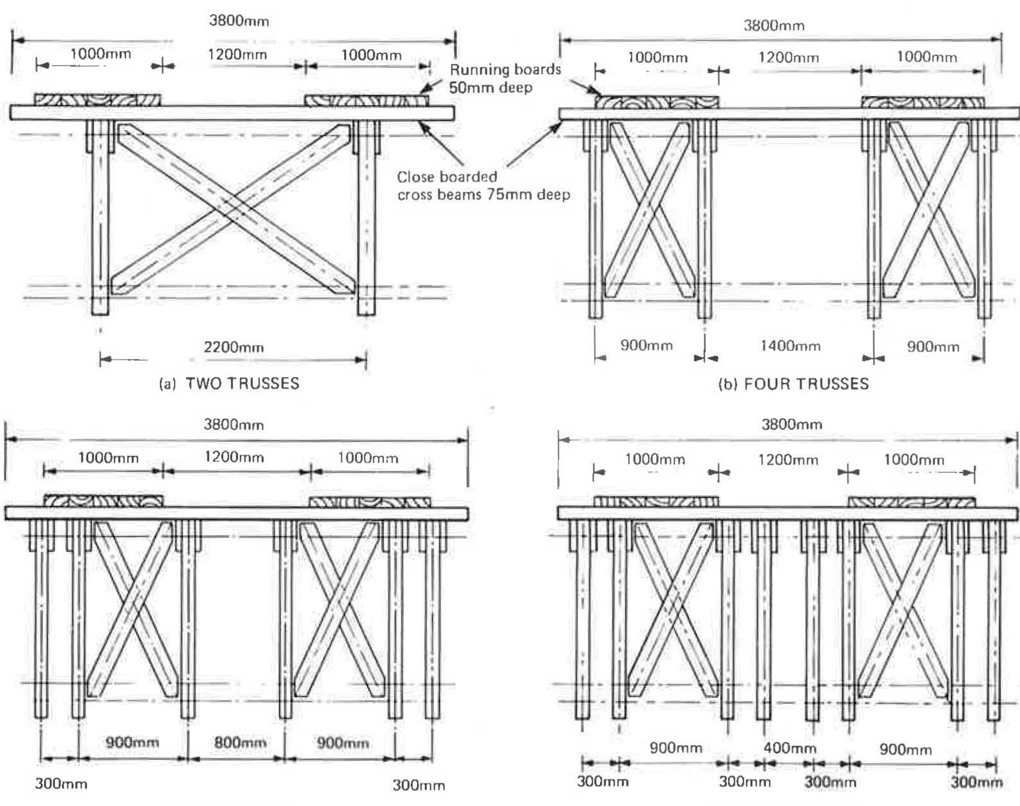

FIGURE 4 Typical cross-sections using (a) 2 trusses, (b) 4 trusses, (c) 6 trusses, and (d) 8 trusses.

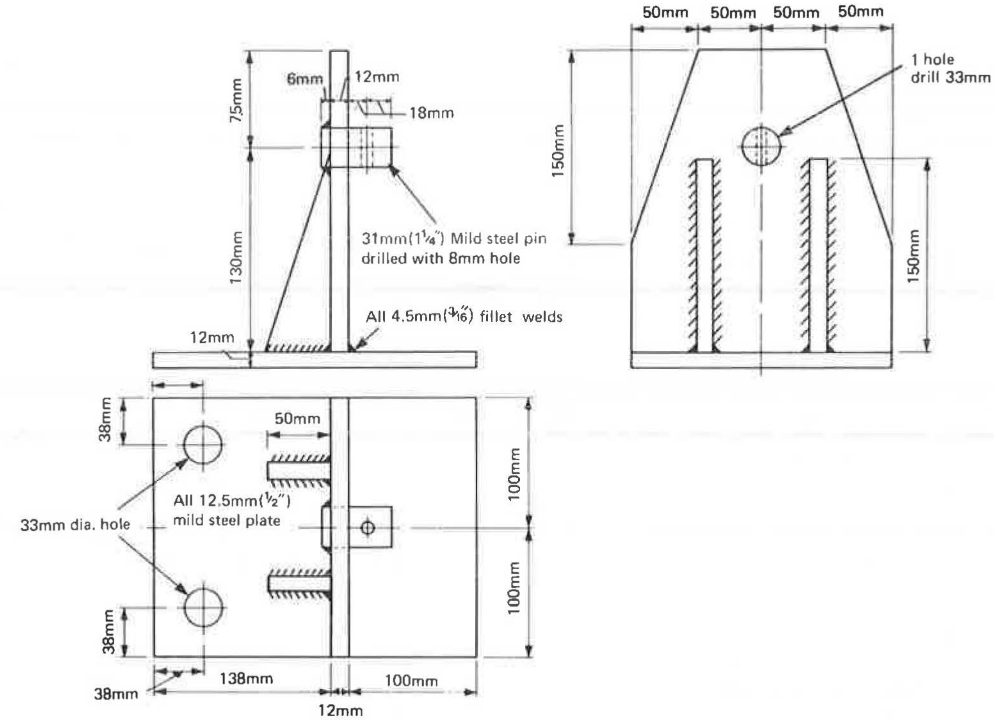

FIGURE 5 Bridge bearing (male); bridge bearing (f) similar but 31-mm (1.250-in.) diameter is omitted.

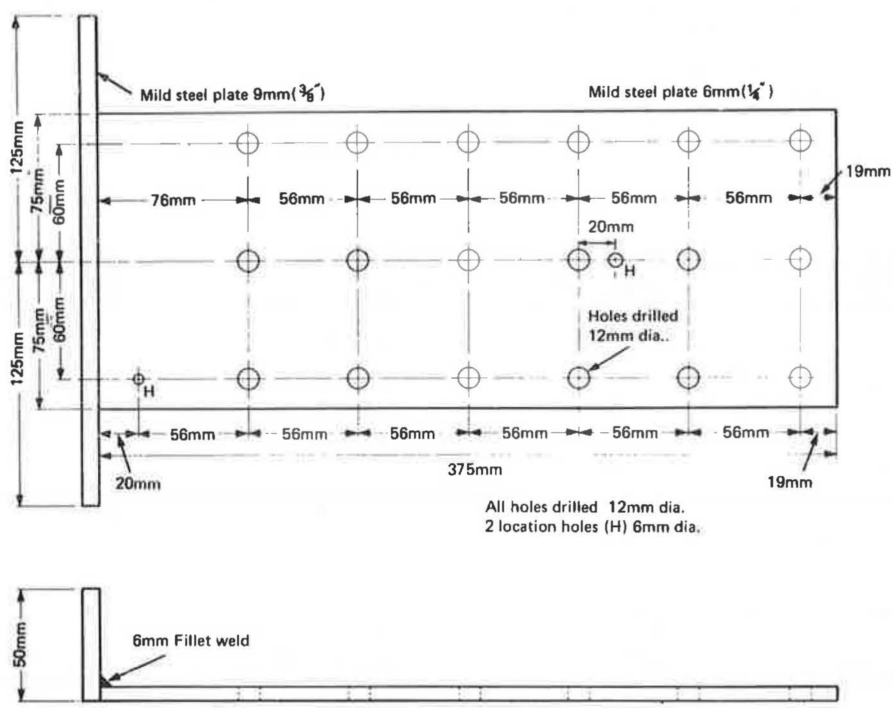

FIGURE 6 Top plate.

permanent tension. This multiple dowel joint is not common, but it has proved to be reliable both for the reversing load in the diagonal members and the end joint, which transmits the maximum shear load to the bearings.

Several types of deck have been tried. The preferred one is close boarded (Figure 10), with timbers 75 or 100 mm deep, nailed down to the trusses and to each other. The result is a strong stress-carrying deck that relieves the trusses of a large part of the load they would be expected to carry.

The bridge abutments are not included in the design. In Kenya, concrete block abutments were used for the bridges but timber or gabion abutment could be used provided suitable bearing surfaces are made to support the brackets that carry the weight of the bridge. These less permanent types of abutment are adequate for emergency or short-term use, but con-

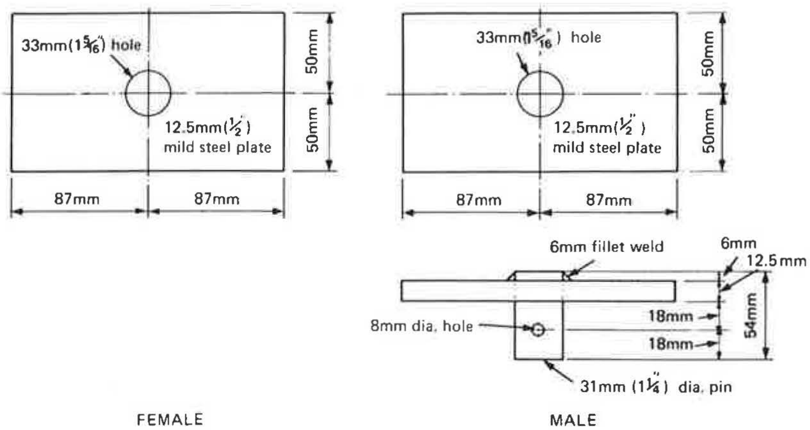

FIGURE 7 End plate.

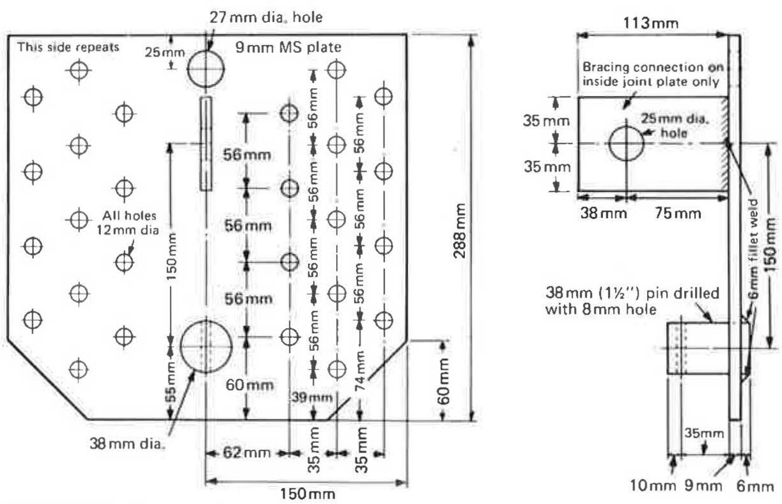

FIGURE 8 Bottom plate.

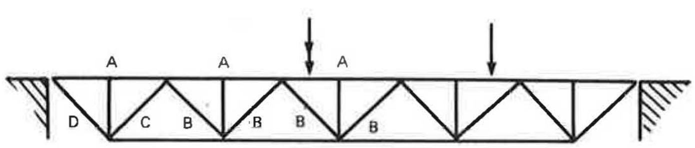

FIGURE 9 Bridge test at Isiolo.

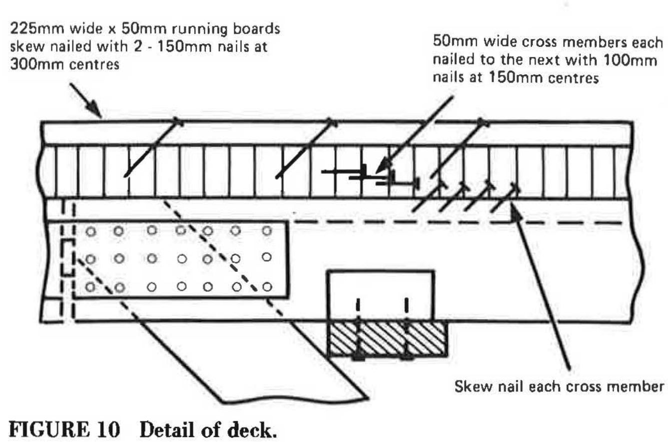

FIGURE 10 Detail of deck.

crete or masonry abutments may be required to last the full life of a bridge which, with good timber protection, is expected to be in excess of 20 years.

TIMBER PROTECTION

In Nairobi, each member of the frames after cutting was dipped for half an hour in a solution of dieldrin with a small portion of pentachlorophenol. This solution was also painted onto newly exposed surfaces after the holes had been bored for the bolts and dowels. On site, the soil was poisoned to a depth of 300 mm for a distance of 1 m behind the bridge abutments to guard against termite attack. (It should be noted, however, that long-term protection against termites is dubious, and the toxic effect of these chemicals on personnel not accustomed to wearing protective garments can be dangerous.)

MANUFACTURE

When the timber members of the frame are cut to size, it is important that lengths and angles are cut accurately and, for this reason, it is recommended that simple jigs be used at this stage. On site, it is necessary that the abutments are built or modified so that the bearing brackets may be placed exactly a multiple of 3 m apart. The frames are then assembled one by one from below, conditions permitting, or pairs of trusses, complete with cross bracing, are launched across the gap (Figure 11) suspended from a cable stretched between two derricks. The bearing brackets are then assembled and anchored to the abutments, and the deck and handrails are built.

DESIGN ASSESSMENT

A number of frames were made at the Transport and Road Research Laboratory and subjected to detailed examination and load testing. A truss was made of three frames and strains were measured in the members as a rolling load was passed across it. These tests resulted in the following conclusions:

1. The dowelled joints showed no sign of weakness during any of the tests;
2. Strains are not evenly distributed among the frames unless great care is taken during manufacture to ensure symmetry and squareness at the frame ends; and
3. Some minor modifications were made to the design resulting in an increase in frame strength.

Examination of the bridges in Kenya showed that

1. Use of a close-boarded deck significantly reduced the stress in the top and bottom chords. This effect probably diminished with time from about 40 percent in 1976 (when the bridges were new) to 25 percent in 1979.
2. The dowelled joints showed no sign of weakness in service, although the bottom joints tended to trap debris, which prevented the timber from drying and so could promote rot.
3. Stresses were not always well distributed between parallel members and trusses.
4. There was no deterioration in the steel parts, whether painted or not, after four years in service. This would not have been the case in a more severe environment.

[Details of the laboratory and field testing are given in Parry (1).]

