Task 1

Review of Literature

Introduction

Stress-laminated deck superstructures consist of a series of lumber laminations that are placed edgewise and are transversely compressed with high-strength prestressing bars to create large structural assemblies. The concept for these bridge designs began in the 1970s as Ontario highway officials developed new methods to rehabilitate nail-laminated timber deck bridges (Taylor and Csagoly, 1979). In contrast to longitudinal glulam deck assemblies and nail-laminated assemblies, which achieve load transfer among laminations by structural adhesives or mechanical fasteners, the load transfer between laminations in stress-laminated bridges is developed through compression and interlaminar friction. This interlaminar friction is created by the high-strength steel stressing elements typically used in prestressed concrete. By 1983, design specifications were available for longitudinal stress-laminated timber deck designs in Canada (Ontario Ministry of Transportation and Communication, 1983; Taylor, 1983). Design specifications for these bridges became available in the U.S. in 1991 (AASHTO, 1991).

The early stress-laminated deck bridge designs had simple rectangular cross sections composed of longitudinal sawn lumber laminations with transverse prestressing (i.e., slab type systems). Early research on these bridge designs was conducted at the University of Wisconsin in cooperation with the USDA Forest Service (Dimakis and Oliva, 1987; Oliva and Dimakis, 1986; Oliva and Dimakis, 1988; Oliva et al., 1985). Construction of these types of bridges in the U.S. began in 1989 with work conducted cooperatively between the USDA FPL and West Virginia University (WVU). Fifteen of these bridges were built in West Virginia between 1989 and 1991, with an emphasis placed on utilizing locally available hardwood lumber. Other bridges using different lumber species were subsequently constructed at many locations around the U.S. These designs proved relatively successful in short span applications and they were relatively cost effective. However, their moment of inertia was limited by the size of the available sawn lumber, which is generally 16 inches or less. Therefore, to achieve longer spans with stress-laminating technology, new concepts were needed.

To meet this need for longer spans, researchers first developed a stress-laminated T-beam type of superstructure. The first such design was completed by researchers at WVU and the Barlow Drive bridge was subsequently constructed in 1989 in Charleston, WV (Dickson, 1995; Dickson and GangaRao, 1990). The system consisted of stress-laminated top flanges connected with continuous webs. Flange material was sawn hardwood lumber while the webs could be constructed of glulam beams, laminated veneer lumber (LVL), parallel-strand lumber, or other non-wood structural products. The Barlow Drive bridge used LVL webs. The next stress-laminated T-beam bridge was not constructed until 1992 and then nine more such bridges were installed in West Virginia with spans up to 119 ft.

Due to the high cost of the T-beam bridge, researchers shifted their emphasis to a cellular or box beam type of superstructure (Barger et al., 1993a, 1993b; Davis, 1992; GangaRao and Latheef, 1991; Lopez-Anido and GangaRao, 1993). The system consisted of stress-laminated top and bottom flanges connected with continuous webs. The first Box-beam bridge was designed in 1990 and constructed in Morgantown, WV. After the first bridge was constructed, 26 more Box-beam bridges were installed across West Virginia through 1994 with spans up to 104 ft. During 1992, other similar Box-beam bridges were constructed in South Dakota and New York as part of the US Timber Bridge Initiative.

While this activity was occurring in the U.S., researchers in Australia also began experimenting with the concepts of stress-laminated timber deck bridges (Crews, 1996; Crews and Walter, 1996; Crews and Walter, 1998; Crews and Bakoss, 1996). Their work began in 1990 and they constructed over 30 prototype slab-type bridges using similar technology to that of the U.S. As in the U.S., they realized that to extend the stress-laminated timber bridge technology to longer spans, they would need to develop different design configurations. Then in 1995, they began a fundamental research and development program for cellular bridge decks. Their basic design calls for using LVL webs and sawn lumber for flange material. To date, they have constructed several prototype bridges with spans up to 82 ft.

Stress-Laminated T-Beam and Box-Beam Research

Analytical Models

In 1975, GangaRao derived a macroapproach for analyzing structural plate systems stiffened by ribs or grids commonly used in the construction of highway bridges. Essentially, the solutions that come from this mathematical approach are a combination of fast converging infinite Fourier polynomial series. The design characteristics that can be analyzed using this macroapproach include deflections, moments, stresses, displacements, and torsional effects. Even though this macroapproach was initially developed for use with concrete highway bridges, the same theories are used as the basis for the structural analysis and design of the T and Box bridges developed by WVU.

