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Fatigue Evaluation of Structural Composite Lumber (SCL) T-Beam Bridge Girders

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**FATIGUE EVALUATION OF
STRUCTURAL COMPOSITE LUMBER (SCL)
T-BEAM BRIDGE GIRDERS**

By

Muhammad Badre Enam

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This dissertation is dedicated to my parents Mr. Badre Ebriz and Mrs. Akhtar Jahan. I also dedicate this work to my wife Salowa Sultana.

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ABSTRACT

Recent innovation in the engineered wood industry has produced Structural Composite Lumber (SCL) that tends to achieve maximum strength, stiffness and efficient use of wood. Product variations of SCL such as Laminated Veneer Lumber (LVL) and Parallel Strand Lumber (PSL), are currently being used in the transportation sector to produce bridge girders, decks etc. for rural and other low traffic volume roads. Although, the elastic and shear properties of SCL materials are clearly understood, no attempt has been made to estimate fatigue performance of these bridge girders. With their increasing use, they are repeatedly being subjected to numerous cycles of wheel load that raises concern about their behavior under fatigue. The current research tested 12 newly procured and 2 old and weathered pre-built SCL T-beam bridge girders for flexural fatigue under a stress controlled test setup. The girders were of 20 ft span and having variation in their constituent material types and applied preservatives. Loading was applied according to the AASHTO LRFD specifications for bridge design. The number of load application cycles was consistent with expected traffic volume for a sixty year time span. In a regular bridge structure comprised of pre-built SCL T-beam girders, transverse post-tension is applied to adjacent girders to form a stress laminated deck. This effect had been simulated on the girders and the effect of fatigue loading on the loss of post-tension force was observed. Results from the study indicated that the SCL T-beam bridge girders were capable of withstanding the repetitive loads coming from the heavy truck traffic. Most of the beams did not suffer any physical damage. A few of them had severe de-lamination at the SCL-epoxy interface. The girders without any damage due to fatigue were loaded statically up to failure. This result was compared to the ultimate flexural strength of fresh beams of similar material type, to investigate their residual strengths after fatigue loading. Some weathered girders were also fatigued under the same experimental setup for comparison purposes. This provided a comparison of the fatigue performance of fresh and weathered SCL T-beam bridge girders. Comparably high strength fatigue loading was applied to two additional girders. The fatigue lives for these girders were less and compared well with those of clear wood at the same stress level. A rudimentary fatigue curve was proposed based

on the data. A finite element model was developed, using layered shell elements, to simulate the structural behavior of these girders. Subsequent analyses of this model produced results that were highly representative of the data obtained in experimental testing of SCL T-beam girders.

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CHAPTER 1

INTRODUCTION

1.1 Problem Statement

Rural bridges in the United States serve an important role in the economy of the country. Although they carry a low volume of traffic, their sheer number demands special attention. On a nationwide basis, rural bridges constitute 80 percent of the total number of bridges (FHWA 2004). These bridges help in transportation of agricultural crops, rural access to emergency and service vehicles. States and local authorities continue to build these rural bridges out of wood, as it is considered a viable option for short span deployment. Wood as a structural material, however, has some shortcomings which need to be addressed through research. The primary response to this problem has been the Timber Bridge Initiative passed by Congress in 1988. Also, Section 1039 of the 1991 Intermodal Surface Transportation Efficiency Act (ISTEA) has included provisions for research and technology transfer into timber structures. The Timber Bridge Initiative has been charged to the USDA – Forest Service; while the ISTEA program is the responsibility of the Federal Highway Administration (Duwadi 1995).

The objective of the Timber Bridge Initiative (TBI) was to further develop and extend the use of wood as a bridge material (USDA 1997). As a part of this objective, emphasis has been placed on the use of new engineered wood products and innovative bridge designs. One group of engineered wood products that has been adapted for bridge application is Structural Composite Lumber (SCL), which is increasingly becoming more popular. This material possesses excellent uniformity in strength, stiffness and dimension as well as excellent treatability using common preservatives. With its increased use in bridge applications, it has

become necessary to investigate some of its long-term properties. Fatigue is one of such properties, which demands attention, as virtually no information is available on the behavior of SCL under fatigue loading. Since SCL T-beam girders are increasingly being used in roadway bridges, they are being subjected to millions of load applications from the wheels of vehicles causing fatigue. How these girders behave under such repetitive loads, therefore, becomes important for the long-term viability of SCL as an alternative to other wood products.

1.2 Structural Composite Lumber (SCL) Bridges

Structural Composite Lumber (SCL) is a generic name for a family of engineered wood products. Although they were introduced in the United States in the early 1970s (Winistorfer 1997), they have gained popularity due to their superior qualities over traditional wood products. Also the abundance of SCL in the absence of continued supply of large timber logs for large-scale lumber productions, has established them as a suitable alternative. Diversification of the SCL product family has popularized their use in home building, commercial construction and transportation industry.