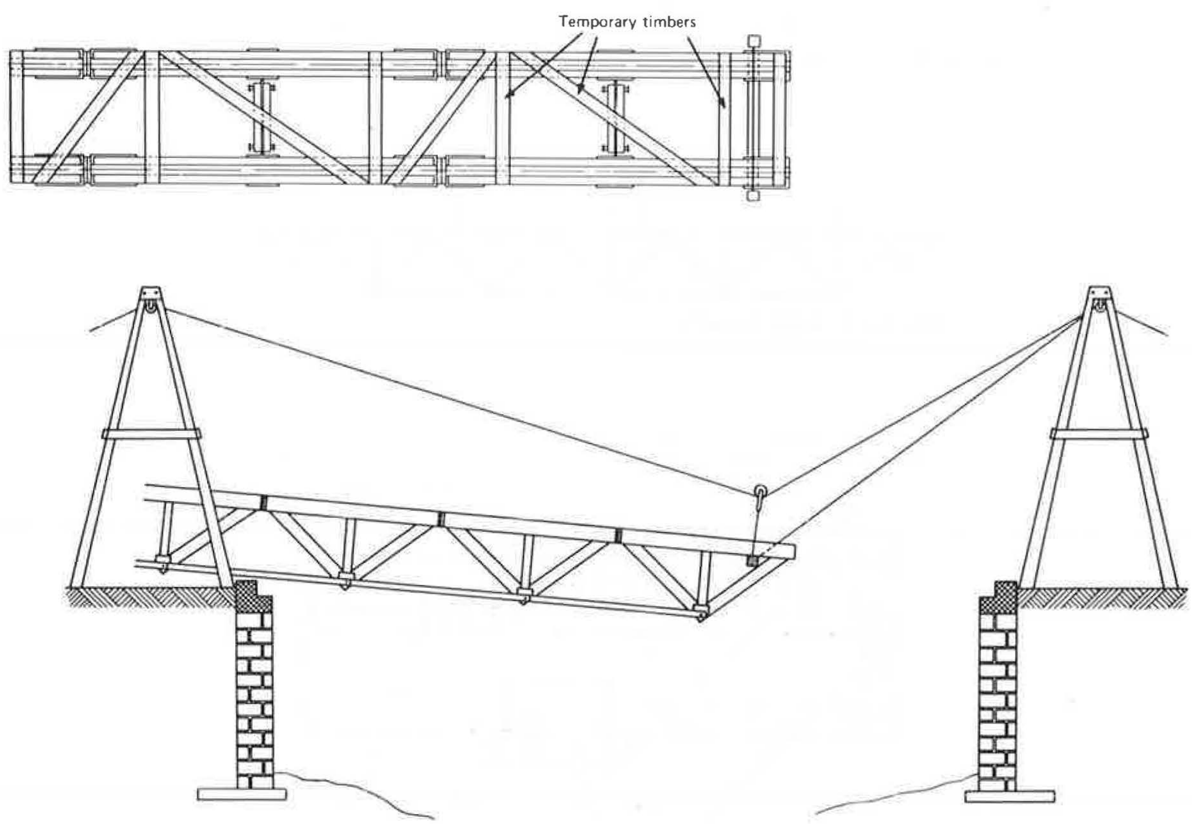

FIGURE 11 Launching a pair of trusses.

COST

Each site will impose conditions on bridge costs, which for a given span may vary by a large factor. Variations in cost attributed to nontypical foundations and abutments are not considered here. As with the design, the cost of a bridge must be determined for each individual circumstance. Following is a simple breakdown of costs for this design of truss and deck only. The prices quoted are the commercial prices applicable in Kenya expressed in United States dollars for a span of 18 m.

Material

Building grade Cypress 100 x 50 mm is $1.04 per m or $208 per m³. Assuming 30 per cent excess for large sections and 20 per cent excess for graded timber, the price becomes $325 per m³.

The following quantities are for a bridge with four trusses:

Material	Price ($ U.S.)	Cost ($ U.S.)
Deck	0.4 m³ per meter length at $208 each	83.20
Frames	0.28 m³ per meter length at $325 each	91.00
Steel plate and dowels	51 kg per meter length at $0.69 each	35.10
Steel chords	34 kg per meter length at $0.69 each	23.40
Nails and bolts	per meter length	10.40
Total	per meter length	243.10
	for 18 meters	4,375.80
8 bearings	44 kg at 0.69 each	30.29
Paint, wood preservatives, and soil poison		260.00
Total		290.29

Therefore, the material costs for an 18-m span are $290.29 + $4,375.80 = $4,666.09.

Wages

Wages for the staff are as follows (allowing 2 weeks for manufacture of jigs and frames):

Labor	Cost ($ U.S.)
5 laborers for 10 days at $3.90 per day	195.00
3 craftsmen for 10 days at $9.10 per day	273.00
Total	468.00
33 percent overhead on labor	156.00
Total labor for manufacture and erection	624.00

For comparison the cost of manufacture only is $4,666.09 for materials and $468 for labor, which totals $5,134.90.

In approximate terms, both Callender Hamilton- and Bailey-type bridges cost about 4 times this sum ex works, or about 5 times delivered by sea to Mombasa.

Steel RSJ beams, if available at the same price as the small sections previously referred to, would cost about $4,550. If imported, the cost would be about $6,500 and, in addition, some 15 m³ of reinforced concrete would be required for the deck, costing about $7,020. If cement were not available, a deck could be made with 8 m³ of timber, costing about $1,690. Transport costs to the site would be high for two steel beams that are each 20 m long and that each weigh 3 tons. Costs of these four types of bridges are summarized in the following table:

18-M Span Bridge--H10	Loading Price ($ U.S.)
Kenya timber bridge	5,200
Bailey/Callender Hamilton	26,000
RSJ with concrete deck	11,050-13,000
RSJ with wood deck	6,240-8,190

In Honduras, the prices estimated by TRADA for a 15-m bridge prefabricated as previously described are:

Item	Price ($ U.S.)
Materials	7,400
Launching equipment	4,000
The cost of plant to set up a workshop	18,000

MAINTENANCE

The maintenance of roads and bridges presents a considerable problem in the large majority of developing countries. In many cases, both pavements and bridges are deteriorating at an alarming rate. This results in the need for rehabilitation or rebuilding works long before the design life is over, and the high cost of rebuilding diverts funds from routine maintenance. This modular timber bridge requires no special maintenance other than that required by any timber structure, but regular inspection should be carried out, and this is often neglected in countries suffering from a shortage of qualified engineers.

At the Transport and Road Research Laboratory, a new bridge inspection guide is being prepared for use in developing countries. The premise of this guide is that under current conditions in some countries, inspection and maintenance is sadly inadequate, because the few available, trained staff members do not go into the field to make inspections. A handbook is therefore being prepared for use by the road foreman, who spends most of his time on the road, so that he can carry out regular preliminary inspections using a simple questionnaire. The accompanying book, which is aimed at district engineers, gives instruction on the preparation and maintenance of a bridge register and some guidance on the interpretation and followup of the road foreman's reports.

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The work described in this paper forms part of the research program of the Overseas Unit of the Transport and Road Research Laboratory. The work was carried out for the Overseas Development Administration, but any views expressed are not necessarily those of the Administration.

Ohio's Experiences With Treated Timber for Bridge Construction

JAMES E. BARNHART

ABSTRACT

Described in this paper are the various types of timber structures with which Ohio has had experience. It includes a discussion of the various timber bridges built in the 1930s, which normally were timber strip floors on steel beams on timber pile abutments and piers. Described also are some of the more modern types of timber structures used including glued-laminated transverse deck panels installed on existing steel beams; longitudinal laminated deck panels that are self supporting; and laminated, panelized timber box culverts. The specifics on these types include actual costs, ease of construction, and performance after installation. The panelized timber structures can be erected quickly with traditional construction equipment. Many treated timber structures, especially glued-laminated structures, have higher initial cost, but lower maintenance costs and a longer projected life.

Ohio has used treated timber in bridge construction for many years. The first bridges built in Ohio were of timber construction and the trend has continued through the years. The most common modern-day type of bridge that utilized timber was the steel beam on timber pile abutments and piers. Standard drawings dating from the early 1930s and most recently revised in 1965 are still being used although on a limited basis. The majority of these types of bridges still in use were built in the 1930s and 1940s. Ohio still has about 1,500 bridges with laminated strip floors in the state highway system. This does not include the hundreds of similar bridges on county and township roads.

Standard treated timber strip floors consisted of 3- x 6-in. dense, structural, southern yellow pine or select structural Douglas fir set on edge and fastened with steel clips to steel beams. The steel beams were generally spaced at from 3-ft to 3-ft and 4-in. centers. The specifications required nailing the individual boards together at 12-in. centers with 60d spikes. The fastening clips were spaced at 12-in. centers staggered on either side of the beams and fastened with 1/2-in. galvanized carriage bolts through the deck. These bridges, although not regularly being built today, lasted many years with minimum maintenance. It was not unusual for these timber strip floors, covered with 2 to 3 in. of asphalt concrete to last from 30 to 40 years with no maintenance except for an occasional overlay with more asphalt concrete.

By the mid 1950s, timber bridges were gradually being phased out by steel beam bridges with reinforced concrete decks. About the same time, road deicing salts were being used to a greater extent and this drastically shortened the life of these floor systems because the timber strip floors leaked, allowing the chlorides to attack the steel supporting beams. As was learned in the 1960s and 1970s, the deicing salts also attacked the new reinforced concrete decks and many started to spall badly after only 5 or 10 years of service. Unlike the reinforced concrete decks, the salts did not affect the treated timber decks, but the effect of the salt leakage onto the beams below was a major problem.

Ohio Department of Transportation, 25 South Front Street, Room 408, Columbus, Ohio 43216-0899.

The demise of nailed timber strip decks was also hastened by heavier loads and higher speeds. Timber decks tended to flex too much and loosen with age, at which time the asphalt-concrete wearing surface began to break up. Reinforced concrete decks had a higher resistance to impact caused by heavier loads and higher speeds.

Even with these drawbacks, it was obvious to many maintenance engineers that the treated timber decks properly constructed on steel beam superstructures would outlast the more modern reinforced concrete decks several times, with extremely low maintenance.

Because leakage through the strip floors was a considerable problem, as was keeping the floor clips tight, the Ohio Department of Transportation began to investigate the glued-laminated timber floors as a method of replacing the old strip floors. The glued-laminated floor looks similar to the strip floor just described, except that the individual boards are glued together at the factory and shipped to the site in manageable panel widths. This type of floor appeared to have several advantages, for example,

1. It was watertight (except at the joints);
2. It was panelized into 4-ft wide panels that could be placed much faster than the conventional strip floor; and
3. The individual boards, being glued together, could distribute the loads more efficiently than the individually nailed boards thus minimizing the possibility of loose floor clips.

In 1976, the Ohio Department of Transportation chose to specify a glued-laminated floor on two different types of bridges. One was a 3-span, steel beam bridge on timber piers and abutments built in 1932. This bridge is 120-ft long and 24-ft wide. The bridge is located in Jackson County on Ohio Route 124, which is a coal mining area and thus carries a considerable amount of coal truck traffic. The abutments and piers, even though they were 44 years old, were in excellent condition and required no work. It is interesting to note that the pH level of the stream under this bridge is quite low; however, the timber piles are still in good condition. The existing steel beams were 21 WF 68 spaced at from 3 to 3.75 ft and were still in good enough condition to reuse with only minor repairs. The beams were originally discontinuous over the piers and as a part of this contract, were made continuous with the use of welded steel splice plates. The tops of the beams were sandblasted, primed, and topcoated before the floor was placed. The bridge is on a 25-degree skew and thus the floor was furnished with skewed ends. The floor specified was 5.125-in. thick with predrilled holes for the floor clips. The preservative was pentachlorophenol and was to be applied only after all gluing, cutting, and drilling of the panels was complete.

The panels were to be placed with no dowels between the joints. A mastic was applied to the mating faces of the panels in an effort to waterproof the joint and a small piece of aluminum flashing was placed over each beam directly under the panel joints. The intent of the flashing was to divert any drainage that might seep through the joints and to divert it away from the steel beams. The flashing was insulated from the beams by two heavy coats of paint, thus minimizing the possibility of bimetal corrosion. The 27 panels were placed in 2 days.

The only two problems that occurred with this project were: (a) the panels were oversaturated with the pentachlorophenol and were still dripping after being unloaded at the site; and (b) the top surface of the panels was too smooth, which resulted in some pushing and shoving of the new asphalt concrete overlay. It has since been learned that a vacuumed and slightly irregular surface is needed on which to place the asphalt concrete. The total cost of this project was $78,366.70 and the cost per square foot of the glued-laminated flooring was $8.30 in place. This route carries an average of 1,200 vehicles per day--100 of which are trucks. The deck is still in excellent condition; however, the wearing surface has some cracks in it.

The second bridge, which was reconstructed in 1976 using a glued-laminated timber deck, is in Lawrence County on Ohio Route 243. The bridge is a 129-ft-long through truss, 14.5-ft wide, built in 1884. The truss itself was in relatively good condition; however, the floor system, which consisted of steel stringers with a nailed timber strip floor, was badly deteriorated. The steel stringers were replaced with new continuous steel stringers and the new glued-laminated floor 5.250-in. thick was clipped to the stringers. The new stringers were spaced at 3 ft-4 in. The floor is still rated in good condition--no repairs needed although the wearing surface has some minor cracks. The total cost for this project was $80,440.44, and the bid price per square foot for the glued-laminated timber decking was $11.50 in place. The traffic count on this bridge is 900 vehicles per day, 40 of which are trucks.

There have been at least five other bridges reconstructed using glued-laminated timber decking since 1976 with the most recent being on Ohio Route 233 in Gallia County and Ohio Route 327 in Vinton County. These two bridges were sold as one project in July 1984. The bridge on Route 233 is 69-ft-and-8-in. long x 28-ft and 6-in. wide, and is a 3-span, steel beam bridge built in 1933. The existing timber strip floor and the existing steel beams were badly deteriorated and, as a result, were completely replaced. The existing steel H-pile abutments were encased in concrete. The new beams furnished were W 14 x 43 spaced at 2-ft and 7-in. centers. The floor specified was a 5.125-in. thick panel 4-ft wide and treated with pentachlorophenol. The bridge is on a 20-degree, 30-min skew and thus all panel ends were skewed.

The tops of all the beams were painted before placing the decking. All cutting and drilling of the flooring was to be done before treatment. The floor was fastened to the beams with galvanized steel floor clips and carriage bolts through the floor at 2-ft centers on each side. The second bridge on this project is 72-ft and 2-in. long x 27-ft and 3-in. wide on a 43-degree, 30-min skew. The existing 20 WF 69 steel beams on 2-ft and 5.5-in. centers were reused.

The total cost of this project was $310,657.18. The successful bidder on this project bid the 3,953 ft^2 of glued-laminated timber decking at $18.00 per ft^2 in place. The average of the four bids for this item was $17.90 per ft^2 in place.

The average daily traffic count on Ohio Route 233 is 360 vehicles per day, 40 of which are trucks. The average count on Ohio Route 327 is 430 vehicles per day, 30 of which are trucks.

The eventual cracking of the asphalt-concrete wearing surface over the panel joints was expected because it was decided not to specify the widely accepted steel dowelling between the panels. This process requires that steel dowels be partially embedded in the matching face of one panel and the adjacent panel be match-drilled to accept the protruding dowel ends. This would give better load transfer between the panels but was anticipated to be difficult to place in the field unless the holes were somewhat oversized, which would defeat the purpose.

In 1979, the Ohio Department of Transportation became aware of another type of timber bridge construction that appeared to have some advantages in spans up to 38 ft. Because treated timber was thought to outperform reinforced concrete and steel from a maintenance standpoint, the concept of longitudinal, laminated deck panels, which were self supporting without the need for structural steel underneath, was very appealing.

In 1979, a recently constructed longitudinal laminated bridge built by the County Engineer of Hancock County was examined. The bridge consisted of three spans of timber on reinforced concrete piers and abutments. The longitudinal, laminated deck slab consisted of 4-in. thick x 8- to 16-in. wide (depending on spans) boards set on edge and mechanically laminated together using 3/8-in. diameter x 15-in. long galvanized ring shank dowels. The decking was furnished in approximately 6 ft wide panels that were completely self supporting, eliminating the need for steel beams in spans up to 38 ft. The individual boards were cut to size, drilled, and pressure-treated before lamination.

A site had already been selected for using an all-timber bridge so that it would blend into the park-like setting in which it was located. (This location was on Ohio Route 551 in Pike County at Lake White State Park.) The existing bridge was a 147-ft long x 14-ft and 7-in. wide, 2-span pony truss. The bridge was badly deteriorated and beyond repair. It was also a 1-lane bridge on a relatively heavily traveled road during the summer months. The current traffic count on this road is 350 vehicles per day including 10 trucks. It was decided to specify an all-timber bridge for this location and to use longitudinal, laminated deck panels. The piers and abutments were also designed to be treated timber using capped pile abutments with timber backing and capped pile pier bents. The new bridge was designed to be 129-ft and 4-in. long x 28-ft wide and on a 45-degree skew. The four equal, 32-ft spans were designed to use 4-in. x 16-in. individually treated timbers set on edge and laminated into 6-ft wide panels using .375-in. diameter x 15-in. long galvanized rink shank dowels. The type of treatment for this bridge was creosote.