Deflection Characteristics

Barger et al. (1993a) conducted a study of stress-laminated T-beam and Box-beam bridges to gain a thorough understanding of the behavior and performance of these types of bridges. Several performance parameters were tested. These included stiffness, strength, load distribution, deck depth, stringer size and spacing, interlaminar stress level, and the use of diaphragms.

Static bending tests were performed on model T and Box beam bridges in the laboratory and were compared with field test data of existing T and Box bridges. They tested two different size combinations of flanges and webs. The smaller bridge size combination consisted of webs that were southern pine glulam beams measuring 3 in. wide by 12.125 in. deep by 20 ft long and flanges that were made from northern red oak nominal 2 by 4 lumber resulting in flange thicknesses of 3.5 in. The larger bridge size combination consisted of webs that were southern pine glulam beams measuring 5 in. wide by 24.625 in. deep by 20 ft long and flanges that were made from northern red oak nominal 2 by 6 lumber resulting in flange thicknesses of 5.5 in. They tested two different web configurations: three webs with a web spacing of 53 in., and four webs with a web spacing of 32 in. Each model was tested with two different levels of interlaminar stress: 100 psi and 50 psi.

The bridge sections were tested by applying a single concentrated load at midspan. In one case, they applied a uniformly distributed line load across the width of the bridge. In the other cases, they applied point loads on either an interior web, an exterior web, or on the laminated deck between two webs. The test span was 19.3 ft. Maximum loads applied during the test appeared to be 8,000 lbs.

With an increase in web depth, they found that deflections and strains in the web decreased when static loads were applied. Also, an increase in the number of webs used in a design was nearly proportional to an increase in bridge stiffness when all other bridge parameters were held constant. When the interlaminar stress level of the stressed bridge system was doubled from 50 psi to 100 psi, there was an increase of the transverse deck stiffness of 20%. Finally, the Box-beam model used for this research was found to be 2.5 times stiffer than its T-beam counterpart because of an increase of the moment of inertia due to the bottom flange of the box bridge system.

They also found that loading on the exterior web leads to higher flexibility of the bridge system when diaphragms were not used. This implies that transverse load distribution is better when diaphragms are used in the bridge design. For the T-beam models used in this study, when diaphragms were placed between both interior and exterior webs, the bridges showed 15% higher stiffness than the models without diaphragms.

The size of the flange lumber was found to be an important factor in terms of improved effective width, composite action, and transverse load distribution. With these bridges, the thinner the flange, the poorer the bridge performed. Although they only tested 3.5 in. and 5.5 in. thick flanges, they recommended that a minimum of a 9 in. flange thickness should be used for field bridges. This thickness recommendation was not conditioned on bridge length or web spacing.

They concluded that transverse wheel load distribution was strongly influenced by flange thickness, bridge cross section type (T or Box), and web size. They also concluded that transverse load distribution was not strongly influenced by interlaminar stress level as long as slip did not occur. They recommended a minimum interlaminar stress level of 50 psi.

The effective flange width of the T-beam models was found to be 50% of the center-to-center distance of two adjacent webs. The effective flange width of the Box-beam models was noted to be 80% of the center-to-center stringer spacing. These findings were based on the tests conducted at two different web spacings (53 in. and 32 in.), which were similar to actual construction practices.

In 1994, Oliva and Subramanian studied the deflection characteristics of a multi-box section. The test specimens were not modeled after a particular bridge, but the model was roughly onehalf scale. The three-cell box specimen was tested under a constant transverse stress-laminating interlaminar stress of 60 psi. Since the transverse prestressing was solely responsible for maintaining the structural integrity of the structure and testing began as soon as the specimen was stressed, this study does not provide details on long term interlaminar stress loss in the system. The loads imposed on the bridge ranged from 0 to 5000 pounds to ensure that the bending stresses would not exceed ½ the published allowable levels. Also, there were no rotational restraints on the bridge at the supports during testing.

The test results indicated that the specimen behaved in a partially-composite manner, which means that the flanges were not able to resist longitudinal bending without experiencing shear slip at the web-flange joints. At the 60 psi interlaminar stress level used, the web-flange joints behaved as if they were only partially rigid in transferring the internal. The authors noted that the selection of an appropriate anchorage configuration for the prestressing rods was critical in forming an acceptable system. They stated that an efficient anchorage system for a box beam bridge should directly transfer the prestress force to the flanges with no out-of-plane bending in the exterior webs.