SCL is manufactured by gluing together rotary-peeled thin sheets of wood that are dried to a lower moisture content. The constituent wood is processed into either strips or thin veneers. The strips or veneers are pressed together with heat and high pressure after an adhesive is applied. This helps the individual constituents to act like a composite material which has more uniform properties and is free of strength reducing defects such as splits, knots, warping etc. The final product is manufactured as billets which can be re-sawn into required dimensions.

Availability of SCL in large dimensions has facilitated its use in transportation industry. Currently it is used to produce railroad ties, marine structures, signposts and light poles, sound barrier walls, bridge girders and bridge decks (Trusjoist.com 2005). The SCL billets can be cut into desired shapes and then combined to produce T-beams or I-joists to be effectively used as bridge girders. The resulting section has nearly uniform material properties which makes the design easier and its behavior predictable. Also, due to the excellent

penetration of preservatives in SCL, such girders may be built to last up to 60 years without being affected by environmental hazards (Trusjoist.com 2005). Currently Hughes Brothers in Nebraska is the sole manufacturer of these kinds of transportation products. They are capable of producing these girders with a span length up to 64 ft (Trusjoist.com 2001).

1.3 Fatigue in SCL Bridges

Over the years many SCL bridges have been manufactured in the United States. Most of these bridges are located in rural areas. Some of them are entirely made of SCL while others may be steel-SCL composites. Forest Product Laboratory of United States Department of Agriculture has done extensive research on these bridges. They have constructed many such bridges and continue to monitor their long term performances. However, since SCL is a newly invented material compared to other construction materials, many of its long term properties are not well documented. One such property is fatigue. According to commercial publications, the life expectancy of bridge girders made out of SCL is 60 years. This is quite comparable to girders made out of other materials like concrete or steel, although, SCL boasts a huge advantage in terms of cost over the other two. During this period of time, these girders will experience huge amount of truck and other vehicular traffic. Also with increased growth of economy the traffic volume is expected to increase exponentially. The behavior of SCL under such repetitive loads has not been explored beforehand. This poses a hindrance in the wide-spread acceptance of SCL in the transportation industry as a low-cost alternative.

1.4 Goals and Objectives

The fatigue resistance of a material is generally represented by an “S-N” curve, which describes the stress range S vs. number of load application cycles N at failure. To this date no such “S-N” relationship has been published for SCL material itself. Constructing such a relationship requires a huge number of fatigue tests of SCL material, properly sampled and sized. The whole procedure involves enormous amount of time and other resources. Another approach towards fatigue performance analysis will be to consider the performance of the whole structural element itself. This implies undertaking fatigue tests on a structural member like beams or slabs and looking at its behavior rather than fatiguing the material itself on a

micro scale. Forces can be applied to the member in accordance with its life expectancy and the estimated load within this time. Modification of the properties due to fatigue can be evaluated and conclusions can be drawn on the members fatigue performance. The following research predominantly adhered to this philosophy and tried to find out the fatigue behavior of full-scale SCL T-beam bridge girders. It was expected that this approach would be more suitable and relevant towards the viability of long-term applicability of SCL in the transportation industry. In addition to the above methodology current research attempts would perform fatigue tests on the SCL girders so as to construct a rudimentary S-N curve. A computer based finite element model would also be developed which might be used to simulate the structural performance of these girders. Such a model would facilitate the deployment of these girders by predicting their behavior during in-service condition without resorting to expensive laboratory testing.

The initial focus of the current study was to perform extensive literature survey of the existing research involving SCL material. This helped in setting up the stage for the experimental procedures conducted in this study. The review of existing research provided means of identifying the parameters that were relevant to the fatigue study. Following the literature review a methodology was developed for the fatigue testing. This methodology tried to create such conditions for the SCL T-beam within the lab, which would closely resemble the conditions of an in-service girder. The simulated loading on the girder was calculated using the AASHTO LRFD specification. Although timber structures, as a general rule, are not designed for ultimate loads, the application of AASHTO LRFD loads ensured that the girder was being subjected to the maximum load possible. A rational calculation then estimated the number of load application cycles, which was based on the expected future truck traffic on the girder. Fatigue testing of the SCL girder was then performed using these parameters.

It is, generally, a common practice in a laboratory experiment to perform the same test on a number of samples to avoid accidental erratic results and to bring out the variability within the test results. Following the custom, fatigue tests were performed on a number of SCL t-beam bridge girders. However, the type of these girders varied in SCL material and preservative type to represent the wide spectrum of SCL T-beam girders those were

commercially available. In addition to the fatigue testing of these girders, some weathered girders were also tested in fatigue. This helped to identify the differences in performances between a new and in-service girder.