Bids for this project were opened on July 7, 1981, and the successful bid was $222,254. The cost for the 3,621.24 ft^2 of the Douglas fir deck panels was $20.44 per ft^2 in place. The average of all six bids for this item was $24.03 per ft^2 in place. This bridge is expected to last many years with minimal maintenance.

Because this bridge was built, several more longitudinal deck panels have been purchased to replace existing steel beam and timber strip floor bridges. A recent order was for an 18-ft long x 30-ft wide span bridge. The panels were to be panelized in 4- to 6-ft widths and be 10-in. thick. The specified timber to be used was Number 1 coastal region Douglas fir. This order was delivered to the Ohio Department of Transportation district yard at a cost of $18.50 per ft^2. The Ohio Department of Transportation current specifications require treatment with Number 1 creosote petroleum in accordance with American Wood Preservers' Association Standard C 14-84. The required retention is 12 lb per ft^3. All members must be precut and prebored before treatment. Unlike the glued-laminated transverse panels, which are treated after they are panelized, these longitudinal panels are panelized after the individual boards are treated. All panel sides, except the fascia panels, consist of one-half-height, 4-in. boards such that the individual panels can be shiplapped together. The laps are predrilled to allow for 5/8-in. diameter drive pins. In all cases, the timber panels can be easily handled with the use of a backhoe or small crane.

Another type of timber structure that has been used in Ohio to a limited extent is the laminated timber box culvert. These structures are constructed of individually treated boards that are laminated together by drive spikes and/or through bolts. The individual panels are furnished in 4- to 6-ft lengths with a special finger-type interconnect where the panels join together in the corners. These units have been installed as either single-, double-, or triple-cell boxes. They have worked extremely well in streams with low pH levels. The oldest two such structures on a state highway in Ohio was installed in 1964. One is a triple cell (5 ft, 5 ft, 5-ft) and the other a double cell (5 ft, 5 ft). The current pH level of the streams going through these boxes is 4.0. Both structures are still functioning with no deterioration of the timber. The interior partitions of the triple-cell structure at the inlet end are slightly distorted, which is apparently caused by debris in the stream. Several such box culverts have been installed in situations where the pH level of the latter is quite low. (Note: there are few types of materials that can be used in low-pH-level stream crossings.) Galvanized corrugated steel rusts through in a few months and concrete structures must be protected with a tile facing.

A recent installation in 1977 was a 7-ft span x 5-ft rise x 38-ft long, single-cell box. The prefabricated culvert cost $5,600 and was installed by maintenance forces in 5 hr. A Gradall excavator was used to aid in the installation. It is important that these structures be installed on carefully prepared bedding and that the sidewall joints be staggered from the top and bottom panel joints. Care also must be taken to keep the units square until backfilling is complete. (Several of these units have also been built by Ohio Department of Transportation maintenance forces and used to extend existing narrow concrete box culverts.)

SUMMARY

It has been determined that the old, conventional, nail-laminated strip floors lasted an average of 30-40 years with minimal maintenance. Reinforced concrete decks have historically required considerable patching within 20 years and the average life before complete replacement has been 30-35 years. A typical reinforced concrete deck costs $13.90 per ft^2 and the last glued-laminated timber deck cost $18.00 per ft^2. The longitudinal timber decking can be compared to a complete superstructure replacement using prestressed concrete box beams. A recent prestressed box beam project cost $22.58 per ft^2. (This compares favorably with the longitudinal deck project, which cost $20.44 per ft^2.)

In general, properly designed and treated timber can and should be seriously considered as an alternative to concrete and steel.

ACKNOWLEDGMENTS

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New Ideas for Timber Bridges

MICHAEL G. OLIVA, ROGER L. TUOMI, and A. G. DIMAKIS

ABSTRACT

Because nearly one-half of the bridges in the United States are listed as either functionally or structurally deficient lends impetus to search for new ideas for building and maintaining bridges. Most of these bridges are on secondary and rural roads where spans are short, which makes timber a prime candidate for construction. The Forest Service, U.S. Department of Agriculture, with a vast number of bridges under its care, is cooperating with the University of Wisconsin to investigate new techniques for timber bridge design and construction. Described in this paper are promising new ideas, which are being examined for bridge construction, rehabilitation, and production of efficient performance and low cost in timber bridge systems. The scope of the research covers reviewing recent advancements in bridge design, applying new techniques to enhance the performance of common bridge types, and evaluating totally new structural configurations for use in bridges. The performance enhancement may be achieved by increasing the transverse spread of load through distributor beams, prestressing techniques, and dowels. New structural configurations include plane trusses, multileaf trusses, and composite beams assembled together to produce parallel chord longitudinal deck systems. The research involves theoretical evaluations to estimate span capabilities. The more promising concepts, which are based on structural potential, estimated cost, and simplicity, are being experimentally tested to verify models, theory, and design procedures.

The bridge problem and its impact on the nation's transportation infrastructure is widely recognized. Nearly one-half of our bridges have been listed as either functionally or structurally deficient, and about 75 percent of them are on secondary and rural roads. The replacement cost of off-system deficient bridges has been estimated at $18.8 billion (1).

Wood has already proved to be a competitive material for short-span bridges. However, wood can become even more attractive if current limitations on available timber, connection methods, configurations, and code-recognized behavior can be improved. Current practice dictates the need for large sizes in preferred species, notably Douglas fir. New concepts may permit the use of smaller sizes, lower grades, or alternative species that will encourage the use of local and low-cost timber for bridge construction.

The development of new ideas for timber bridges requires the screening of a wide array of different concepts in the hope of finding a viable system or systems that will supplement or replace current practices. Recently developed bridge-building methods can be adopted, existing techniques can be upgraded using new technology, new materials can be used in previously unseen forms, and new design techniques may be implemented using modern analytical tools.

Some recent concepts have been introduced by others in an effort to solve particular bridge problems. The Canadians have been leaders in new bridge technology and some interesting work has recently been completed in the United Kingdom on segmental bridges. Other ideas might be reclaimed from the past. Some older systems have disappeared because of an intrinsic weakness (e.g., fasteners) and new techniques and technologies may overcome these problems.

New materials and technologies developed for other purposes may find applications in bridge construction. Foundation Grade Plywood, developed for building construction, has proven durability and might serve as the structural diaphragm for bridge components. Metal plate truss connectors might provide a better way to join structural elements. New design configurations or better methods of structural analysis might lead to more efficient bridges. Modern analytical tools (most notably high speed computers), allow the investigation of complex systems previously not amenable to slide rule solutions.

The Forest Service, U.S. Department of Agriculture, has a vast number of bridges under its care that represent a sizable investment for both new construction and maintenance. The Washington Office, Division of Engineering, requested the USDA Forest Products Laboratory (FPL) to investigate new and better ways to build and maintain bridges. The FPL, in turn, is cooperating with the University of Wisconsin (UW) to help investigate new construction concepts.

This paper will outline current ideas and research aimed at developing new timber bridge construction and design methodologies. Many of the new ideas cited herein may prove unworkable for one reason or another, but hopefully some will emerge as feasible. These concepts all work in theory, although some depend on techniques or assumed behaviors that are not recognized in the current national design codes. The logical sequence will be to screen those ideas that appear promising, verify theoretical designs experimentally, and, finally, build prototype systems to monitor in-situ performance. Actual acceptance and use of workable concepts by bridge designers will be contingent on the definition of design criteria and recognition by AASHTO.

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CURRENT BRIDGE SYSTEMS

Timber bridges have been used throughout the national bridge system, but the majority are used on secondary and rural roads. Timber was the primary construction material for only about 13 percent of U.S. bridges over 20 ft in span, but more than 85 percent of these were located on secondary and rural roads. Substantial federal funding has been made available for bridge replacement. Timber bridges are being used, as seen recently in Wisconsin, to replace deficient bridges on local and county rural roads.

Timber bridges are quite versatile and well suited for a wide range of applications. Glued-laminated construction tremendously extended the capabilities of timber span. The Keystone Wye bridge near Mount Rushmore is a 155-ft arch that supports a 290-ft 3-level interchange. A parallel chord bridge near Randall, Washington, has a main span of 120 ft. The Canadian Forest Service has designed and built king post and queen post bridges for clear spans ranging from 125 to 180 ft. The British Columbia Forest Service has built simple beam bridges using glued-laminated I-beams, which span 150 ft (2).

However, timber bridges are most commonly used in short-span situations. Of nearly 11,000 Forest Service bridges, 40 percent fall in the 20- to 40-ft range, and almost 80 percent are 60 ft or less in span. If these figures are similar to rural and county bridge inventories, it is obvious that the biggest market, in terms of numbers, is for bridges under 60 ft. The concepts discussed in this paper focus on these short-span bridges.

TIMBER BRIDGE TYPES

Timber bridges might be described as falling within four categories--girder, longitudinal deck, parallel chord, and special. Figure 1 shows the basic components and layout that might be found in each of the first three categories. The last category (not shown) would include "special" types that are not widely used.

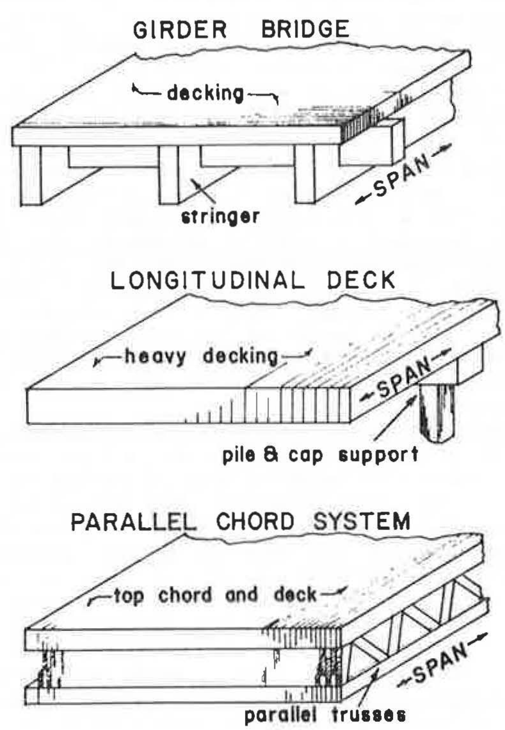

FIGURE 1 Timber bridge types.

Girder Bridges

The most widely used of all timber bridge superstructures is the longitudinal stringer or straight girder bridge. The stringers reach across the entire length of the span and support a transverse deck. Transverse diaphragms may be used to brace the main stringers. Glued-laminated construction took over much of this market years ago when high-quality timber in the required sizes became scarce and more expensive.

One unique girder bridge was built in the George Washington National Forest in 1977 using Press-Lam, a thick form of laminated veneer lumber (LVL). It is produced by parallel-laminating thick sheets of veneer to produce structural-sized members. The process was developed to improve resource recovery and to provide a means of producing large members from small trees.

LVL members contain butt joints and lathe checks, which must be considered in design. However, because wood defects are randomly dispersed in the laminating process, bending stress can be increased and stiffness is more uniform. Also, the lathe checks provide paths for improving penetration and retention of oil-based preservatives. The Virginia Department of Highways and Transportation has monitored the performance of this demonstration bridge and has prepared progress reports (3).

The traditional deck system used with girder bridges was a nail-laminated assembly of nominal 2-in. dimension lumber placed transverse to the stringers with the wide face in the vertical direction. Installation consisted of through-nailing the laminations and toe-nailing to the stringers. Alternate wetting and drying plus impact loading tends to loosen the fasteners and degrades performance. Also, glued-laminated stringers are further apart, thus placing added distress on the deck system. This led to the development of the glued-laminated deck system, which provided a stiff and durable deck and also protected the girders from the elements. Weyerhaeuser Company in Tacoma, Washington, designed a method of attaching the glued-laminated deck panels to the stringers. The system uses angular clips fastened in place in the field.

The FPL developed a process for designing transverse decks based on orthotropic plate theory (4,5). Continuity between individual panels was provided by steel dowel connectors that could effectively transfer both moment and shear across the joint between adjacent panels. Although the system performs extremely well, its use is not nearly as widespread as it could be. Experimental bridges and those erected by experienced crews went together very well in the field. However, construction problems were often encountered in the field with production bridges. These problems appear to be related to the experience of the erection crew and preciseness of drilling dowel holes by the fabricator. The lesson to be learned here is that requiring tight tolerances and high precision tends to deter wide acceptability of a particular system.

Longitudinal Deck Bridges

The longitudinal deck is probably the simplest form of bridge in existence, being a simple slab spanning between supports. This single component system is easy to fabricate and install. The low profile is an added advantage in that it provides maximum clearance during times of high water. There are currently three forms of longitudinal deck bridges: nail-laminated, glued-laminated and panelized (spiked).

The secret to achieving optimum performance of any longitudinal deck bridge is to distribute the load transversely to increase the effective width of deck elements resisting the applied forces. Ideally, the deck should approach full plate action. AASHTO currently recognizes transverse distribution afforded by mechanical fasteners (spikes) between laminations or glued joints, and the effective width is taken as the wheel width plus two times the deck thickness. Additional methods of achieving transverse distribution of loads are distributor beams, transverse prestressing, and dowelled connections between members.

From a practical standpoint, the distributor beam may be the simplest means of achieving transverse load distribution and would be the easiest to fabricate and install on site. A distributor beam is a transverse structural member secured to the bottom of a deck to increase the transverse stiffness. Iowa State University recently concluded a research study for the American Institute of Timber Construction (AITC) on distributor beams for glued-laminated panels (6), which defined distribution factors to be used in design. A new design procedure has been proposed to AASHTO based on this study and should appear in the next edition of the code.

A system of compressing wood perpendicular to the laminations to induce high interlaminar friction was developed in 1976 by the Ontario Ministry of Transportation (7,8). The concept was originally intended as a method for rehabilitating existing nail-laminated decks, but has been extended to new bridge decks as well. To date, some 15 bridges have been built or rehabilitated with the Ontario system. High-strength steel rods are positioned perpendicular to the span direction and tensioned against steel bearing surfaces along the two outside edges of the bridge. The system depends on a minimum stress level in the wood to produce plate action, and displays good load distribution characteristics. Design procedures are contained in the latest Ontario Highway Bridge Design Code (9).

One longitudinal deck bridge was built in the Hiawatha National Forest using glued-laminated panels and short dowels similar to the transverse glued-laminated deck system. Dowels were positioned at 1-ft intervals for the full 38-ft length of the panels and each had to be aligned before the panels could be pulled together. Although the system went together well and is performing satisfactorily, construction is too complicated to be attractive.

Special Bridges

Numerous special bridge systems such as arches, king post and queen post trusses, and long-span parallel chord systems are in use. They are designed for a particular location and require special materials and construction procedures. One new, special bridge that should be mentioned is the prefabricated modular type of a specific design intended for multi-use situations. The United Nations Industrial Development Organization (UNIDO) developed a prefabricated wooden bridge system for use in developing countries (10, 11). The concept was first introduced in Kenya and proved highly successful. The Timber Research and Development Association (TRADA) was awarded a contract by UNIDO in 1981 to extend this technology for use in Latin America, with bridges recently completed in Honduras.