In order to make the Box-beam bridges stiffer, Oliva and Subramanian, (1994) suggest the use of diaphragms or mechanical shear connectors. They suggested that the diaphragms be designed to transmit the same amount of wheel load and deflection to each of the webs, and that the diaphragms will not resist the transverse axial, or prestress, force.

GangaRao, et al. (1997) studied the occurrence of slip in T-beam bridges at loads near 40 to 60% of the ultimate load capacity of the individual webs. At these loads, horizontal shear stresses caused slip between the glulam webs and the flanges. This slip causes a loss of composite action in the bridge and a reduction of the effective flange width. These two factors caused an overall loss of stiffness of the bridge system.

In this study, the authors showed that experimentally-determined load-deflection curves can be used to estimate the point at which slip begins. Also developed in this study were two analytical methods to determine the onset of slip. The first method uses the coefficient of friction and a given applied stress to compute a horizontal resisting force due to friction. Then, an average compressive stress on the flange was computed. A slip load was calculated by equating the slip-resisting force to the average compressive stress. Finally, the slip load was compared to the estimated experimental slip load. The second analytical method used classical beam theory to calculate EI values for the beam when it was acting as a fully-composite unit and a partially-composite unit. Then, the interlayer shear stress between the web and flange was calculated using the shear stress equation VQ/It. The force that the shear stress had to overcome to cause slippage was calculated for each transverse compressive stress. The interlayer shear stress was equated to this force to find the point at which slippage occurred. They found that the first of these two methods was a better approximation of the point at which slip occurred. However, since the friction coefficient is not known, there really is no best method.

Effective Flange Width

In 1992, Sara Davis performed a study on an I-beam cross section to evaluate effective flange width, the deck width over which the normal stress can be assumed constant. From her experimentation and modeling, she determined that an effective flange width of .7 times the actual flange width should be used for spans of 23 feet and .8 times the actual flange width for spans of 45 feet. Also, it seems that the ratio of span length to flange width may be a factor in determining the effective flange width.

Also in 1993, Davalos and Salim used a special finite element program and a regression analysis to develop a prediction equation for the effective flange width of T-beam timber bridges. In their analysis, actual bridge dimensions, orthotropic material properties, and AASHTO truck loads were used to create 125 different bridge models to be analyzed by finite element analysis. The regression analysis performed on the data created by the finite element analysis was used to develop a prediction equation for the determination of effective flange width. Because of the complexity of the derived equation, a simplified linear equation was determined so that the findings from this study could be used by engineers in an everyday design capacity. The authors determined that the variables that had a major effect on the effective flange width were web spacing, bridge span, ratio of web depth to thickness, and the ratio of the web's longitudinal elastic modulus to flange elastic modulus.

Barger et al. (1993b) outlined preliminary effective flange width guidelines for both T- and Box-beam bridges. In their study, they performed static testing of scale models of T- and Box-beam bridges with two different web spacing configurations, one consisting of a three web configuration and the other a four web configuration. The experimental midspan deflections and transverse load distribution factors were then evaluated. Using the laboratory data, experimental stresses and a composite moment of inertia was calculated and the effective flange width was determined. For a T-beam system, they recommended that an effective flange width of S/3, where S was the web spacing, for interior webs and S/4 for exterior webs should be used. For Box-beam bridges, they recommended that effective flange widths of S/4 for interior webs and S/5 for exterior webs should be used. They also found that over a long period of time, the interlaminar stress level would stabilize near 50 psi. They recommended that this value should be used as the interlaminar stress level for design purposes. Also, they noted that the use of diaphragms effectively increased the transverse stiffness of the T-beam bridge system.

Bending Characteristics

In 1988, Dickson and GangaRao designed a stress-laminated T-section bridge that consisted of a stressed timber flange with LVL webs. Their research attempted to quantify the effects of load distribution, composite action, creep, laminate separation, and slip. These experimental data were then used to confirm finite element models and to arrive at design recommendations for the use of stressed timber T-beams for highway bridges.

At the time this research was done, there was no information on the load distribution properties or factors for the stressed T configuration being studied. Hence, the first logical step was to experimentally determine a load distribution factor for safe and economical web design. Using strain gauges on the stringers and dial gauges to measure deflection across the deck width, they found that the directly loaded web carried 50% of the applied load and the adjacent webs carried 25% of the applied load.

The next issue studied was composite action in the bridge system. A major assumption in the preliminary design of stress-laminated T-beam bridges is total composite action between the flange and web. Theoretically, the T-section design takes advantage of the much stronger longitudinal bending strength of the decking by placing it parallel to the LVL webs. Also, the stressing bars further contribute to the overall strength of the bridge system by increasing transverse moment capacity. By comparing the experimental stresses imposed on the web and the calculated stresses, they determined that the assumption of composite action in the stressed-T system was valid.