Some of the fresh SCL T-beam girders, which had not been loaded in fatigue, were tested in static loading. This type of static testing was also performed on fatigued girders which did not suffer damage due to cyclic loading. In each case the beam was loaded up to the point where failure occurred. A comparison of the failure load of a fatigued girder to that of a fresh girder provided an estimation of the residual strength available in SCL T-beam girders at the end of their life-span.

The construction technique of bridges made entirely out of SCL involves placement of number of SCL girders side by side, with subsequent pre-stressing of the girders in the transverse direction. This pre-stressing is performed using Dywidag bars with appropriate anchor plates and bolts placed on both ends. The whole system then produces a stress-laminated deck system which does not require additional strengthening and is capable of handling heavy truck loads. However, with fatigue loading over time, the stresses in the Dywidag bars may be lost gradually and the deck will not show composite behavior. Therefore, it is required to have an estimate of the loss of pre-stress in these bars. The current fatigue test had provisions for measuring the pre-stress within the Dywidag bars. Attempts were made to measure this value at various stages of the fatigue test.

Since the pre-built SCL girders were expensive, it was not possible to construct a truly representative S-N curve by fatiguing a large number of these at different mean stress. However, a prediction of such a curve was derived using available published data and subjecting a few specimens to fatigue loading at different load levels.

A finite element modeling of the SCL T-beam girder was also produced. In the absence of practical “S-N” values for SCL and due to the approach taken within the computer program towards fatigue modeling, the finite element model of the SCL T-beam was not tested for fatigue. Emphasis was given on the accuracy of the model in achieving results that

were close to the experimental values. The notion here was to come up with a model that would closely resemble the elastic performances of these types of girders provided that some of their exact material properties are supplied. Such a model would give engineers an efficient tool to predict the performance of their design before actual on-site deployment.

Following is a summary of the objectives that were prominent for the current research.

- Investigate the existing literature for fatigue studies of SCL material. In case such information was not found, relevant studies for wood and wood laminates would be researched.
- Identify the parameters that are relevant to a laboratory based fatigue study of full-scale SCL girders.
- Procure SCL girders based on a test matrix that included variation in SCL material types and other relevant parameters.
- Calculate the design load in fatigue cycles according to AASHTO LRFD specification so as to maximize the effect from repetitive truck traffic.
- Perform fatigue testing on full-scale SCL girders in the laboratory which closely simulates cyclic loading similar to the combined vehicular traffic expected during the life expectancy of the girders.
- Observe the extent of damage within the girders due to fatigue loading.
- Identify the effect of cyclic loading on post tension force applied to the girder that simulates the composite deck effect of a real life in-service girder.
- Apply static loading to some of the un-fatigued SCL girders so as to determine their ultimate strength.
- Statically load the girders that survived the fatigue loading. Load would be applied to determine their ultimate strengths. A comparison of ultimate strengths of fatigued girder with those of fresh girders would provide a measure of their residual strengths.
- Identify the effect of SCL material types and other parameters on the results obtained.
- Prepare a rudimentary S-N plot of the SCL girders, if possible, based on the fatigue load applied and the fatigue life of the girders, which is denoted by the number of load cycles that each girder withstood before failing in fatigue.

- Prepare a finite element model of the girder based on the material characteristics of SCL and perform static loading on the model so as to identify its performance compared to the results obtained in the actual laboratory testing.
- Discuss the results obtained in various testing and analysis and make recommendations accordingly.

CHAPTER 2

LITERATURE REVIEW

2.1 Structural Composite Lumber

In recent years, large diameter high quality timber logs have become scarce. However, the demand for such logs for the production of full scale solid sawn lumber has increased by many folds. This has led to the invention of Structural Composite Lumber (SCL). SCL is an emerging group of engineered wood product that is generally manufactured using small wood pieces glued together into sizes common for solid-sawn lumber. Currently SCL is being used as a substitute of lumber in various applications. It is also used to manufacture ready-to-deploy engineered wood structural elements as beams, columns, I-joists, T-beams, shear panels etc. These products are designed to fully utilize the excellent material properties of SCL which are superior to traditional lumber products.

2.2 Types of SCL

Structural Composite Lumber is commercially available in various types. This variation is generally attributed to the manufacturing process. Also SCL may vary in their constituent material wood and the type of preservatives applied. Currently SCL is being made out of Douglas Fir, Southern Yellow Pine etc. However, it is also possible to produce SCL from Aspen or other underutilized species that are not commonly used for structural applications (Moody 1991). SCL can also vary in their orientation of constituent wood material. This variation produces SCL of different directional strength properties which are suited for particular structural applications. The following is a list of available SCL types.