The system uses standard triangular panels that are approximately 10-ft long and 5-ft deep (see Figure 2). The panel is the basis of the prefabricated module and several are joined in line to make trusses. Bridges are designed this way for spans approaching 100 ft with 40-ton loads, but an 80-ft length would be more practical for AASHTO H20 loading. The segments are joined with matching steel end plates at the top chords and steel tension bars or wood members for the lower chords. This modular bridge can be erected without lifting cranes or highly skilled labor. Although costs have not been firmly established, TRADA claims that material and transport costs are lower than for prefabricated steel or concrete. A particular advantage of this system is that all the modules are alike and can therefore be disassembled and reused for relocatable bridges.

NEW IDEAS IN BRIDGE CONSTRUCTION

The bridges mentioned previously are already in widespread use and are competitive with concrete and steel bridges in short-span applications. The new ideas presented in this section are intended to enhance performance, reduce cost, and simplify construction. Many of the innovations will also make use of small size and short-length materials in various species, taking advantage of regional timber supplies and cost reductions.

The concepts discussed in this paper focus on short span bridges. Some goals must be set for these "new ideas" on timber bridges. To be practical, the concepts should meet the following criteria:

• A bridge must be cost effective. Both initial construction and life-cycle costs must be competitive with current designs and other alternative materials. The added cost to gain greater durability and life service must be included when selecting the initial design.

In other words, the anticipated life of the bridge must be considered. Some bridges, particularly in the Forest Service, are temporary and will be taken out of service following a particular operation. One option in such an instance would be to build the minimum-cost structure that will safely perform for a short time. For example, native log stringer bridges are often used for temporary structures on logging roads. A second option would be to design a more expensive relocatable bridge in anticipation of using it in the future at another location.

• It must be simple to fabricate, transport, and erect. The market for short-span bridges is scattered geographically and could best be served by local firms. Thus, to be attractive, the bridge system should be simple to fabricate and one that does not require large capital investments or special equipment. The base material for bridge elements and ancillary components should be easily procurable from nearby sources.

Transportation and erection considerations should recognize several factors. The bridge site is often in a remote area with difficult access. The contract value of a job may not support the cost of heavy lifting equipment, or other special apparatus required for some sophisticated systems. Temporary shoring and falsework should be avoided wherever possible. Finally, because bridges are so widely dispersed geographically, the work force may have had little or no experience in timber bridge construction. These smaller jobs simply do not justify sending a trained crew around the country. The erection process should be at a level consistent with average journeyman skills employing common tools of construction.

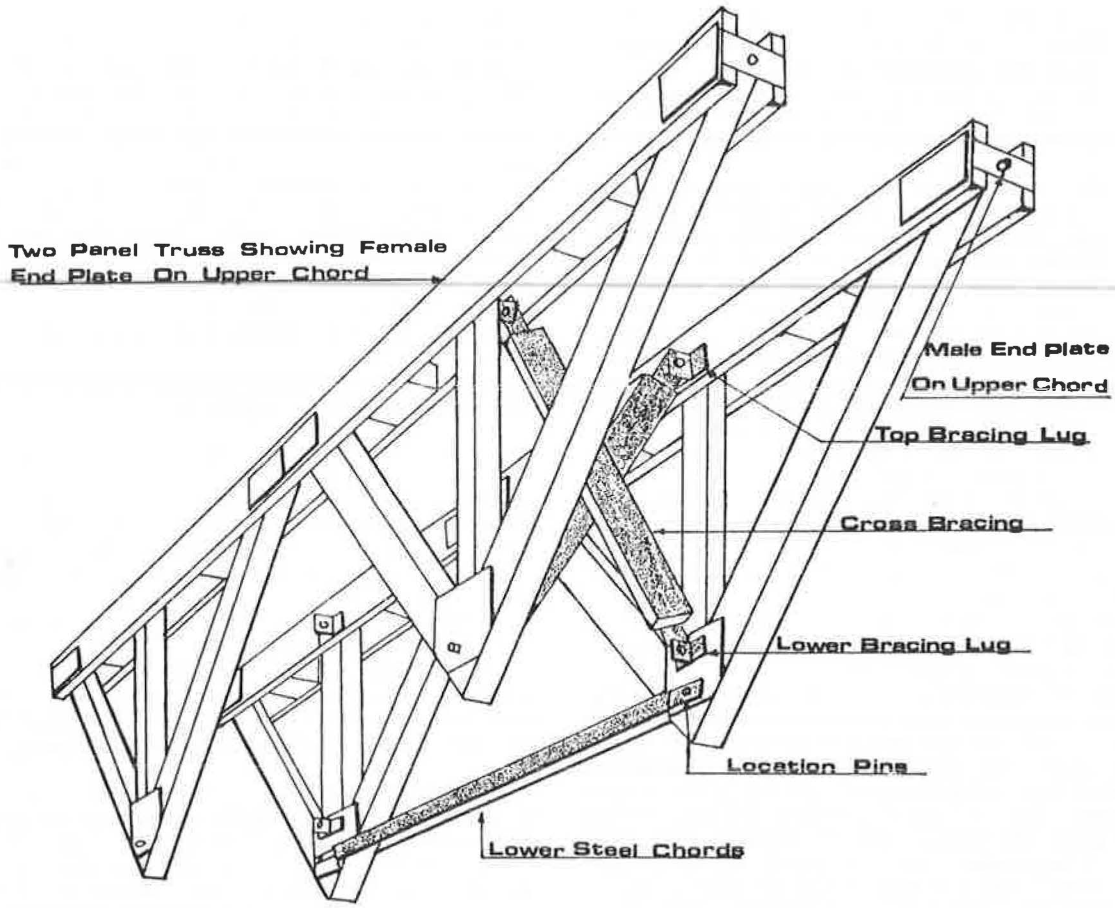

FIGURE 2 UNIDO bridge system: four triangular panels shown with connecting elements.

Girder Bridges

The glued-laminated industry, under the lead of the AITC, has had an active and effective research program for many years. The AITC is continually seeking new ways to improve the performance of glued-laminated systems and this type of bridge construction is in good hands.

Glued-laminated decks with steel dowel connectors on girder bridges might be more attractive if erection could be simplified. Fabrication tolerances could be relaxed if some type of a plastic filler material could be developed, which would be used to fill one of each pair of matching dowel holes. The filler would allow oversized holes to be drilled so that field alignment problems would be reduced.

Longitudinal Deck Bridges

The simplest form of bridge is the longitudinal deck bridge. True "deck" behavior can only be attained when plate behavior exists, and requires continuity between the individual elements comprising the deck. Improved deck performance can be obtained by developing new connection techniques that will spread resistance to an applied load transversely across the deck. Three possible ways to spread the load include distributor beams, transverse stressing of the deck, and dowels. The relative effectiveness of the three options is not yet known, but the upper limit in performance would be total plate action.

Distributor Beams

The use of distributor beams, as noted, has been investigated when used with glued-laminated panels. Wheeler Bridge Company has been using distributor beams with their spiked, panelized, longitudinal deck system for years because they know it improves performance. Yet, they have been unable to take advantage of this factor in their designs because effects of the distributor beams are not yet recognized by the design codes.

With glued-laminated systems, the distributor beam transfers loads across the joints between relatively stiff panels that may be 4 ft or more apart. Thus, the forces are highly concentrated at the joints as the beam attempts to enforce a continuous deformation profile across joints between panels. For a longitudinal deck bridge constructed of dimension lumber or individual timbers, the transverse deflection profile is more continuous and uniform than in glued-laminated panels. Distributor beams could more effectively spread uniform transverse deflection across these bridges and might prove to be more efficient than with glued-laminated panels. Part of the current UW/FPL research program is to test the effectiveness of distributor beams with nail-laminated decks and to devise an analytical model.

Stressed Wood Decks

Transverse stressing of longitudinal decks to create friction connections between laminae was developed by the Ontario Ministry of Transportation and Communication and is included in the Ontario Highway Bridge Design Code (9) but is not yet recognized by AASHTO. Most of the 15 existing bridges using this system are multispan longitudinal decks supported by pile caps or floor beams. The stressing technique requires special proprietary equipment and material that might discourage its use for small jobs in remote areas.

Research is currently underway at the UW Structures Laboratory to examine use of the stressed wood system over longer spans (48 ft) with the deck assembled of random short timbers that could be of local wood species. Techniques of hand-stressing the rods are being examined in an attempt to simplify the stressing process and limit the need for extensive equipment. Orthotropic finite element analytic procedures are being used to achieve correlation with the test results.

Dowelled Systems

Dowel rods extending across the full with of the bridge through prebored holes can distribute forces by "beam-on-elastic-foundation" action. The same plate theory developed for deck panels can be extended to a different range of aspect ratios (e.g., span-to-width ratios) to predict the structural response of longitudinal deck with dowels. For longitudinal decks, the normal deflections would be many folds greater than the short-span deck panels. Hence, the tolerance needed to achieve full dowel action could be relaxed considerably compared to that in short transverse deck panels. Also, the individual lamination, being but 2-in. nominal lumber, could be flexed a little in the field to accommodate a small amount of vertical misalignment.

A perceived advantage of the dowelled system over the Ontario system is that it would not require any special equipment or materials to construct. Also, the concerns over loss of stress due to relaxation or changes in moisture content would be alleviated. This new idea is being investigated under the current UW/FPL research study.

Parallel Chord Bridges

A new idea for short span bridges is borrowed from recent technology developed for light-frame construction. The parallel chord bridges would be made up of trusses fabricated primarily from dimension lumber rather than glued-laminated or timber. The adjacent trusses would be in contact much like the elements in a longitudinal deck. The resulting system forms a wooden box girder extending the length of the span and acting similarly to concrete box-girder bridges.

The parallel chord bridges may relieve the primary limitation of simple longitudinal decks--the need for deep solid material to achieve sufficient stiffness in moderate spans. At first glance, the parallel chord concept using adjacent trusses might appear to be material-intensive, but it may not be. In a typical 20-ft girder-deck bridge, over 70 percent of the wood is in the deck. The amount of wood in the deck exceeds that in the stringers up to spans of about 50 ft. Also, the quantity of material alone does not control the total cost. The cost per unit of material, and ease of transportation and erection must also be considered. With trusses, shorter pieces can be utilized and lower quality lumber can be selectively used for some members. Individual trusses can be fabricated full length and are self supporting for erection. Also, individual trusses are relatively light for easy handling on site.

Three types of parallel chord bridges are being studied by UW/FPL: plane trusses, multileaf trusses, and composite beams (see Figure 3). The first phase is to evaluate the span capabilities of different configurations based on direct loads, and the second will look for ways to enhance performance through various means of transverse load distributions.

PARALLEL CHORD TRUSSES

TYPE		POTENTIAL MAX. SPAN	
Multi-Leaf Truss			
24" depth	(61cm)	25ft	(7.6m)
30"	(76cm)	32ft	(9.8m)
36"	(91cm)	36ft	(11.0m)
Metal Plate Connections			
24"	(61cm) deep	27ft	(8.2m)
30"	(76cm)	35ft	(10.7m)
36"	(91cm)	40ft	(12.2m)
Composite Beam			
24"	(61cm) deep	23ft	(7.0m)
30"	(76cm)	30ft	(9.1m)
36"	(91cm)	35ft	(10.7m)

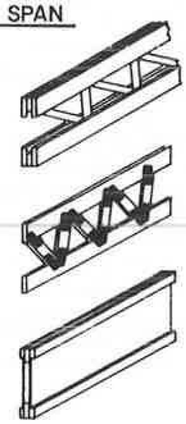

FIGURE 3 Parallel chord bridges: types and potential spans.

Plane Trusses

Plane trusses are most commonly fabricated using metal truss plates. This type of truss is shown under the "metal plate connection" type in Figure 3. The truss plate industry comprises some 2,000 fabrication plants strategically located throughout the United States, so bridge contracts using this type of truss anywhere in the country could probably be served by local fabricators.

Structurally, plane trusses can readily span 40 ft under H-20 loads without the benefit of transverse load distribution. The major compromise is in reducing the truss depth to achieve a low profile versus using a depth that provides optimum performance. With shallow trusses, the number of web elements becomes excessive, which adds greatly to the fabrication costs.

Alpine Engineered Products has patented a new truss configuration, the J-24 series, which may be particularly well suited for bridges (see Figure 4).

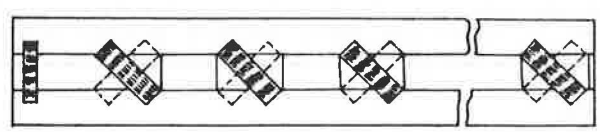

FIGURE 4 Patented J-24 series of Alpine Engineered Products (plates drawn with dashed lines are on the back side of the truss).

The J-24 truss is a kind of hybrid, a combination of a metal plate truss and a Vierendeel truss. It has no diagonal wood web members--only spacer blocks, which are positioned between the flanges. Two metal truss plates, one on each side, are pressed in diagonally opposite each other. This concept looks extremely simple to fabricate, and might provide an economic way to produce bridge trusses.

However, some unanswered questions remain on the use of metal plates for exposed structures. First, it is not known how they will perform under cyclic changes in moisture content. Although the plates will be tightly nested between adjacent trusses, some joint slip might occur. Second, treatment would most likely be with an oil-based preservative and there does not appear to be any information on how metal plates perform in oil preservative-treated wood. The current UW/FPL study will investigate plate performance.

The Ontario Ministry of Transportation and Communications built a prototype bridge in 1981 using metal truss plates. It is a slant-leg rigid frame (12) rather than a simple truss, and future performance studies could provide good data on transverse load distribution and suitability of using metal truss plates in treated wood under cyclic loading and exposed conditions. It is critical that questions on the performance of metal plates under adverse conditions be fully resolved before this idea can be extended to operational bridges.

Multileaf Trusses

Multileaf trusses provide an alternative for parallel chord bridges that can be easily fabricated in a small shop without special equipment. This concept uses three pieces of dimension lumber in both top and bottom flanges. The two outside pieces would be 2 in. deeper than the center one in both flanges, as shown in Figure 3. The ends of the diagonal web members would butt against the center flange piece, and the two side pieces would straddle it providing a clamping action. Webs would be secured by nail-gluing each end. Theoretically, this type of truss could span to about 40 ft without the benefit of transverse load distribution. The critical design consideration appears to be the end restraint on tension web members. The performance of this joint will need to be confirmed experimentally.

Composite Beams

The advent of Foundation Grade Plywood provides a new opportunity for the design of composite beams. Foundation Grade Plywood has a proven record of durability, and should perform well for bridges. Standard I-beams, with single webs of plywood or oriented strand board, are rapidly gaining in popularity for residential and light commercial building. These composite I-beams are cost effective, lightweight, and can be fabricated in any desired length. However, preliminary analysis of composite beams indicates that the web is critical in shear under truck wheel loads. A single web using standard plywood thicknesses does not appear to provide the required shear area, but a multileaf configuration with two webs might.

As before, it is envisioned that the two flanges would be built up of three pieces of dimension lumber, but, in this case, the center piece would be 2-in. deeper than the other (see Figure 3). The ledge provided by the center piece would provide the bonding surfaces for attaching plywood webs on both sides. The total width of each composite beam would be controlled by the thickness of lumber as the plywood would be recessed below the flanges. Thus, it would be possible to reduce the thickness of plywood away from the high shear areas.