In any stressed wood deck, creep is a factor to be taken into consideration. Creep is prevented by checking the post-tensioning in the bridges periodically and retensioning the rods when necessary. When tension levels are too low, poor load sharing and slip are the result. The Ontario Highway Bridge Design Code (OHBDC) guidelines specify that creep will begin at loads equal to 1/8 that of the compressive strength of the timber. The stress losses from creep in the experimental bridge were very close to what the OHBDC recommends as the maintenance level. Dickson and GangaRao suggested that retensioning be done within the first 6 to 8 weeks after the bridge is initially tensioned to eliminate creep loss.

Another problem created by an inadequate amount of bar force on the bridge system is separation of the deck laminations. Theoretically, when the laminae separate when loaded, the contact area between two adjacent planks is reduced and slippage can occur. However, when the experimental bridge deck was loaded, gaps occurred between some of the laminae, but no slippage occurred. The areas in which compression increased by the applied loading seems to adequately compensate for the loss of contact area.

The final concern in a stressed-T bridge system that was examined by Dickson and GangaRao was the potential of slippage between adjacent deck laminations. If stressing bar forces in the bridge become too low, the load carrying capacity of the bridge becomes dependant on the rod-wood connection rather than the force applied by the tensioning rods. They stated that even though one of the safety features of the design was to sustain an AASHTO HS-20 live load with no post-tensioning, slippage still makes a stressed timber bridge unsafe. Experiments were conducted on a stressed-T system deck where no evidence of slippage had occurred.

Finally, a finite element model and orthotropic plate equations were employed to try to accurately model the stressed-T timber bridge system. Both the finite element model, using plane quadrilateral elements, and Dr. GangaRao's plate equations modeled the bridge system well with modulus of elasticity, shear modulus, and coefficient of friction as the required inputs.

Dimakis and Oliva (1988) set out to develop a new type of stress-laminated timber bridge. After considering several types of truss and composite beam possibilities, they settled on the Vierendeel or box type truss. This bridge system consisted of two parallel chords spaced 8 inches apart on center with web members, measuring 4 inches by 24 inches, running perpendicular to the parallel chords spaced 4 feet apart on center.

The first stage in their study was to test the bridge's resistance to truck loading. Three different loading cases were considered. A double-wheel truck loading was placed at midspan, 1.5 feet from each edge of the deck in the transverse direction, and two point loads were placed 4 ft. to either side of midspan (directly above the web openings). These loadings were tested on clear span lengths of 50 ft., 39.5 ft., and 31 ft. These experimental loadings were then compared against a finite element analysis. The finite element model developed by the authors paralleled the experimental truck loadings and model results were within 10% of the deflections in all of the load cases and span combinations.

In addition to the static loadings, dynamic loadings were also studied. A cyclic loading simulating an HS20 truck was applied on the butt-jointed flange to determine if cumulative slip of the butt joints would occur and to determine if a reduction of the elastic stiffness of the bridges occurred due to a repeated load. After all of the cyclical loadings were applied, there was no significant deterioration of the bridge stiffness. These results held true if and only if the interlaminar stress across the butt joint was maintained at a proper level of 50 psi.

The next phase of this study was to determine the best configuration for the anchorage systems for the prestressing rods. Four slightly different rod configurations were tested. They determined that these variations had little effect on load resisting behavior as long as the anchorages were reasonably designed to transfer prestressing force into both the chord and webs. Also, anchorage plates must be designed to avoid the crushing of wood underneath the plates.

Through these tests, the composite action of the parallel chord truss was examined. The test results showed that the moment of inertia was 40% of what the moment of inertia would be if the chords were perfectly connected. Much of this moment of inertia reduction was due to shear deformation of the truss system in the region between the web blocks.

In 1996, Crews and Bakoss investigated the fundamental behavior of stress-laminated timber bridges. In this study, they primarily investigated the mechanisms at work when the bridge fails. For both T- and Box beam bridges, the slip between laminations is the governing factor in the structural performance of these bridges. When the web-flange interface slips, this marks the beginning of non-linear load deflection behavior and the potential for permanent deflection in the deck after the load has been removed. Also, Crews and Bakoss noted that the interlaminar stress level in the bridge has a marked effect on the stiffness of the deck and the potential slippage problems stated above.