2.2.1 Laminated Veneer Lumber

Laminated Veneer Lumber or LVL is the most commonly available structural composite lumber. Several companies like Trusjoist (Weyerhaeuser), Georgia Pacific Corp., IP Wood, Pacific Woodtech Corp. produce LVL in large quantities. LVL is manufactured by bonding together wood veneers using adhesive so that the grain of all veneers is parallel to the longitudinal direction. This is a bit different from the construction of plywood where adjacent veneers have their grains aligned in orthogonal directions. In the US, LVL is produced using rotary-peeled veneers 1/8 to 1/10 in thick, which are hot-pressed with phenol-formaldehyde adhesive into lengths from 8 to 60 ft or more. These veneers are graded very carefully to maintain certain moisture content and weight criteria. They are also subjected to ultrasonic testing to ensure that the LVL will have desired engineering properties. Figure 2-1 shows the production steps in LVL manufacturing.

Figure 2-1: Production Steps of LVL

Commercial LVL is produced in sheets with 2 to 4ft width, having a thickness of 1.5 in. Using continuous press technology sheets can be manufactured having unlimited lengths. These sheets are then cut into various widths and desired lengths as per the application requirement. A widely available variety of LVL produced by the Trusjoist Company has a commercial name of Microllam.

2.2.2 Parallel Strand Lumber

In contrast to LVL, which constitutes wood veneers; Parallel Strand Lumber or PSL is made out of wood strands or strips with wood fibers primarily oriented along the length of the member. Generally 1/8 in. thick wood veneers are cut into 3/4 in. to produce these strips. The strands will have a minimum length of 24 in. Because of the strand shape, the PSL production has the advantage of using rotary-peeled veneers from round logs as well as waste material of plywood or LVL production. The strands are coated with a waterproof structural adhesive, commonly phenol-resorcinol formaldehyde. Afterwards, they are placed in a longitudinal press, properly oriented and having equal distribution along the length of the press. The pressing operation causes densification of the pile of wood strands while microwave heating technology dries up the adhesive. The resulting product is a billet of PSL. PSL billets are generally larger than those of LVL and are commonly sized at 11 by 19 in. cross-section. The billets are then sawn into smaller pieces according to the requirement. Figure 2-2 shows the production steps of PSL while Figure 2-3 shows the whole manufacturing process.

Virtually any variety wood species can be used for this purpose. Most commonly used species for PSL production include Douglas-fir, Southern Yellow Pines, Western Hemlocks and Yellow-poplar. Although LVL is commercially produced by many companies in the United States, production of PSL is not wide-spread. Currently, PSL is produced by only the Trusjoist Company, who sells the product under the commercial name of Parallam.

Figure 2-2: Production Steps of PSL

Figure 2-3: Manufacturing of PSL

2.2.3 Laminated Strand Lumber and Oriented Strand Lumber

Laminated Strand Lumber (LSL) and Oriented Strand Lumber (OSL) are manufactured by varying the same technology used to produce Oriented Strandboard (OSB) structural panels. One type of LSL uses strands that are about 12 in. long, which is somewhat longer than strands commonly used for OSB. Waterproof adhesives are used in the manufacture of LSL. Another type of product uses an isocyanate type of adhesive that is sprayed on the strands and cured by steam injection. This product needs a greater degree of alignment of the strands than does OSB and higher pressures, which results in increased densification.

2.3 Material Properties of SCL

Structural Composite Lumber is free of common defects of regular solid sawn lumber such as knots, joints, insect attacks and warping. These defects are the primary cause of strength-reduction and non-uniformity of the material properties. Since SCL is made out of carefully selected wood veneers and strands, there is no chance of any introduction of defects. Also sophisticated quality control of the manufacturing process ensures an even dispersion of the constituent material within the finished SCL product. Thus SCL boasts of a very uniform material property distribution and higher design values. Figure 2-4 shows comparison of various strength properties of LVL (Microllam), PSL (Parallam) and regular solid sawn lumbers of Douglas Fir and Southern Yellow Pine. The comparison is based on tests carried out on members with 12 in. depth. It is evident that the allowable bending stresses of SCL materials are almost two times greater than those of regular commercial lumbers. The modulus of elasticity of SCL, however, is closely comparable to the regular wood. This demonstrates the fact that, although the constituent material of LVL and PSL is the same as solid sawn lumbers, the design values for SCL is much higher because of their superior manufacturing process which eliminates the strength reducing factors of natural wood. Table 2-1 shows the material properties of SCL made out of Southern Yellow Pine in more detail. Tabulated design values are for normal load duration and dry service condition. Figure 2-5 shows the details of the joist and plank orientation as mentioned in the table.