Engineering calculations show that the three preceding concepts could carry highway trucks over reasonable spans without the benefit of transverse load distribution. Only the members within the wheel width were considered in resisting load. However, the open space in the webs provides easy access to install load distribution devices. If the distributor beam and stressing system are effective for longitudinal deck bridges, they should also work with parallel chord bridges. This would be a logical extension for research to parallel chord systems.

The current UW/FPL research is analytically studying the three parallel chord systems using truss analysis programs to evaluate their span capabilities and connection requirements. Experimental investigation of metal splice plate joints in preservative-treated wood is planned.

SUMMARY

Several new ideas have been presented for building more efficient and low-cost timber bridges to satisfy current needs in replacement of deficient bridges and future needs for new bridges. Some of these ideas may prove to be viable and competitive. Many of these innovations are being examined cooperatively by the University of Wisconsin and the Forest Products Laboratory of the Forest Service, U.S. Department of Agriculture.

The present UW/FPL research is experimentally investigating the effectiveness of distributor beams, transverse stressing, and metal dowels in improving the transverse load distribution of longitudinal decks. Test results are being used to verify computer-aided analytical methods for predicting behavior. Preliminary analytical studies are underway to evaluate the efficiency of various parallel chord systems. Each of the proposed new ideas will eventually be studied.

The concepts will first be analyzed to estimate their behavior and theoretical capacities. Then, the more promising ideas will be tested experimentally to confirm the analytic theory and to prepare design models. Finally, prototype bridges need to be built using the most practical concepts to obtain field data on actual performance under service loads with environmental effects.

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Performance and Rehabilitation of Timber Bridges

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ABSTRACT

Eighteen timber bridges were inspected to assess their long-term performance. In general, they were in excellent structural condition with glued-laminated decks performing better than nail-laminated decks. Extensive moisture content readings indicated that wet-use stresses should be used when designing bridge decks, regardless of deck type or treatment. Dry-use stresses are appropriate for the stringers. A comprehensive program, including new technologies and demonstration projects, must be developed to address the repair and rehabilitation of older nailed-laminated decks.

Many recent developments have increased the interest in timber bridges, such as new materials and manufacturing methods, improvements in preservative treatment, a systems approach to bridge engineering, alarm over needed bridge replacement and rehabilitation, and improved technology (1,2). A number of these developments reflect the advancement in knowledge of the general behavior of wood as a structural material (3). The importance of these developments has been underscored in comprehensive state-of-the-art reports and technical presentations and publications (4-7). A recent workshop helped identify research needs that are pertinent to timber bridge engineering (8).

The Forest Service (U.S. Department of Agriculture) is keenly interested in this engineering area because it maintains over 10,000 road bridges and adds 100-250 bridges to the system annually. Wood is the major construction material in over one-half the existing and new bridges. In the late 1960s and early 1970s, the Forest Service sponsored the construction of timber bridges containing novel features that would be expected to improve performance. These bridges were built in various national forests in seven states across the northern United States. They were primarily constructed with transverse glued-laminated (glulam) panel decks and a variety of interpanel connections. Some bridges had nail-laminated (nail-lam) decks for comparative purposes. Also, different types of members, construction, and materials were used in the remainder of the superstructure and substructure. Some of the new features were, at the time, considered to be experimental.

Although these bridges had received routine inspections over the years, little had been done to specifically monitor the structural performance and condition of the experimental features. The relative performance of glulam decking and traditional nail-lam decks is of particular interest. Although nail-lam construction is still used at some sites and numerous nail-lam decks are still in service in Forest Service timber bridges, the glulam panel deck is now the preferred material.

There are thousands of older, poorly maintained, timber bridges in the United States with nail-lam decks that have deteriorated badly. At some time, a decision must be made in each case to abandon the

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structure, reduce the load limit, or repair the deck. Options for repair of nail-lam decks include total replacement or reinforcement.

This paper reports on research studies aimed at developing criteria to improve the performance of timber bridges. The objectives of the work were to (a) determine the in-place performance of timber bridges, particularly those made with glued-laminated panel decks and dowel connectors, (b) determine patterns of moisture content in the deck and stringer systems to assess the merits of dry-use versus wet-use design stresses, and (c) determine methods of extending the service life of existing timber bridges with nailed-laminated decks.

DOWEL-CONNECTED GLULAM DECKS

Glulam decking systems were developed in the 1970s to ease construction and provide longer life than nail-lam decking. Typical glulam deck panels are 4-ft wide and composed of vertically laminated, dimension lumber that spans continuously across the supporting stringers. McCutcheon and Tuomi investigated several alternative deck interconnection concepts, including steel dowels, tongue-and-groove joints, and splines, to determine the most effective vertical shear transfer mechanism (9). They determined that steel dowels were the most effective for achieving load transfer and continuity between adjacent panels. This work produced design procedures for glulam decks and steel dowel connectors that were incorporated into AASHTO bridge specifications and led to the development of standard plans and details for glulam bridges by the American Institute of Timber Construction (AITC) (10).

Proper erection procedures for dowel-connected glulam decks are detailed by Tuomi (11). However, construction problems such as tightness, misalignment, or binding of dowels have discouraged more widespread use of this system. These problems appear to be related to the inexperience of the erection crew and imprecision in the fabricator's drilling of deck panels. Gutkowski investigated the influence of the size of the dowel lead hole; his results suggested that the usual lead-hole oversize of 1/32 in. might be relaxed to 1/16 in. (12).

DRY- AND WET-USE STRESSES

Until recently, the AITC recommended dry-use stresses for deck panels if heavy oil and creosote preservative treatments were specified; otherwise wet-use stresses should apply (10). Dry-use stresses were and are recommended for stringers, although this practice has been questioned by some bridge engineers, particularly when site conditions and climate suggest high moisture content. Industry standards define the dividing line between wet- and dry-use stresses as 16 percent moisture content for glulam and 19 percent for sawed timbers. In practice, 20-percent moisture content is commonly considered the level at which potential decay is a concern, but this limit includes a considerable margin of safety against damage caused by fungal growth. Serious decay occurs only when the moisture content exceeds the fiber saturation point, which is approximately 30 percent for species commonly used in timber bridge members. Thus, field measurement of the in-service moisture content of bridge stringers was a prerequisite for assessing the merit of dry-use design stresses. As will be seen, AITC changed one of its recommendations as a result of this study.

REHABILITATION OF NAIL-LAM DECKS

Numerous short-span timber bridges are in need of deck rehabilitation. The majority of these were built with preservative-treated lumber that was nail-laminated together to provide a continuous deck. Such decks can be constructed easily and rapidly at low initial cost. After several years, the mechanical fasteners in these decks tend to loosen because of repetitive loadings and moisture cycling and the decks often become unserviceable. Methods are needed to extend the service lives of these bridges.

As previously noted, there are several options for bridges with nail-laminated decks that have deteriorated badly: abandonment, reposting with lower load limits, total deck replacement, or deck reinforcement. Because the bridges are needed, abandonment has seldom been an option. The most common response to poor bridge decks has been to lower the load limit.

Rehabilitation or replacement of nail-lam decks rarely occurs on Forest Service timber bridges. Lack of funds places maintenance at low priority and rehabilitation even lower. Glulam panels are usually used in cases of "deck only" replacement. Complete bridge replacements are typically made of steel or concrete girders with concrete deck. Decayed laminations, excessive maintenance needs, loss of tightness, poor load distribution, and asphalt deterioration are major reasons for replacement, the first two being the most compelling factors.

Reinforcement of timber bridges with steel is also uncommon. Some exploratory attempts were conducted in the Forest Service from 1965 to 1975. Several longitudinal decks were transversely reinforced with ordinary A36 steel rods. This proved unsuccessful primarily because the effective post-tensioning force could not be maintained. Recent field inspection of two sites indicated the physical condition of the reinforcing to be excellent, but the chip seal surfaces had deteriorated badly.

PROCEDURES

To address the first two objectives previously stated (determining structural performance and moisture patterns), eighteen bridges were inspected during the summer of 1983. The bridges, which were distributed across the northern United States (Figure 1), included a variety of types and sizes (Figure 2) and were from 7 to 17 years old. The determination of methods for rehabilitating nail-lam decks, was addressed through a literature survey and extensive discussions with bridge engineers throughout the Forest Service.

Inspection Procedure

Structural performance of the deck and connectors was determined by observing the condition of the wearing surface, especially cracking in the surface and gaps between deck panels. The inspectors also looked for signs of swelling in the deck and checked for tightness and fit of the connections. Drainage conditions on the bridge decks (i.e., whether water can drain freely from the road surface or is trapped by blocked drainage paths, potholes, etc.) were noted. Also, the underside of each bridge was examined for signs of water penetration through the deck. The condition of each bridge was thoroughly documented with photographs.

Moisture Content Readings

Extensive measurement of moisture content was a key feature of the inspections. A trio of readings was taken at each of over 600 locations on the 18

FIGURE 1 Eighteen bridges were inspected across the northern United States.

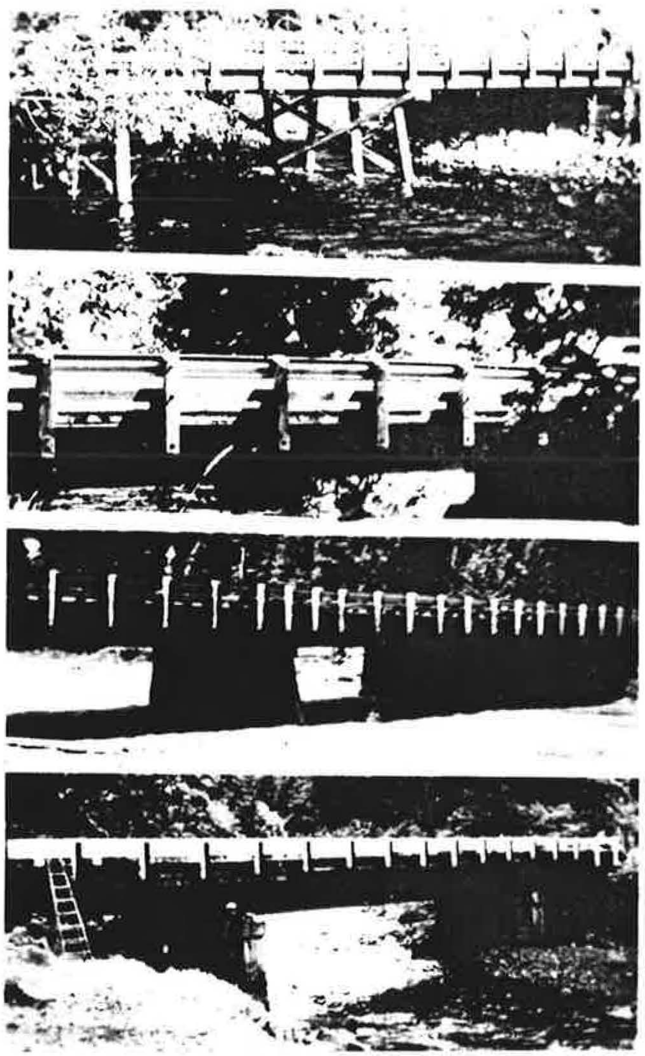

FIGURE 2 Study included a variety of bridge types and spans.

bridges. On average, this represents 100 readings per bridge.

An electrical resistance-type moisture meter was used to take readings at penetrations of 1/2 in., 1 in., and 2 in. With few exceptions, the moisture reading at the 2-in. penetration was the highest of the three readings. Therefore, readings are reported only at that penetration.

All readings were taken during the summer months. Although some variation in moisture content exists during the year, large treated timbers change little from season to season.

Deck Rehabilitation

The feasibility of rehabilitating nail-lam decks was investigated by interviewing Forest Service bridge engineers and reviewing technical literature. The conversations with Forest Service engineers revealed the extent of the rehabilitation needs together with current practices and constraints. The literature survey included an evaluation of technology that may be applicable to rehabilitating transverse nail-lam decks.

RESULTS OF VISUAL INSPECTIONS

Structural Integrity

The inspection of the 18 bridges revealed few instances of significant structural problems. The bridge superstructures were commonly found to be in excellent condition. In most cases, the glulam stringers and bottom sides of deck panels appeared like new. Occasionally, some leaching had discolored the stringers at the abutment ends. Glulam deck panels were tightly butted. Nail-lam decks were providing a smooth road surface (except at abutments) even when moisture content was high and decayed boards were present. Although future physical deterioration of the nail-lam decks appeared inevitable, major deck replacements would likely not be warranted for many years. Glulam decks generally appeared drier and in better physical condition with essentially no evidence of imminent deterioration. Typically, backing planks in abutment walls were tightly mated. At three sites, gaps up to 3/4-in. wide existed but all walls were structurally sound.

Asphaltic Wearing Surfaces

Twelve of the eighteen bridges had asphaltic or chip seal wearing surfaces. Five of these exhibited some degree of transverse cracking in their wearing surfaces. In all cases, the cracks had neither propagated laterally nor seriously affected roadway smoothness. Potholes, as distinguished from cracks, were noted on only two bridges.

Drainage and Appearance

With a few exceptions, drainage at the curbs and abutment corners was unhindered regardless of deck surfacing method. Two bridges had major blockage at the curbing caused by heavy accumulation of gravel resulting from blading the roadway surface and a third had similar blockage on one side of the bridge because of collected roadway debris and gravel. Two others had ponding at the abutments and one of these had scoured embankments at all four bridge corners. Gravel roadways and approaches were generally well graded and maintained. However, buildup of moisture and mud at the abutments for nail-lam decks was a general concern. Such buildup greatly contributes to moisture penetration through the deck. Bleeding of creosote was evident at five bridge sites. Although this was of no structural consequence to the members, it detracted from their otherwise good appearance.

RESULTS OF MOISTURE INVESTIGATION

Decks

Moisture content varied greatly among individual laminations of nail-lam deck, with many boards above the fiber saturation level. The moisture content and deterioration in nail-lam decks were consistently greatest adjacent to the abutment. Wear caused by the impact of entering vehicles and subsequent ponding of water appear to be the primary causes. Where running planks are present, water is often channeled to the abutment zone. Adjacent to abutments, nail-lam decks averaged 26 percent moisture content with 30 percent of the readings above 30 percent and away from abutments they averaged 18 percent with 7 percent of the readings above 30 percent.

Passage of moisture through nail-lam decks was greater than through glulam decks. Both the moisture data (stringer top versus bottom laminations) and visual observations attest to this. There was virtually no evidence of saturated boards in the glulam decks, and tight mating of deck panels generally prevented moisture from penetrating to the stringers at the deck joints.

In the abutment area, above exterior stringers and in the overhanging portions, nail-lam decks typically had higher moisture content than glulam decks. Between interior stringers, nail-lam decks were drier than glulam. The overall moisture content for nail-lam decks was 21 percent with 15 percent of the readings above 30 percent and for glulam decks, the average was 20 percent with only 3 percent of the readings over 30 percent. Visually, the nail-lam decks consistently appeared much wetter in comparison with the glulam decks than was indicated by the measured data. Indeed, the bottom surface of glulam deck often looked dry and new, while the nail-lam looked moist and aged.

Wearing Surfaces

The average moisture contents in nail-lam decks and in the top laminations of stringers were lower below asphaltic wearing surfaces than below running planks (with gravel). This is consistent with the frequent occurrence of saturated running planks and the field observation that the space between planks is often a reservoir for water and the buildup of wet gravel.