Crews and Walter (1998) studied the bending characteristics of both T- and Box-beam bridges in Australia. The Box-beam bridges studied are similar in concept to the ones developed at WVU. The main differences in the Australian Box-beam bridge is the closer spaced (no greater than 500 mm) and thinner webs (45 to 65 mm) and the webs are made from LVL. The testing program tested individual box cells with depths of 600 mm, 900 mm, and 1200 mm with interlaminar stress levels of 1200, 1000, 700, and 500 kPa. The authors found that the onset of slip and non-linear deflection behavior began at a interlaminar stress level of 700 kPa and when the interlaminar stress level slipped below 500 kPa, there was a drastic change in the stiffness and strength properties of the box cell specimen. Hence, Crews and Walter suggest that the interlaminar stress level on the Box-beam bridges be maintained at 700 kPa with an absolute lower bound of 500 kPa.

Stress-Laminated Timber Bridge Design Methodologies

WVU Design Method

GangaRao and Raju (1989) presented the WVU method of designing steel, reinforced concrete, and stress-laminated T-beam and Box-beam bridges. The study was done specifically to determine transverse load factors for various types of bridges. In this approach, most of the important parameters (e.g., aspect ratio, number of lanes, material orthotropy, stiffness of webs, and number of webs) have been accounted for in the development of design equations for stresses and deformations. The major steps of this design methodology were to establish the flexural rigidity of the deck, obtaining an edge deflection coefficient, which is a function of the rigidities of webs and bridge width, and finding a wheel load fraction for the particular bridge loading.

Davalos et al. (1992) presented a revised design methodology for stress-laminated T-beam bridges. Their method is based on defining a wheel load distribution factor derived from a macro-flexibility orthotropic solution of a plate stiffened by webs. This wheel load factor reduces the bridge design to that of a T-beam section. Also included in the design procedure was the determination of effective flange width and the local effects caused by wheel loadings applied between two adjacent stringers.

Davalos et al. (1993) used a macro solution for a stiffened plate and orthotropic plate finite element modeling to verify the WVU method as a viable way to design stress-laminated T-beam bridges. The models were verified by actual load testing of the Camp Arrowhead bridge. Also, details of fabrication and testing of the Camp Arrowhead bridge were explained in the report.

All of the design methods developed and used by WVU are based on allowable stress design. Crews (1996) promotes and has developed a design methodology for timber bridges using a limit states, or load and resistance factor approach. In this limit states approach, the wood strength properties are developed using in-grade testing of randomly obtained timber from stocks of commercially available timber instead of clear wood samples used to obtain the strength properties on which the allowable stress design approach is based. For design purposes, it is more informative to the designer to know how weak a structural member is rather than how strong it is. Essentially, this is the difference between the two philosophies. The limit states approach shows how weak a piece of wood is and the allowable stress procedure shows how strong a piece of wood is. Until the time of this report, a strength modification factor was used in conjunction with the allowable stress design procedures to account for the higher factor of safety that bridges require. In this report, Crews begins to develop a limit states, or reliability-based, design procedure. His first step was to replace the strength modification factor with a capacity reduction (phi) factor, which is based on a probabilistic analysis of load effects and material variability. This phi factor reduces the likelihood of failure due to a combination of excessive overloading and inadequate timber strength. To go along with the phi factor, Crews then defines the vehicular loads normally considered: strength and serviceability requirements for a normal truck loading and the strength requirements for an abnormally heavy truck load. The serviceability limit state is mainly concerned with deflection limits.

Bridge Performance and Case Studies

The majority of the field performance reports in the literature are summarized by Dickson (1995). He summarized performance of the T-beam and Box-beam bridges constructed in West Virginia as part of the Timber Bridge Initiative. Field performance data for Box-beam bridges constructed outside West Virginia were given by Wacker et al. (1997, 1998). Although different types of stress-laminated T-beam bridges are being constructed in Australia (Crews and Walter, 1996; Crews and Walter, 1998), field performance results for these bridges have not been reported in the literature.

Box-Beam Bridges

Dickson (1995) summarized performance of Box bridges constructed in West Virginia as excellent. These bridges were generally constructed in accordance with design procedures developed at WVU (Lopez-Anido and GangaRao, 1993). The design load for the bridges was an AASHTO HS25 truck (AASHTO, 1989). All Box-beam bridges in West Virginia used southern pine glulam webs and flanges consisting of Red Oak lumber. All wood components were preservative treated with creosote. The stressing bars were spaced 2 ft apart and the design interlaminar interlaminar stress level was 100 psi. Design procedures allowed for a 60% loss of interlaminar stress before a restressing operation was recommended.