Figure 2-4: Compariosn of Strength Properties of SCL and Regular Wood

Figure 2-5: Different Orientation of SCL Beams

Table 2-1: Strength Properties of SCL

Product	Grade or Species	Member Thickness	Joist or Beam Orientation			Plank Orientation			Axial		Modulus of Elasticity E (psi)	Weight (plf/in ²)
			Shear Stress F _v (psi)	Bending Stress F _b (psi)	Compression ⊥ to Grain F _c ⊥ (psi)	Shear Stress F _v (psi)	Bending Stress F _b (psi)	Compression ⊥ to Grain F _c ⊥ (psi)	Tension F _t (psi)	Compression F _c (psi)		
Microllam Laminated Veneer Lumber (LVL)	2.0E	1½" to 1¾"	285	2750 ^(a)	750	190	3255	480	1660	2635	2.0 × 10 ⁶	0.292
	2.0E	1⅞" to 3½"	285	2750 ^(a)	880	190	3255	525	1805	2635	2.0 × 10 ⁶	0.292
Parallam Parallel Strand Lumber (PSL)	2.0E	1¾" to 11"	290	2900 ^(b)	750	210	2800	475	2250	2900	2.0 × 10 ⁶	0.319

- (a) For depths other than 12" regardless of thickness, multiply table values by (12/d)^{0.136}
(b) For depths other than 12" regardless of thickness, multiply table values by (12/d)^{0.111}

2.4 SCL in Bridge Construction

Recent innovations in the engineered wood industry and advances in the bridge design and component fabrication technology has led to cost-effective and time saving bridge projects which require minimal maintenance and provide long-lasting quality and beauty. Structural Composite Lumber is a material which fits perfectly into this segment of bridge construction. According to industry experts, the effective lifespan of properly treated SCL bridges are more than 60 years which is very much comparable to similar steel and concrete construction and even longer than conventional timber bridges.

2.4.1 History of SCL Bridges

The very first LVL Bridge in the United States was constructed in 1977. This was a prototype structure which was jointly sponsored by the USDA Forest Service and the Virginia State Highway Department. The bridge was made out of “press-lam”, a variation of LVL developed at the Forest Products Laboratory (FPL) of Forest Service. It consists of a 3.13 in thick LVL deck supported on 4.5 by 20 in. LVL beams spaced 30 in. on-center. The next known LVL Bridge was constructed in 1988. This was a 73 ft span experimental stress-laminated T-beam bridge in West Virginia. For this structure, flanges constructed of nominal 1.5 by 9 in. oak lumber laminations were stress laminated between 6 by 45 in. LVL webs spaced 36 in. on-center.

The first attempt to develop and market a bridge system constructed entirely of SCL was initiated by Trusjoist MacMillan in 1989. These bridges consist of a series of stress-laminated T-beams with box section along the outside bridge edges (Figure 2-6). Approximately 20 such bridges were constructed by 1993 with clear spans ranging from 24 to 50 ft having widths of 10 to 36 ft. LVL was replaced by PSL in 1993 and since then numerous short-span bridges have been constructed using the same design configuration. However, there was one bottleneck to the widespread acceptance of such bridge system. The *Standard Specifications for Highway Bridges* published by the American Association of State Highway and Transportation Officials (AASHTO) did not dictate any design values for SCL material which prevented many designer from utilizing this material. This was subsequently resolved

by the inclusion of design values for SCL in the 1995 interim Specifications. Since then SCL has become very popular for new bridge construction and as well as for the replacement of old dilapidated bridges.

Figure 2-6: Typical SCL T-beam Bridge Cross-Section

2.4.2 Advantages of SCL Bridges

Following are the advantages of bridge systems constructed using SCL.

- The SCL material itself makes far better use of available timber resources. Production of SCL bridge girders does not require harvesting of large-diameter trees for sawmilling which is necessary for ordinary timber bridge construction. Specially PSL is made primarily from small-diameter, second-growth trees and does not rely on the limited supply of high quality sawn lumber.
- SCL bridge systems are environment friendly. SCL is a wood-based product and comes from a renewable and abundant natural resource. In comparison, products based on steel and concrete require much more energy to harvest, manufacture and deliver and are derived almost exclusively from finite natural resources.
- SCL is manufactured to uniform moisture content. This makes the fabrication of bridge components much easier. Uneven moisture content in SCL would have resulted in uneven warping of different components of the bridge system.