Stringers

Whether solid-sawed or glulam, stringers typically appeared sound and dry. There was little difference in appearance between the two types. The moisture readings support this finding. Combining all data, glulam stringers averaged 17 and 14 percent moisture content in the top and bottom surfaces, respectively. Only 1 of 64 data points exceeded 30 percent. Treated solid-sawed stringers averaged 14 and 15.5 percent in the top and bottom zones, respectively; the highest reading was 20 percent. Thus, preservative-treated solid-sawed and glulam stringers performed about the same with regard to moisture.

Substructure

Posts typically appeared solid and sound. In contrast, the piles often looked saturated. However, above the stream level, piles and posts were usually not saturated. Only 4 percent of the moisture readings exceeded 30 percent. None of the abutment walls showed visual evidence of deterioration, and although backing planks exhibited high moisture content on the exterior face, they were usually not saturated. Creosote-treated glulam pile caps were markedly drier than creosote-treated solid-sawed pile caps.

Preservative Treatments

When two bridges having identical component materials but different preservative treatments were compared for moisture, creosote-treated deck and stringers exhibited lower average moisture contents but greater frequency of readings above the fiber saturation level than those components treated with waterborne salts. No distinction could be made between the moisture contents of decks and stringers treated with creosote versus those treated with pentachlorophenol in oil. Average moisture readings and ranges were nearly identical for the two.

Miscellaneous

Some components other than the decks and stringers were expected to be saturated. However, only 8 of 285 moisture readings in these other components exceeded 30 percent. Although some high moisture readings were obtained, posts, wheel guards, and bridging were typically the driest components in any given bridge. Seasoning checks in the heartwood of solid timber wheel guards were frequent. Curbing blocks were typically the components with the highest moisture content. A pile-up of wet soil resulting from blocked drainage was a contributing factor. Rails were measurably wetter than the supporting posts. Seasoning checks and broken end pieces were common and are likely to have left these components particularly susceptible to moisture penetration. Rusted metal parts were uncommon. Moisture readings adjacent to metal fastenings were not unusually high compared to other locations.

RESULTS OF REHABILITATION INVESTIGATION

The rehabilitation investigation revealed a promising technology and developed recommendations for carrying out a comprehensive program.

An effective method for post-tensioning old nail-lam decks and prestressing new decks has been developed, tested, and embodied in design code criteria in Ontario, Canada (13). The method has been implemented on several longitudinal nail-lam deck bridges (for which it was specifically developed) but has recently also been applied to a transverse deck. The fundamental concept is promising; however, there are concerns about loss of pretension, need for periodic retightening, effects of humidity, and initial costs.

A long-term solution to problems of bridge maintenance, repair, and rehabilitation for the Forest Service, or any organization with a large inventory, will require the following steps:

1. The computerization of the bridge inventory to put statistics on bridge condition in a common format and identify and clarify needs.
2. The organization of workshops to disseminate information to administrators, engineers, and maintenance personnel.
3. The demonstration of new methods and technologies, such as the Ontario prestressing procedure, for display and evaluation purposes (note that the Forest Service is planning several such projects).
4. The development of a long-term program modeled after the Ontario Ministry of Transportation's successful efforts to upgrade timber bridges (14). The

Forest Service is proceeding with such a program that will include field surveys of current conditions, economic studies, experimental and theoretical development of new rehabilitation concepts, and dissemination of the new methods to the appropriate audience.

SUMMARY

Overall, the 18 bridges investigated in the northern United States were in excellent structural condition. Typically, roadway conditions were excellent, providing for smooth passage regardless of surfacing. There was extensive asphalt cracking only where the surface was unusually thin. Evidence of deterioration either from propagation of cracks or presence of potholes was rare. Dowel-connected deck panels were tightly mated and the dowels provided continuity between the individual panels.

Glulam decks generally provided a more effective roof over stringers than nail-lam decks, but both types of deck had high moisture contents (over 20 percent). In contrast, the stringers were relatively dry, with few readings in excess of 20 percent. Because of this finding, AITC now recommends wet-use stresses for all glulam decks, regardless of treatment type, and dry-use stresses for stringers (15).

A long-term solution to the problems of repair and rehabilitation of nail-lam decks will require a coordinated program involving the development of new technologies, the dissemination of information, and the showcasing of new techniques through demonstration projects.

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Repair of Timber Bridge Piling by Posting and Epoxy Grouting

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ABSTRACT

Evaluated in this paper is the repair of timber bridge piles by posting and epoxy grouting. The repair procedure consists of cutting out the damaged pile section and replacing it with a new section. After spiking the new section in place, the joints are epoxy-injected to form a permanent bond. The first phase of the study consisted of laboratory testing of repaired pile sections in both compression and bending. Full axial compression strength could be restored through the repair process; however, the ultimate bending strength could only be restored to typical allowable stress values. Flexural stiffness could also be restored to expected values for undamaged piles. To study the durability and long-term effects of the repair procedure, a field repair was also initiated. On a low-volume bridge located in a Louisiana national forest, three piles were repaired and instrumented. After 2 years, no significant movement or deterioration has been observed.

One of the most common types of bridge support is the timber pile. Piles are found extensively on short span, low-volume bridges and have been widely used over the last 40 years. Nationwide, many of these older bridges have piles that have been seriously weakened by decay or insect attack. Typically, decay occurs at or above the average waterline and is associated with cyclic wetting and drying, while insect attack usually occurs below the waterline. Many of these bridges are in good condition and still functional except for significant damage to a relatively small number of the pile supports. A method of repair for such piles that is both structurally sound and cost effective would significantly extend the life of these bridges.

The purpose of this paper is to evaluate the effectiveness of posting and epoxy grouting for replacing damaged pile sections. A two-phase program of laboratory tests and field repairs was utilized in this investigation.

LITERATURE REVIEW

A survey of the literature reveals relatively little information on the repair and rehabilitation of timber piles. One area of attention relates to defining and quantifying the nature of pile deterioration. Williams and Norton (1) have analyzed the rate of pile deterioration and found it to be exponential in nature. They developed a mathematical model to predict the rate of pile decay. Various authors (2-4) have described the nature of pile attributed to both decay and insect attack. Recommendations on controlling pile deterioration generally relate to chemical treatments of protective coverings (5,6).

The rehabilitation of damaged piles falls into five categories: replacement, augmentation, grouting, jackets, and cutting and posting as summarized in Table 1. Replacement is probably the most widely used procedure. However, it can be quite expensive. Augmentation (7) involves the addition of material and connectors. Such procedures typically consist of bolting splices of steel or wood, attaching a new section of pile to an existing one, or the addition of bracing. One serious drawback is that the connectors used are subject to future deterioration. A less severe approach is to grout the damaged area. Large voids have been successfully grouted with cement (8), while lesser damage can be grouted with epoxy (9). However, little published information is available on strength after repair or on methods for deciding when the damage can be repaired by grouting. A more popular procedure is to reinforce a damaged pile with a jacket (10-12). As shown in Figure 1, the procedure is to pour a reinforced concrete jacket around the damaged pile. The form consists of fiber-reinforced plastic or fabric that can be fitted around the pile in the form of a sleeve. The process can be used to increase the strength of the pile and will prevent further deterioration. However, the pile size is increased and specialized equipment and personnel are required.

The procedure of cutting and posting, which is the object of this investigation, consists of cutting out the damaged section and replacing it with a new piece. Typically, the piece is spiked or bolted to the existing pile and a relatively weak connection is formed. However, the use of epoxy offers the potential for increasing the pile capacity associated with this approach. In order to evaluate the effects of using epoxy injection for bonding the new piece into the existing pile, both a laboratory study and a field pilot project were conducted.

DESCRIPTION OF THE REPAIR PROCEDURE

The basic repair procedure is outlined as follows:

1. Determine the limits of damage by taking borings. Drill bit borings are recommended in which an electric drill is used. The chips are visually examined for soundness. A hand auger or incremental borer can also be used, although such methods are much slower than drilling.
2. Provide temporary shoring if necessary. Repairs can often be made during dry periods when stream beds are low or dry. In such cases, temporary

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TABLE 1 Repair and Rehabilitation Procedures for Timber Piles

Procedure	Applicable Type of Damage	Advantages	Disadvantages	Strength after Repair: Test Results Available	Strength after Repair: Analytical Procedures for Predicting Repair Strength	Reference
Replacement	Any type where piles can be redriven	Complete restoration May increase capacity	Expensive May require superstructure removal	None reported	Standard procedures for pile capacity	–
Grouting						
Cement	Internal decay primarily above water	Inexpensive Ease of application	No criteria for deciding applicability Variability of results	Some conducted but not reported	None	8
Epoxy	Internal decay and exterior splits above water	Inexpensive	Skilled workmen required No criteria for deciding applicability	Reported on wood but not piles	None	9
Pile Augmentation	Decay, splits, and breaks	Inexpensive Applicable under water	Subject to deterioration Increases pile size	None reported	None	7
Jackets	Decay, splits, and breaks	Protect against deterioration May increase capacity Applicable under water	Increases pile size Specialized equipment and personnel required	Some conducted but not reported	None	10-12
Cutting and Posting	Decay, splits, and breaks above water	Ease of application Inexpensive	Flexural strength reduced	Axial and flexural	Criteria given	Author

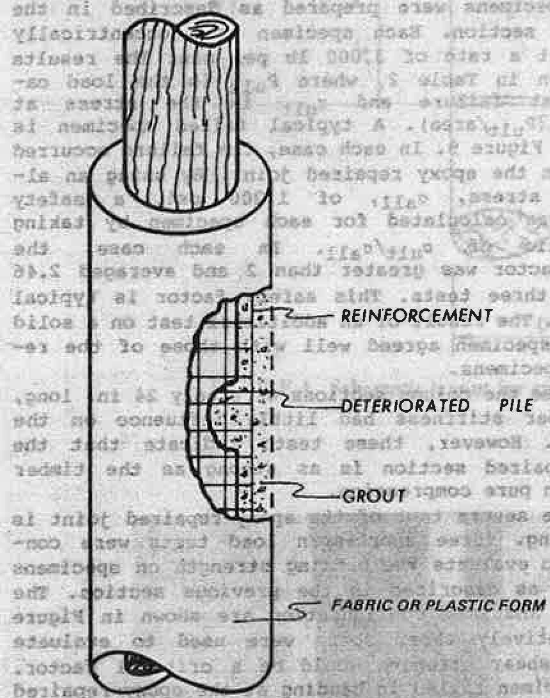

FIGURE 1 Reinforced concrete jacket for pile repair.

supports can be used. Otherwise, the pile cap must be relied on to transmit the dead load to adjacent piles. An obviously deteriorated pile is shown with a jack supporting the cap in Figure 2.

3. Cut the deteriorated section out of the pile. Cuts should be made perpendicular to the longitudinal axis of the pile. If deterioration extends to the pile cap, then the entire upper section should be removed. An example of a pile with a removed section is shown in Figure 3.

4. Prepare the replacement section by cutting a similar diameter section to fit with 1/8- to 1/4-in. clearance at top and bottom. To maintain the spacing, add washers secured by roofing nails to both ends as shown in Figure 4. To prevent epoxy from migrating parallel to the grain in surface checks during injection, cut an epoxy trench around the circumference

FIGURE 2 Seriously deteriorated pile with jack supporting pile cap.

of the pile approximately 9 in. to either side of the joint (Figure 4). For short replacement sections, the trench can be omitted in the replacement section. The trench should be 1/4-in. wide and 3/4- to 1-in. deep. Inspect the base of the trench for deeper checks. If any are found, drill a 1/4-in. hole to the bottom of the check. During the sealing stage, this trench will be filled with epoxy gel to prevent

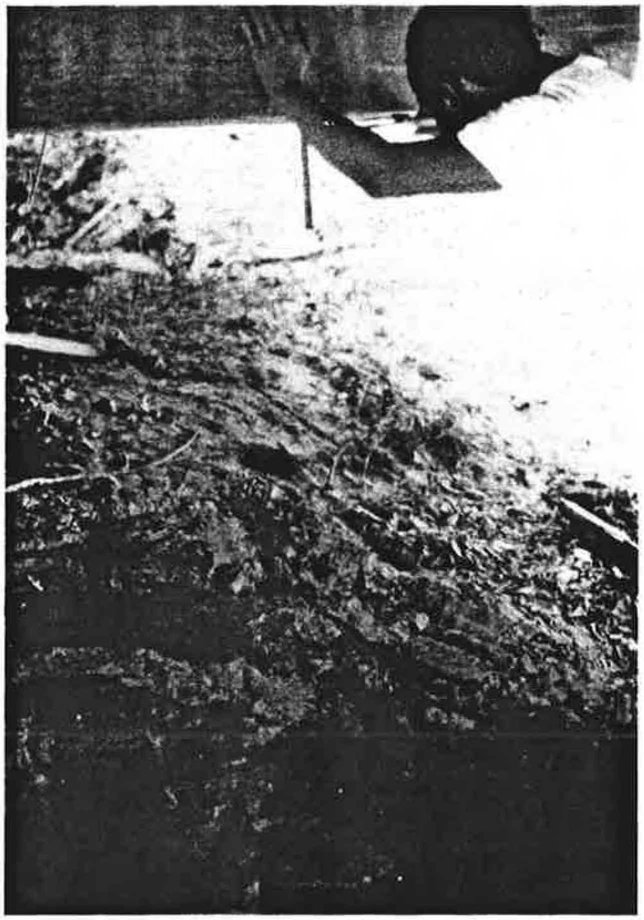

FIGURE 3 Pile stub remaining after removal of section below pile cap.

longitudinal injection epoxy migration much as a slurry trench prevents fluid migration through soil.

5. The replacement section is placed in position and wedged tight with wooden wedges (Figure 4). The wedges maintain a constant opening during the curing process. A section in place is shown in Figure 5.

6. Tie pins are placed at each cut location. Four holes are drilled at approximately 6 in. above each joint with a 14-in. long, 9/16-in. drill bit. The holes are spaced at 90 degrees around the pile and drilled at a 60-degree or larger angle with the horizontal as shown in Figure 4. The steel pins are 3/8-in. square bars twisted to form a spiral of one revolution for each 6 in. of length. The pins are lightly driven until flush with the surface (Figure 6).

7. Set injection and venting ports and seal the joint between epoxy trenches. Sealing in progress is shown in Figure 7. The epoxy trenches and holes in the base of the trenches are filled completely with an epoxy gel of putty-like consistency such as Sika Dur Hi-Mod Gel. Injection and venting ports of 1/4-in. copper tubing are placed at the joint (two minimum) and at the opening for each pin and sealed in place with gel (see Figure 4). Then the entire section between trenches is coated with gel.

8. Leak test the sealed joint by temporarily capping all ports and pressurizing the injection port with air or gas. A soap film brushed over the joint can be used to detect leaks. Repair all leaks before the final step.

9. Pressure inject a low viscosity epoxy such as Sika Dur Hi-Mod LV into one port while using the others as venting ports. (A nozzle pressure of 40 psi is recommended.) As venting ports leak epoxy, they are capped and the injection continues. When all ports are capped, injection should be continued until approximately 20 psi pressure is maintained for at least 5 sec. The procedure is completed by capping off the injection port.