Dickson (1995) described general results of a monitoring project for the Fieldcrest and Cedar Creek State Park bridges. Over the two-year monitoring period, bar forces dropped to approximately 60% of the initial design bar force. Moisture contents of the flange boards fluctuated between 20% and 25% while web moisture contents fluctuated between 12% and 18%.

Load testing showed the bridges to be as stiff or stiffer than the design methods predicted. They noted low transverse stiffness of the bridge decks. Camber losses were detected on both bridges monitored, but the amount of creep loss they measured was only slightly larger than that expected from creep. Their continued observations showed that the rate of loss of creep was greatest in the first year of service for the bridge. Dickson indicated that all Box-beam bridges in West Virginia were maintaining positive camber. He also noted the presence of creosote leaching on the bridges.

Dickson (1995) also discussed specific problems encountered during the erection and use of all the Box-beam bridges in West Virginia. These problem areas included: interior diaphragms, camber losses during construction, non-square box modules, and excess creosote.

On the earliest constructed Box-beam bridges, interior diaphragms were specified. These diaphragms, if fabricated too long, prevented the stressing operation from compressing the flange lumber as intended. Apparently this occurred on the Upper Five Mile Creek bridges and it led to slippage of the top flange boards. Subsequent bridges had the requirement removed for interior diaphragms.

The early bridges also had problems with loss of camber during their erection resulting from small modules (more than two modules per bridge) that were not completely stressed together before installation. The solution to this problem was to increase the size of web members to deal with non-composite behavior before final stressing. This method led to more expensive designs since the webs were larger than the final bridge really required. Therefore, most subsequent designs used bridges composed of only two modules.

One of the most significant problems noted by Dickson was the occurrence of non-square Box modules. These non-square modules resulted from a non-uniform compression of the flanges and they caused transverse bending of the web members. Since the transverse bending of the web members stressed the glulam beams in their weak axis, there were some beams that cracked. To solve this problem, steel beam bulkheads were recommended to distribute stresses across the face of the web.

A common problem in many timber bridge systems has been excess creosote coating the surfaces of the wood components. On the early Box-beam bridges, excess creosote accumulations coated the surfaces of the glulam beams and dripped onto the area under the bridges. To partially alleviate this problem, the design specifications currently recommend steam cleaning the beams before they leave the treatment plant.

Although most of the Box-beam bridges built have been constructed in West Virginia, Wacker et al. (1997, 1998) described the field performance of two Box-beam bridges in other states. Wacker et al. (1997) discussed performance of the Spearfish Creek bridge near Spearfish, South Dakota. The design load for the bridge was an AASHTO HS20 truck (AASHTO, 1989). This bridge, which was installed in 1992, used southern pine glulam webs and flanges consisting of nominal 2 in. by 6 in. Ponderosa pine lumber. The webs were preservative treated with creosote and the flange lumber was treated with pentachlorophenol. In this bridge, the stressing bars were placed 34 in. apart and the design interlaminar stress level was 100 psi. Using this interlaminar stress level, stressing bar spacing, and flange thickness, the design stressing bar force was 19,600 lb.

During the monitoring period on this bridge, they recorded moisture content losses for both flange lumber and webs. At the time of bridge construction, the mean moisture contents of the flange lumber and the glulam webs, respectively, were approximately 20% and 15%. During the next two-year period, the mean moisture contents of the flange lumber and the glulam webs, respectively, decreased to approximately 15% and 14%.

During the monitoring period extensive losses in stressing bar forces were observed. Bar forces decreased 50% during the first year after construction. At this point, a restressing operation brought the forces back up to design levels; however, one year later, the bar forces had dropped 60%. An additional restressing operation took place approximately three years after construction. Then in the following months, the bar forces decreased 40%. They noted that future restressing operations may be needed. This excessive loss in bar force was attributed to the combined effects of reductions in moisture contents and stress relaxation in the flanges. The phenomenon of stress relaxation was discussed by Ritter (1990).

Results of static load tests showed that the reductions in bar forces affected the transverse load distribution in the bridge. However, the loss in interlaminar stress had relatively small effects on the longitudinal stiffness of the bridge due to the stiffness of the glulam web members. Maximum predicted deflection values during the load tests, for loads equivalent to those from an AASHTO HS20 truck, corresponded to approximately L/738 and L/731 for the first and second load tests, respectively.