- Prefabricated SCL components accept preservatives better than similar products made out of sawn lumber and glulam. The manufacturing process of SCL creates avenues for the preservative treatment to penetrate to the beam's core.
- Installation of SCL bridge system takes only a few days, using smaller and inexpensive erection equipment. The labor requirement for each project is also minimal. The lightweight, prefabricated components of the bridge system can be installed year-round without the requirements of any curing, vibrating or temperature sensitivity which are common to concrete constructions.
- SCL bridge system does not require heavy maintenance. SCL material is capable of resisting factors like warping, thermal expansion, shock and de-icing salts which can affect the integrity and long-term durability of other bridge superstructures.
- Bridges made out of SCL provide an aesthetic quality that blends in with natural, rural surroundings.

2.4.3 SCL Bridge Construction

Individual members of the SCL bridge system are custom fabricated in accordance with approved specifications and shop drawings. These members are made from sawn LVL and PSL with structural epoxy applied to the interface and pressure being applied to hold them in shape. The finished Tee and box beams, as well as guardrail components, are shipped from the fabricator to the treatment plant. Preservative treatments, which are environment friendly and water resistant, are then applied on these bridge members. After treatment, the structural elements along with necessary hardware and accessories are shipped to the jobsite for installation.

At the bridge site, the prefabricated T-beams and box girders are placed on abutments side-by-side. The girders are then connected transversely at the flanges, using 5/8" diameter, high-strength, galvanized steel rods placed 30 inches on center (Figure 2-7). Post-tension is applied on the steel rods and the girders forms a composite bridge structure with a stress-laminated deck which is capable of resisting loads with better wheel load distribution and having minimum deflection. Figure 2-8 shows the installation of an SCL bridge system which requires minimum labor.

Figure 2-7: Isometric View of Typical SCL Bridge

Figure 2-8: Installation of an SCL Bridge System

2.5 Fatigue in SCL Bridge Girders

It has been commercially claimed by the SCL manufacturer that the current system of LVL and PSL bridge girders are expected to remain in-service for a time period of 60 years. Since all the developments of SCL has been carried out within the last 30 years and specially the research on SCL bridge systems started very recently in 1989, there is no way to test such girders for long term service performance and thereby validate the claim. Although the static strength property of LVL and PSL girders are quite impressive, one property becomes prominent for any structural members subjected to long term application. This property is structural fatigue, which is defined as the process of progressive localized permanent structural change occurring in materials subjected to conditions that produce sustained or fluctuating stress and strains at some point or points that may lead to cracks or complete fracture after time or sufficient number of fluctuations (Smith 2003).

Due to its recent innovation, real life fatigue data for SCL bridge girders are not available. There is also no evidence of laboratory studies of SCL girders in recent times. Although, SCL as a material is gaining popularity increasingly, there has not been any attempt to study the fatigue properties of the SCL material itself and as such no S-N data for the material is available. There has been, however, very few investigation of wood and wood laminates in terms of fatigue. Since the primary constituent in SCL is wood, it is believed that a study of the fatigue characteristics of wood and wood laminates will shed light to the path of the current research.

Kyanka (1980) mentions the neglect of fatigue investigations of wood products. He also mentions the lack of standardized test criteria and the lack of knowledge on wood fracture morphology. Tsai and Ansell (1990) have conducted flexural tests of 270 mm (10.63 in.) long laminated hardwood and solid Sitka spruce beams. An S-N curve established for the spruce specimens at a stress ratio of $R = 0.1$ indicated that their fatigue life was 10^7 cycles at 75% of their mean flexural strength. No fatigue limit was observed. The Wood Handbook (1999) gives a fatigue life of 2×10^6 cycles for clear-grained solid-sawn flexural specimens loaded to 60% of their estimated static capacity in four-point bending with $R = 0.1$. Similar specimens with small knots and a 1:12 slope of grain have a fatigue life of 2×10^6 cycles when

loaded to 40% of capacity with $R = 0.1$. All specimens had initial moisture contents ranging from 12% - 15%, but the Wood Handbook does not list the species.

Hansen (1991) performed four-point bending fatigue tests of 2000 mm (78.74 in.) long Nordic spruce glulam beams. The tests were displacement controlled and run at a frequency of 10 Hz, where the minimum applied displacement was 3 mm (0.12 in.) and the peak displacement was varied to produce different peak loadings. An additional variable considered in the tests was grain angle, which varied from 0° – 12°. The most common types of fatigue failure were compression near a point of load application and a tensile rupture near the bottom of the beam. Sloping grain was shown to significantly reduce the fatigue life of the beams. Gowda (1994) tested three 4300 mm (169.29 in.) long curved and cambered glulam beams in fatigue. Two of the specimens were reinforced with vertical steel dowels with glue applied on them, and one was unreinforced. The fatigue tests employed a constant value of $R = 0.1$, but seven distinct load levels were used during the fatigue testing of the unreinforced specimen.