LABORATORY EXPERIMENTAL STUDY

To evaluate the effectiveness of the posting and epoxy grouting repair procedure, two series of tests were conducted. In both test series, cut and posted pile sections were repaired and tested. The epoxy used was Sika Dur Hi-Mod LV, which has been found to be effective in the repair of timber structures (13). The first was a set of axial load tests on a pile segment with a single joint. The pile configuration is shown in Figure 8a. The pile segments were taken from Southern pine telephone poles obtained from South Central Bell. Although smaller than typical bridge piles (8 in. versus 12 in.), the piles are large enough to model typical bridge pile behavior. Three specimens were prepared as described in the previous section. Each specimen was concentrically loaded at a rate of 3,000 lb per min. The results are shown in Table 2, where P_{ult} is the load capacity at failure and σ_{ult} is the stress at failure (P_{ult}/area). A typical failed specimen is shown in Figure 9. In each case, the failure occurred away from the epoxy repaired joint. By using an allowable stress, σ_{all}, of 1,200 psi, a safety factor was calculated for each specimen by taking the ratio of $\sigma_{ult}/\sigma_{all}$. In each case, the safety factor was greater than 2 and averaged 2.46 for the three tests. This safety factor is typical for wood. The result of an additional test on a solid control specimen agreed well with those of the repaired specimens.

Because the column sections were only 24 in. long, the member stiffness had little influence on the strength. However, these tests indicate that the epoxy-repaired section is as strong as the timber itself in pure compression.

A more severe test of the epoxy-repaired joint is in bending. Three short-span load tests were conducted to evaluate the bending strength on specimens repaired as described in the previous section. The specimen and load configuration are shown in Figure 8b. Relatively short spans were used to evaluate whether shear strength would be a critical factor. Each specimen failed in bending at the epoxy-repaired joint. The test results are shown in Table 3, where P_{ult} is the load capacity at failure; σ_{ult} is the extreme fiber bending stress at failure; and M_{ult} is the bending moment at failure. The safety factor based on an allowable extreme fiber bending stress, σ_{all}, of 1,850 psi averaged 0.94 for the three tests and was obtained by taking the ratio of $\sigma_{ult}/\sigma_{all}$ at the centerline. The deviation of each failure stress from the mean was small. This characteristic was expected because the quality control of the epoxy formulation is quite high. A safety factor of at least 1.7 would be desirable for such a material. Based on these tests, the ultimate bending strength of the repaired joints approximately equals the allowable strength of an undamaged pile. The properties of the wood itself are significantly more variable and a normal wood safety factor would be much higher--approximately 2.25.

A failed section is shown in Figure 10. An examination of each of these sections indicated that the extreme fiber tension section failed initially. The

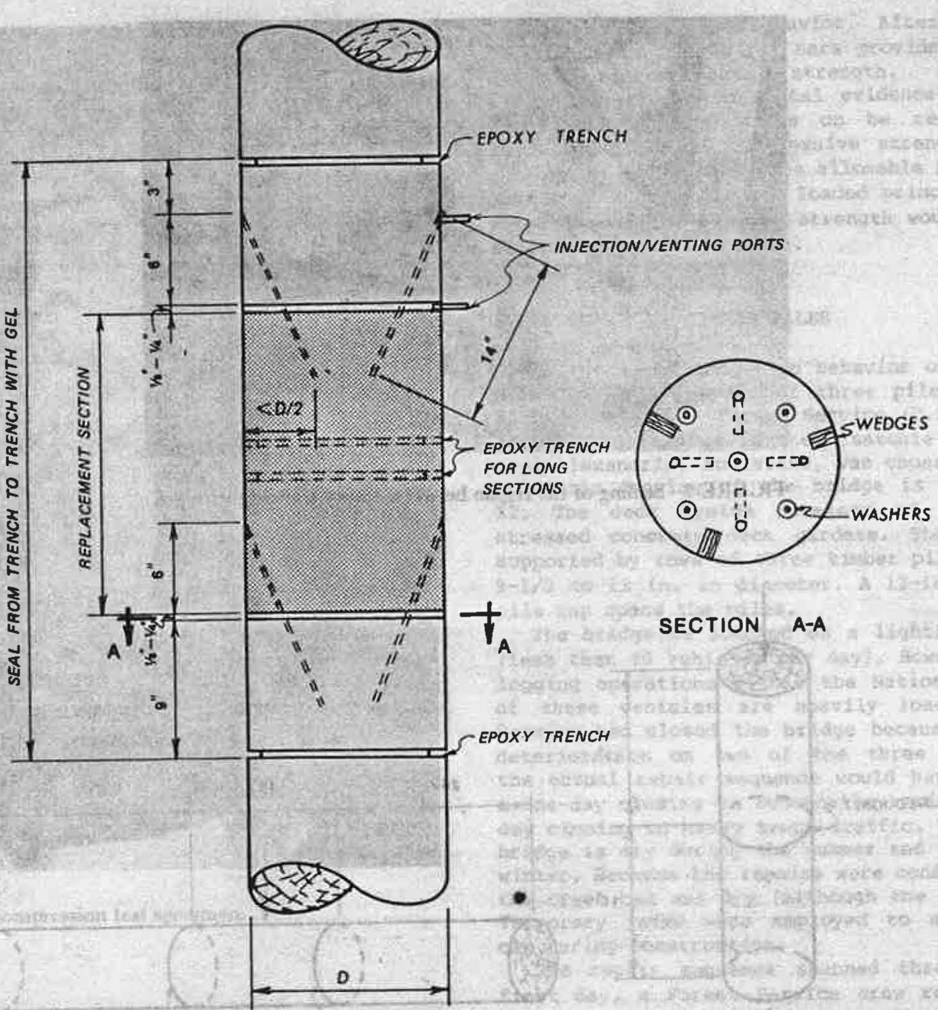

FIGURE 4 Schematic layout for epoxy repair of timber piles.

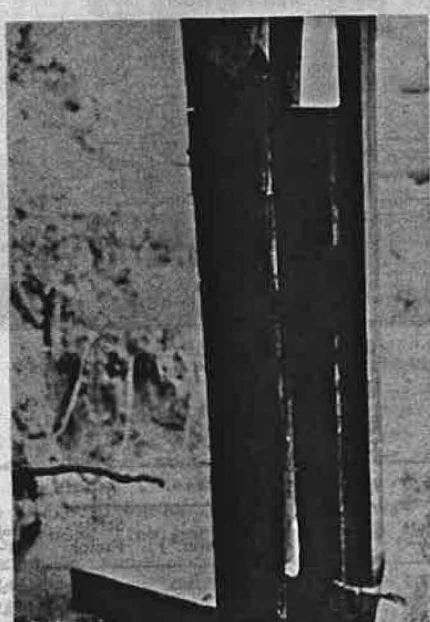

FIGURE 5 Pile with replacement section in place.

FIGURE 6 Spiral steel rod being placed at joint.

FIGURE 7 Sealing of the region between epoxy trenches with gel.

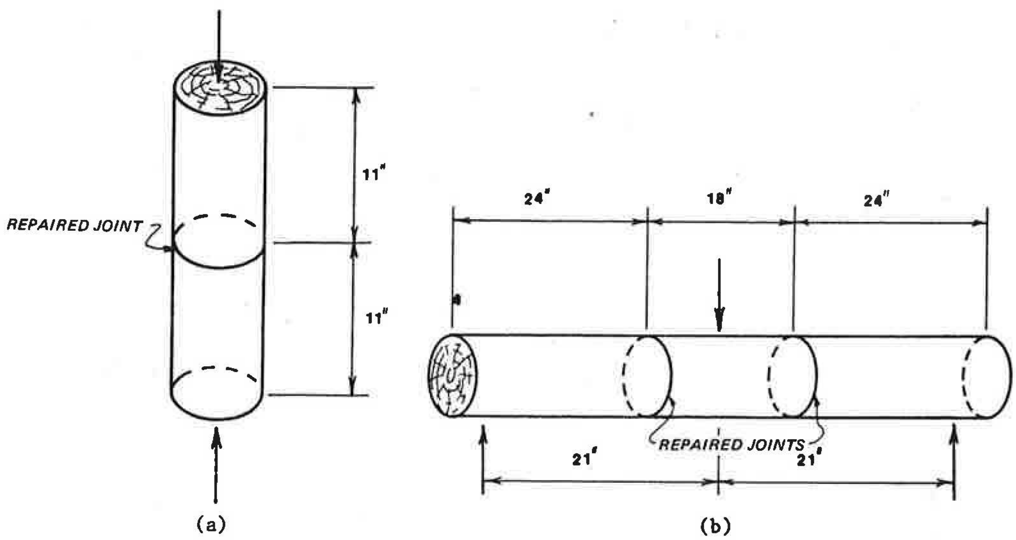

FIGURE 8 Typical test specimens for axial and flexural load tests.

TABLE 2 Axial Load Tests of Timber Piles

Specimen No.	Diameter (in.)	Area (in.)	P_{ult} (kips)	σ_{ult} (kips/in.^{2})	σ_{all} (kips/in.^{2})	Safety Factor
10	6.93	37.7	98.5	2.61	1.2	2.18
11	8.03	50.7	147.5	2.91	1.2	2.43
12	8.31	54.2	180.0	3.32	1.2	2.77
Average				2.95		2.46
Undamaged control specimen		41.3	120.5	2.92	1.2	2.43

TABLE 3 Flexural Load Tests of Timber Piles

				At Centerline		At Repaired Section			
Specimen No.	Diameter (in.)	Section Modulus (in.^{3})	P_{ult} (kips)	σ_{ult} (kips/in.^{2})	M_{ult} (ft-k)	σ_{ult} (kips/in.^{2})	σ_{all} (kips/in.^{2})	Safety Factor	M_{ult} (ft-k)
1	8.49	56.6	15.6	2.89	13.7	1.65	1.85	0.89	7.8
2	7.62	45.7	13.8	3.18	12.1	1.82	1.85	0.98	6.9
3	7.41	43.1	12.4	3.02	10.9	1.73	1.85	0.94	6.2
Average				3.03		1.73		0.94	
Undamaged control specimen	9.41	69.5	38.3	5.78	33.5	5.78	1.3	4.44	33.5

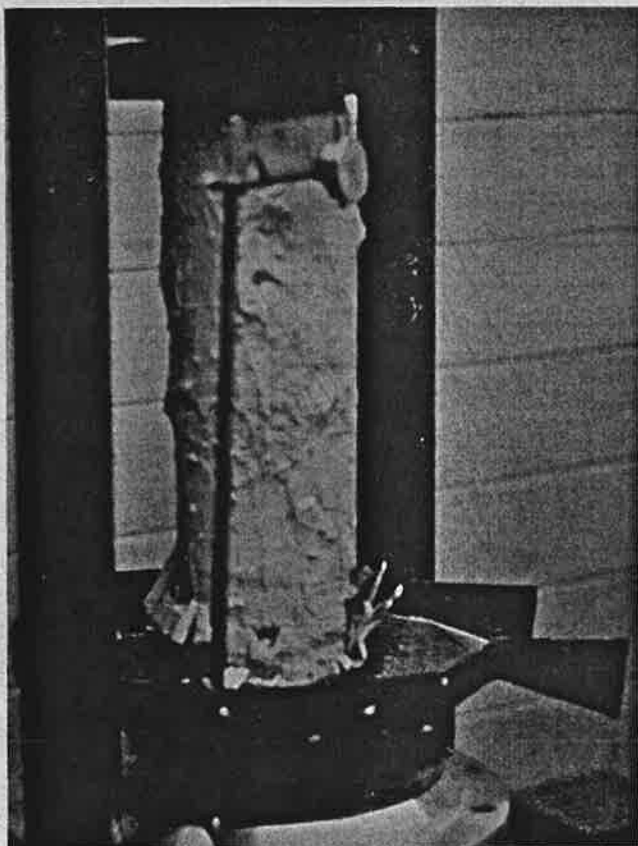

FIGURE 9 Failed compression test specimen.

FIGURE 10 Failed joint of a typical flexural test specimen.

failure did not occur at the epoxy-to-wood bond, but rather in the epoxy itself. This failure type indicates the effectiveness of the pressure-injected epoxy at the bond line because the epoxy failed before the bond.

A load-deformation curve was plotted for each specimen. These plots are shown in Figure 11. It can be seen that the stiffnesses of the epoxy-repaired sections are similar to that of the control specimen, which was a solid section of the same stock. Although not measured, it should be noted that the failures represented ductile behavior. After ultimate load was obtained, the spiral bars provided a significant level of reserve ductile strength.

In summary, experimental evidence indicates that the epoxy-repaired piles cn be restored to both satisfactory axial compressive strength and bending stiffness, but only to the allowable bending strength level. Because piles are loaded principally in axial compression, the bending strength would generally be a secondary consideration.

FIELD REPAIR OF TIMBER PILES

To evaluate the long-term behavior of epoxy-repaired piles, a field repair of three piles was conducted in August 1983. A Forest Service (U.S. Department of Agriculture) bridge in the Kisatchie National Forest near Alexandria, Louisiana, was chosen for repair. A schematic drawing of the bridge is shown in Figure 12. The deck system consists of double-web prestressed concrete deck girders. These girders are supported by rows of three timber piles ranging from 9-1/2 to 12 in. in diameter. A 12-in. square timber pile cap spans the piles.

The bridge is located on a lightly traveled road (less than 20 vehicles per day). However, because of logging operations within the National Forest, some of these vehicles are heavily loaded. The Forest Service had closed the bridge because of the severe deterioration on two of the three piles. However, the actual repair sequence would have only required a one-day closing to automobile traffic and a three-day closing to heavy truck traffic. The creek at the bridge is dry during the summer and flows during the winter. Because the repairs were conducted in August, the creek bed was dry (although the soil was moist). Temporary jacks were employed to support the pile cap during construction.

The repair sequence spanned three days. On the first day, a Forest Service crew removed soil from around each damaged pile until sound wood was reached. A backhoe was used with from 2 to 4 ft of soil being removed from around the piles. This operation required about 2 hr. The extent of damage was determined on each pile by drilling and examining the chips. The three replacement sections were selected from a local plant and brought to the site during the first day.

The actual repairs began on the second day with a four-man crew. The jacking, cutting, posting, sealing with epoxy gel, and leak testing was completed for all piles. On the third day, the epoxy injection was completed and the deformation gauges were set.

The damaged piles are shown in Figures 13 and 14. Piles 1 and 2 were damaged from the pile cap to a depth of 5 ft and were 10 1/2 and 11 1/2 in. in diameter, respectively. These 5-ft sections were replaced with new treated sections obtained from a local lumber yard. Pile Number 1 with the damaged section removed is shown in Figure 3, and the new section in place at Pile Number 1 is shown in Figure 6. Pile Number 3 was repaired by replacing a 3 ft section beginning at 2 ft below the pile cap and having a 9 1/2-in. diameter. The new section in place is shown in Figure 5. All piles were repaired using the procedure described previously. After the replacement sections were placed and wedged, the jacking forces were released. Figure 7 illustrates the sealing phase of the repair on Piles 1 and 2. The repaired Piles 1 and 2 are shown 1 year after repair in Figure 15 and 2 years after repair in Figure 16.