They noted no crushing of wood under the steel anchorage plates. Also, they noted only minor reflective cracking in the asphalt wearing surfaces over the bridge supports.

Wacker et al. (1998) reported results from a monitoring program on the Christian Hollow Box-beam bridge located in Steuben County, New York. This bridge, which was installed in 1992, used southern pine glulam webs and flanges consisting of nominal 2 in. by 6 in. mixed hardwoods lumber. Both the webs and flange lumber were preservative treated with creosote. In this bridge, the stressing bars were placed 36 in. apart and the design interlaminar stress level was 100 psi. Using this interlaminar stress level, stressing bar spacing, and flange thickness, the design stressing bar force was 21,600 lb.

During the monitoring period on this bridge, they recorded moisture content losses for both flange lumber and webs. At the time of bridge construction, the mean moisture contents of the flange lumber and the glulam webs, respectively, were approximately 26% and 20%. During the next two-year period, the mean moisture contents of the flange lumber and the glulam webs, respectively, decreased slightly to approximately 25% and 19%.

During the first year after construction, bar forces decreased 50%. At this point, a restressing operation brought the forces back up to the design level of 100 psi. After this activity, it appeared that the bar forces began to stabilize to a final interlaminar stress of 70 psi. Over the 28-month monitoring period, there was a slight camber loss in the bridge. Under static load tests at the beginning and end of the monitoring period, the bridge exhibited linear elastic behavior. However, a 10% decrease in longitudinal bridge stiffness was observed with the 30 psi drop in interlaminar stress. Maximum deflection values, for loads equivalent to those from an AASHTO HS20 truck, corresponded to L/1123 when interlaminar stress was approximately 70 psi.

They noted no crushing of wood under the steel anchorage plates. Also, they only noted minor reflective cracking in the asphalt wearing surfaces over the bridge supports. In addition, there was slight rutting in the asphalt wearing surface.

T-Beam bridges

In 1990, Dickson and GangaRao presented the results of a field monitoring program of LVL T-beam bridges. The results indicated that the force level, 21.8 kips (67% of original jacking force), in the bars was satisfactory, flange and web deflections were at acceptable levels, and the weathering of all structural timbers was negligible. The two main concerns from the author's viewpoint were creep and the misalignment of stringers during fabrication. Another concern was the high cost of the bridge due to the high rental crane costs, the high LVL costs, and the conservative design of the bridge. Among the author's recommendations for future LVL T-beam use were limiting spans from 40 to 60 feet to make costs reasonable, increase stringer spacing from 27 in. to 36 in., and chose stringer and deck material by geographic location.

Dickson (1995) summarized performance of four T-beam bridges in West Virginia, in a two-year monitoring program, as excellent. He noted that bar force losses were greater than those of other stress-laminated bridge designs. During the monitoring period, bar forces dropped to approximately 50% of the initial design bar force. Moisture contents of the flange boards fluctuated between 20% and 25% while web moisture contents averaged 15% initially but rose to approximately 20% at the end of the two-year period. Although recommendations on steam cleaning of glulam components for Box-beam bridges were adopted for T-beam bridges, there were still cases of creosote leaching from wood components exposed to direct sunlight.

From the results of load tests conducted on the T-beam bridges in the monitoring program, he concluded that design procedures were "reasonably accurate and safe." Their measured stresses and deflections were stated to be "within allowable limits for AASHTO HS-25 loading." Dickson's discussion of specific problems encountered during the erection and use of the T-beam bridges centered around non-square T modules. As described for the Box-beam bridges, when the modules were tensioned, the flanges did not compress uniformly and the modules became unsquare (webs were not perpendicular to the flanges). Therefore, when the modules were positioned on the bridge bearings, there were gaps between the tops of the modules and, when the modules were tensioned together, the outer modules were lifted off the bridge bearings.

To combat the problem of the non-square modules, an additional set of stressing bars was added below the flange at distances that were typically two to three inches below the flange as shown in the inspection notes. These bars were placed on approximate 12 ft centers. When the extra bars were tensioned along with the main sets of bars, the modules were forced into the desired shape.

Dickson (1996) reported on the cost, fabrication, and installation of the Camp Arrowhead and Nebo T-beam bridges. The Camp Arrowhead bridge measures 63.5 ft in length by 23.75 ft. wide and the Nebo bridge measures 33.5 ft. in length by 21.25 ft. wide. In this report, Dickson explains the major complication during the fabrication process, misshapen modules when the bridge was stressed together. The superstructure costs for the two bridges were $54/sq. ft. and $79/sq. ft. For both bridges, the stiffness measured by a live load test was higher than anticipated, moisture levels were acceptable, and creosote retention was adequate. In the Camp Arrowhead bridge, the bar forces dropped rapidly but stabilized at an acceptable level. The bar forces on the Nebo bridge were above 50% of the applied prestress force.