Davids and Richie (2003) tested large-scale FRP-glulam beam samples under four-point bending. They loaded the beams at 1-2 Hz up to two million cycles at a stress ratio of $R = 0.33$. Results indicate that girders with full-length or properly confined partial-length reinforcing have sufficient fatigue resistance for use as bridge girders. Kommers (1943) performed tests on Sitka spruce, Douglas fir, five-ply yellow birch and five-ply yellow Poplar. Fatigue strength was found to be 27% of static strength at 50 million cycles. Dietz and Grinsfelder (1943) found fatigue strength to be 25% of static strength at 2 million cycles while testing on two and three-ply birch plywood. Imayma and Matsumoto (1970) performed three-point bending test on Sugi wood and found fatigue to be 35% of static strength at 1 million cycles. Ibuki et al (1962) tested Japanese cypress bonded with urea or phenolic resin. The fatigue strength was obtained to be 15-20% of static strength at 10 million cycles.

2.6 Parameters of Fatigue Study

Based on the literature review conducted so far, following is a list of the parameters that tend to vary within the fatigue tests performed by the afore-mentioned researchers. The

list also provides brief explanation of the parameters and their relevance to the current investigation. It should be mentioned that these parameters relate to wood and wood laminates and may not equally apply to SCL.

- Type of loading
- Stress ratio
- Number of load application cycles
- Loading frequency
- Size of specimen
- Moisture content
- Preservative
- Grain alignment

2.6.1 Type of Loading

For fatigue investigation a four-point loading test is perfectly suited and has been employed by Hansen (1991) and Davids and Richie (2003). The test setup has been shown in Figure 2-9. This arrangement has the benefit of producing a constant moment at a considerable portion of the middle of the beam. Also adjustment of the load heads along the length of the specimen helps to accommodate desired amount of shear during testing. Having the girders set up in this arrangement, the specimens may be flexurally loaded with a constant deflection loading or a constant stress loading. Most of the previous investigations, except one or two, use constant deflection tests to evaluate the fatigue performance of wood and wood laminates. However this method has its disadvantage. In a constant deflection test the specimen becomes more compliant over time and the load progressively decreases. It may happen eventually that the specimen will take infinite number of cycles without showing any failure due to fatigue. The constant stress loading, on the other hand, will ensure that the specimens are loaded, in each cycle, to a stress resembling the actual service conditions. Also this method allows simulating the stresses of a large span in-service girder through testing of a smaller span specimen in the laboratory.

Figure 2-9: Four Point Bending Test Setup

2.6.2 Stress Ratio

Stress ratio, R , is defined as the ratio of the minimum stress to maximum stress applied in each loading cycle of the fatigue test. Stress ratio may vary from $+1$ to -1 . A negative stress ratio implies that reverse loading is applied, which means the girder will buckle upwards. Hansen (1991) investigated the effect of stress ratio on the fatigue properties of Khaya Ivorensis laminates (Figure 2-10) and found that the fatigue strength decreases for reverse loading. It is highly unlikely that reverse loading will occur in a highway bridge. Hence, negative stress ratio was not considered in the current project.

2.6.3 Number of Load Application Cycles

Tsai and Ansell (1990) mentioned that wood and wood laminates, subjected to a test procedure where constant deflection is applied, show peak loads that decrease significantly with time. This can result in a test that can continue indefinitely without visible signs of failure, so the point at which fatigue test ends becomes arbitrary. It is commonly accepted that two million fatigue cycles of stress of a specimen is a good representation of the damage due to vehicular loads over the life span of a bridge girder. An in-service girder will experience far more than two million cycles of fatigue. However, most cycles do not stress the girder to its allowable design strength. This method has been employed by other fatigue researchers such

as Senthilnath et al (2001) and also by Fieder et al (2003). Davids and Richie (2003) also employed a two million cycles fatigue test for their FRP-glulam specimens. Also the two million of load cycle application is a standard for steel bridge design and is dictated in the Wood Handbook. Another approach to the determination of load application cycles is to consider the traffic volume experienced by the girder. There has been report of using even smaller number of load application cycles for relevant study. Geoffrey et al (2003) has applied 500,000 cycles of load on SCL panels connected to steel girders to study the reflective cracking of decks in an SCL-Steel composite bridge system.