All piles were successfully injected with little leaking. In Piles 1 and 2, the epoxy migrated upward to the pile cap in spite of the epoxy trench. Pile

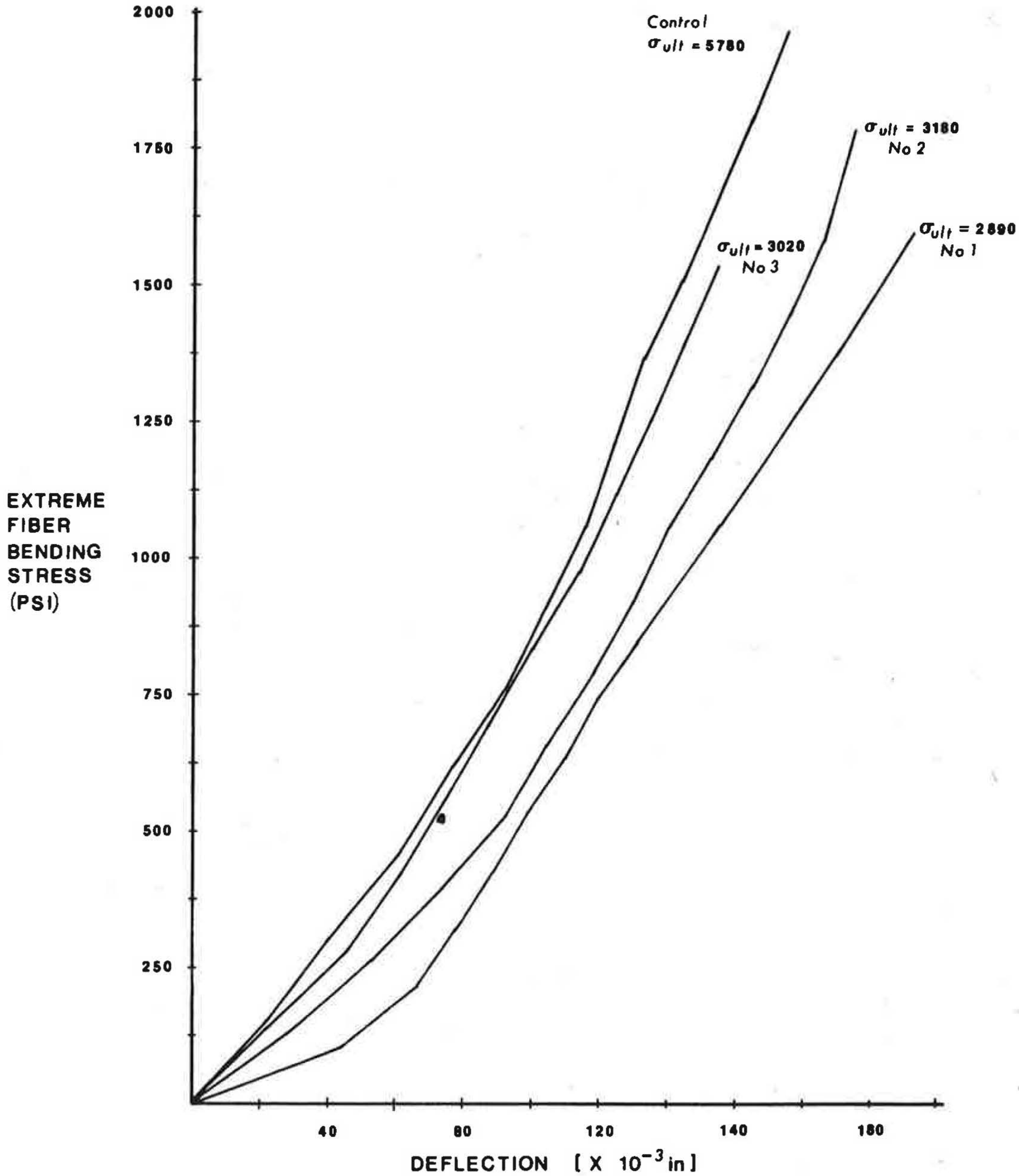

FIGURE 11 Load-deflection curves for flexural tests.

Number 3 showed no migration problems. For each case, a nozzle back-pressure of approximately 20 psi was obtained. A set of permanent deflection measuring devices was placed on each repaired joint as shown schematically in Figure 17. These devices are being monitored over a multi-year period to evaluate long-range behavior of the epoxy-repaired piles. The results from the gauge readings over the first 2 years are given in Table 4 and indicate that practically no relative movement has occurred at any of the epoxy bond lines.

SUMMARY AND CONCLUSIONS

A method for posting and epoxy grouting timber piles is detailed and a laboratory investigation has been conducted to evaluate the strength and behavior of epoxy-repaired timber piles. In addition, three piles were field-repaired at a bridge in the Kisatchie National Forest near Alexandria, Louisiana. These piles are being monitored for long-term behavior.

Although the study sample was small, some tentative conclusions can be drawn from the research, for example, (a) posting and epoxy grouting will provide a predictable level of structural restoration, (b) the axial compressive strength can be restored to the original design capacity, and (c) the flexural strength can be restored to a level of approximately one-half the original design capacity. It is therefore recommended that design computations on strength after repair be based on: (a) the full design allowable stress of the existing pile in compression; and (b) 45 percent of the allowable stress of the existing pile in flexure.

The repair procedure restores the flexural stiffness to approximately that of the original pile. In addition, the flexural failure mode is ductile. When the epoxy bond line fails in tension, there is a reduction in load-carrying capacity. However, the

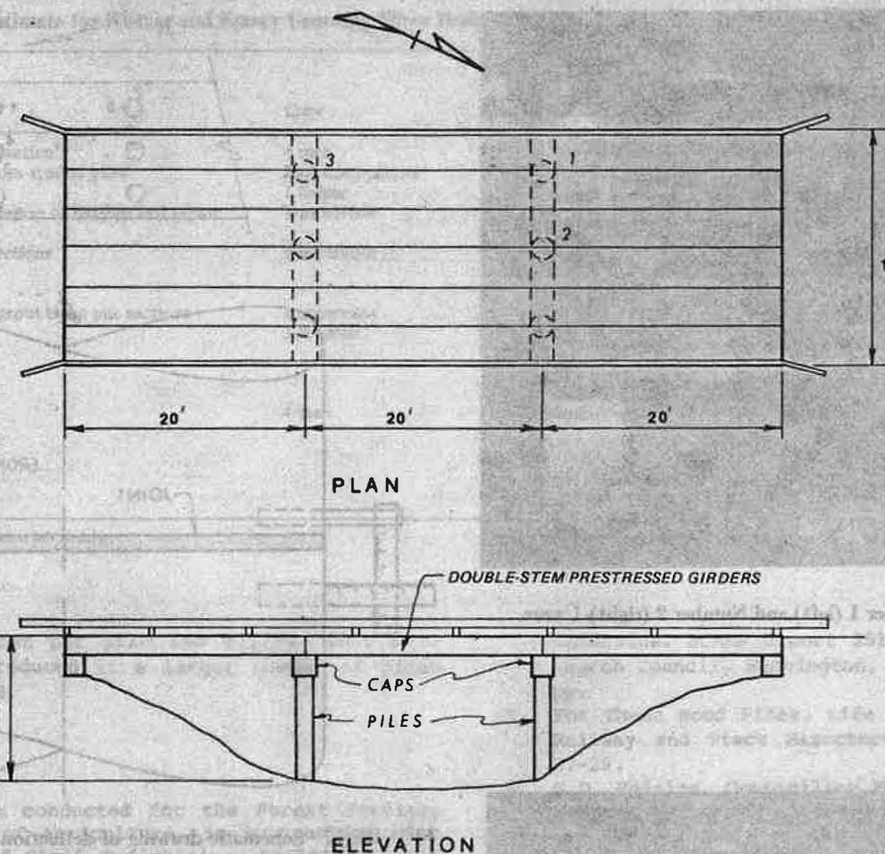

FIGURE 12 Schematic diagram of bridge to be repaired.

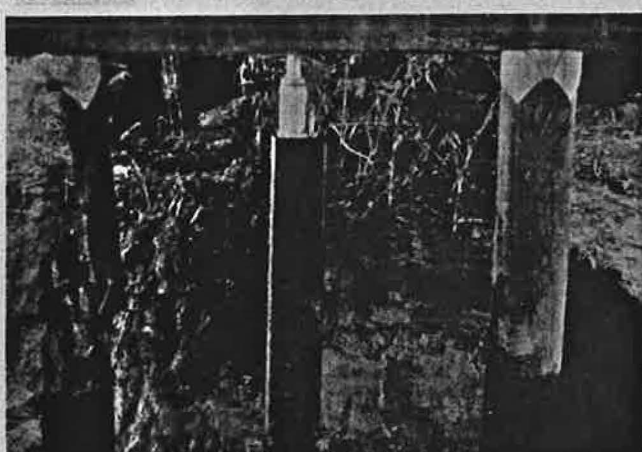

FIGURE 13 Damaged piles Number 1 (left) and Number 2 (right) with jack in place.

twisted steel pins remain bonded and provide a significant ductile range in the load-deformation curve.

An important aspect of forming a strong epoxy bond line is the pressure injection. The use of pressure ensures that the epoxy is forced into the wood fibers to produce a strong bond. It provides a level of quality control on the bonding process that cannot be obtained by simply pouring epoxy into the joint as is sometimes recommended by contractors.

Because the epoxy gains strength very rapidly,

FIGURE 14 Damaged pile Number 3.

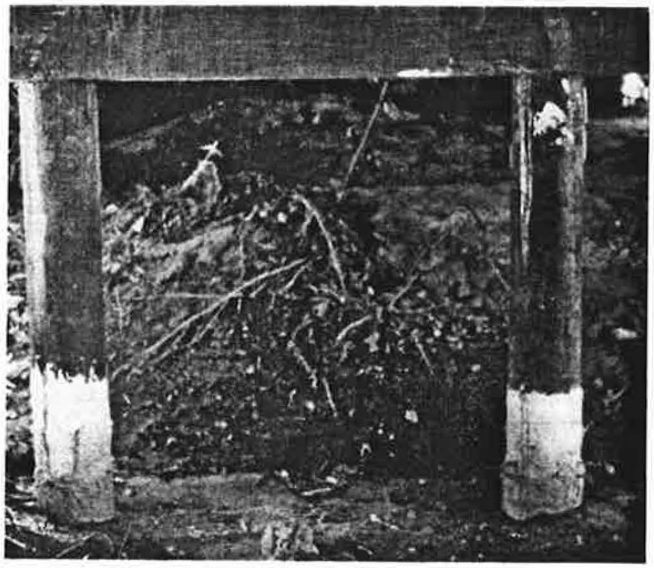

FIGURE 15 Piles Number 1 (left) and Number 2 (right) 1 year after repair.

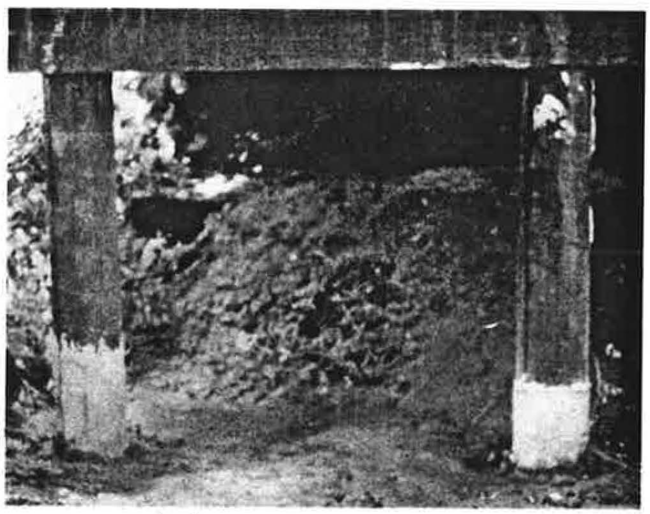

FIGURE 16 Piles Number 1 (left) and Number 2 (right) 2 years after repair.

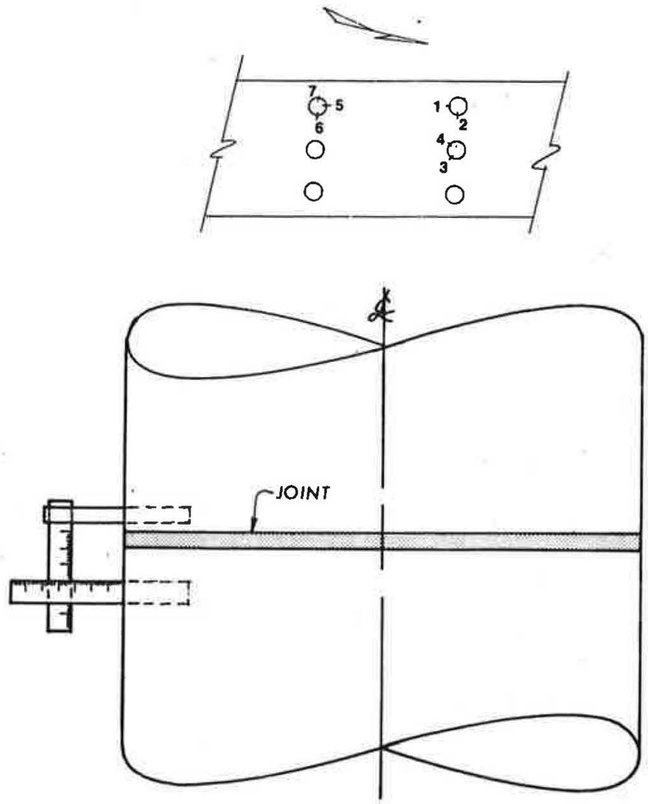

FIGURE 17 Schematic drawing of deflection gauges at a repaired pile joint.

the bridge can be quickly reopened to traffic. For both the epoxy used here and similar types used in wood repair, the initial cure time varies with temperature, but is typically about 4 hr. At initial cure, the epoxy has 50 percent of its ultimate compressive strength. The final cure time to obtain full compressive strength ranges from 24 hr at 90°F to 72 hr at 50°F. Thus, a bridge can be opened to full traffic within one to three days, depending on the temperature.

A final area of interest is the cost of the repair. Because unit costs vary considerably in different localities, only an approximate estimate can be given. A cost breakdown for the three pile repairs near Alexandria, Louisiana, is given in Table 5. For

TABLE 4 Results of Long-Term Bridge Monitoring

Gauge No.	Change in Deflection from Initial Reading on Aug. 11, 1983 (in.)					
	11 Days		1 Year		2 Years	
	Horizontal	Vertical	Horizontal	Vertical	Horizontal	Vertical
1	1/64	0	1/64	1/64	1/64	0
2	1/64	0	0	0	1/64	0
3	1/64	1/64	0	0	0	0
4	0	0	0	1/64	0	1/64
5	0	1/64	0	1/64	0	1/64
6	0	0	0	1/64	Gauge Damaged	Gauge Damaged
7	0	0	0	0	0	0

TABLE 5 Cost Estimate for Posting and Epoxy Grouting Three Bridge Piles near Alexandria, Louisiana (1983)

Task	Crew	Material/Equipment	Unit Cost ($)	Units	Total Cost ($)
Mobilization/demobilization[a]	4 men	Truck transportation to site	0.50/mi	360 mi	180
Remove soil/debris from around piles[a]	Backhoe operator	Backhoe	100/hr	4 hr	400
	1 helper		10/hr	4 hr	40
Inspect and evaluate degree of damage and repair procedure	1 supervisor		40/hr	2 hr	80
Obtain replacement sections	1 supervisor		40/hr	4 hr	160
		Three 12-in. diameter replacement sections x 10 ft long	3/ft		90
Cut, post, and epoxy-grout three pile sections	1 supervisor		40/hr	12 hr	480
	3 laborers		10/hr/man	36 hr	360
		Epoxy	40/gal	6 gal	240
		Generator	10/hr	12 hr	120
		Miscellaneous supplies	200	Job	200
Crew living expenses	4 men	Motel/meals	200/day/crew	2 days	400
Subtotal					2,750
Overhead and profit (40%)					1,000
Total					3,750

[a]May vary considerably from job to job.

this job, the cost per pile was $1,250. This unit price would be reduced if a larger number of piles had been repaired.

ACKNOWLEDGMENTS

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