An alternative to the T-beam bridges developed by WVU was developed by the Trus Joist MacMillan Corporation. This bridge system consists of several one-piece T cross-sections made from LVL; the individual T's are stressed together to make a bridge. Ritter et al. (1996) evaluated the performance of six of the Trus Joist stress-laminated LVL T-beam bridges and noted that the bridges were performing sufficiently at that time and that they should continue to perform well for several years to come. Specifically, the LVL responded quickly to changes in the environment, namely moisture content. The loss of bar forces in the LVL was not a problem because as the LVL material gains moisture it tends to swell and thereby allows the bridge to maintain relatively high bar forces. Also, the response of the LVL bridges to loading was linear elastic, similar to that of stress-laminated bridges built from sawn lumber.

Camber Loss

Dickson and GangaRao (1995) studied the causes of camber loss in stress-laminated bridges. Camber loss occurred more rapidly in bridges that did not have full-length members, had higher than average daily truck traffic, and on bridges with a large span to depth ratio. Camber loss was found to accelerate when transverse interlaminar stress dropped below 30 psi and when there were impact loadings on the bridge.

Other Case Studies

In 1992, McCutcheon evaluated the performance of the Mormon Creek bridge, a stress-laminated parallel-chord truss bridge in Michigan. The bridge is 40 feet long and 16.5 feet wide and was constructed of No. 1 Douglas Fir. This bridge was monitored for 3 years after its erection and was load tested twice. The most notable characteristic of this bridge was the crushing of the softwood blocks used to transmit forces into the trusses. These blocks were placed directly behind each bearing plate. Most of the bearing plates were embedded into the softwood blocks at the completion of the stressing activities. These blocks were replaced with hardwood blocks that performed adequately, even though some crushing is evident. However, all other aspects of the bridge's performance were satisfactory. All of the stressing rods retained adequate force and the bridge showed, through load tests, adequate stiffness under design loads. Also, the bridge retained a positive camber at the time of the tests.

Cost Effectiveness

Verna et al. (1984) highlighted the benefits of using wood as a bridge material. They noted that timber is very resistant to deicing agents and normal water exposure, and its low maintenance are important factors that put timber ahead of steel and concrete. In one of the bridge replacement cases in the study, the costs per square foot for the bridge replacement alternatives (excluding approach and stream work) were $37 for timber, $70 for reinforced concrete, $77 for an open steel grid, and $83 for a concrete filled steel grid. Construction of beams and decks with timber cost less than steel or concrete primarily because smaller equipment can be used since timber is comparatively light.

Sarisley (1990) reported on the construction and cost of installing stress-laminated timber bridges to replace a nail-laminated timber bridge. Sarisley estimated a productivity improvement of 300% over the pneumatic nailing process, but he commented that improvements in the jacking process to stress the bridge needed to be made. The author states that the formulation of a complete and well-planned material specification list is a crucial factor in the success of a completed project. The initial construction cost of the stress-laminated deck was 68% of traditional steel and concrete superstructures for that area. Also, the predicted low maintenance cost will further favor stress-laminated decks for bridge life comparisons.

In 1995, the Timber Bridge Information Resource Center released a report detailing the superstructure costs for vehicular timber bridges based on geographic location, timber bridge type, bridge length, and timber species used in the bridge (Coole, 1996). The average cost for all timber bridge superstructures in the United States was $49.31 per square foot. The regional cost varied from $63.22 per square foot in the northeast, which includes West Virginia, to $42.33 per square foot in the region that includes Idaho, Nevada, and Utah. For all of the different types of timber bridges in the United States, the average cost of a timber bridge was $60.26 per square foot. According to this report, the average cost for T-beam and Box-beam bridges was $68.18 and $64.83 per square foot, respectively. This report also shows that there is no correlation between the cost of timber bridge and the length of the superstructure. The most expensive timber type to use are Northern hardwoods, which costs $77.49 per square foot, and the least expensive is Douglas Fir, which costs $40.38 per square foot. Southern pine falls in the center of the spectrum at $61.32 per square foot and bridges made of a combination of red oak and southern pine, as were the stress-laminated T-beam and Box-beam bridges in West Virgina, cost an average of $68.07 per square foot to build.

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