Figure 2-10: Effect of Strength Ratio on Fatigue Strength of Khaya Ivorensis in Four Point Flexure (Ansell, 1987)

2.6.4 Loading Frequency

Fatigue loading frequency is defined as the number of load application cycles per second. It is based on the purpose of the investigation. Wood is a material with time dependent properties, and loading time must be taken into account. Figure 2-11 shows that

very low frequencies drastically affect the fatigue strength. For low frequencies, the loading condition becomes more similar to creep loading rather than fatigue. It may also be observed from the figure that frequencies of 5-40 Hz do not affect the fatigue strength. However, frequency range above 30 Hz should be avoided because some adiabatic heat is generated within the wood at this rate of loading, which reduces the moisture content and hence increases the fatigue strength. Loading frequency is also dictated by the experimental setup at hand. Since most of the loading mechanisms currently available in the laboratory test setup are powered by hydraulic pumps, the capacity of the pumps in most cases becomes the deciding parameter for this quantity. Researchers, however, are more inclined to setup a loading frequency of 1-2 Hz. Geoffrey et al (2003) has employed a loading frequency of 1 Hz.

Figure 2-11: Effect of Loading Frequency on Fatigue Strength of Wood (Bach, 1979)

2.6.5 Size of Specimen

The fatigue characteristics of the SCL T-beam girders will be affected by their sizes. The size of wood members has a significant effect on its strength. Small specimens typically demonstrate greater strengths. Obviously, the fatigue strength for the full-scale girders will be different from that of small pieces of SCL material. For the development of an S-N curve for a material, it is convenient to choose a smaller sized specimen to perform numerous amounts

of fatigue tests. Development of an S-N curve for a large scale built-up structural member is not feasible due to the enormous quantity of samples required.

2.6.6 Moisture Content

According to Hansen (1991), the moisture content has a significant influence on the characteristics of wood including the fatigue properties. Figure 2-12 shows that increasing moisture content result in a decrease in fatigue strength.

Figure 2-12: Effect of Moisture Content Fatigue Strength of Khaya Ivorensis Laminates in Four Point Flexure (Ansell, 1987)

2.6.7 Preservative

Lewis (1962) investigated the effect of coal-tar creosote preservative on fatigue strength of wood specimens. He found that treatment reduces the static properties. He also observed some reduction in fatigue resistance when it was plotted as a percentage of static strength. Most of the girders to be tested for the current investigation are treated with various preservatives. The variation of fatigue phenomenon due to different preservatives is, therefore, very important. Also, as a part of this investigation, the fatigue performance of some untreated and weathered SCL girders will be evaluated.

2.6.8 Grain Alignment

The effect of grain alignment in wood and wood laminates also plays a significant role in their fatigue behavior. Lewis (1962) compared the fatigue performance of straight-grained specimens and specimens with a 1:12 slope of grain. Straight-grained specimens were found to have slightly higher fatigue strength. The trend was reversed in green specimens. Air-dried straight-grained specimens showed compressive failures followed by final tensile failure, while specimens with 1:12 slope of grain usually failed in cross-grain tension without compression. According to Hansen (1991), there is a remarkable decrease in fatigue properties as the angle between the fiber directions for wood laminates and the beam axis increases. SCL, due to its manufacturing process, is free of any effect from grain alignments.

CHAPTER 3

SELECTION OF FATIGUE TEST PARAMETERS

3.1 Girder Dimension Determination

The current research was directed towards the fatigue behavior identification of full-scale T-beam girders of SCL bridge systems. However, these girders varied in their dimensions depending on the span length, lane width, construction techniques etc. Hence, to have a generalized idea about the fatigue performance of such categories of girders would have required the testing of a huge number of girders with different spans and cross-sectional dimensions. This was not practically feasible in terms of both time and financial constraints. The proper approach in this regard was to select a representative girder, which was most likely to be used commonly in real-life, in-service SCL bridge systems. This chapter describes the identification of such an SCL T-beam bridge girder with appropriate dimensions. Also, the fatigue test parameters and their relevant values that applied to the current research will be discussed in following sections.

3.1.1 Selection of Bridge Span

As stated before spans for SCL girders vary depending on their application and physical conditions at bridge site. Commercially available SCL girders, however, are limited by a maximum span length. This is due to the structural properties of wood and engineered wood products. Since wood have material properties lower than steel and concrete, girders made out of wood is limited by certain span lengths if they have to have a sizeable cross-section. In other words, it is possible to make girders with larger span length using wood; but in that case, the cross-section of the girder will be huge and unwieldy adding to the dead weight of the girder and requiring more sophisticated equipments for installation. Currently

Hughes Brothers Inc. of Seward Nebraska, which is the sole manufacturer of SCL T-beam bridge girders, produces these girders with a maximum span length of 60 ft.

For the current study a span length of 20 ft was selected. This length was primarily selected because of the space constraint in the Structures Laboratory at the FAMU-FSU College of Engineering. The selection of this span length was also justified by analyzing the statistical data of existing bridges as described in following sub-sections.

3.1.2 National Bridge Inventory

The National Bridge Inventory, or NBI, is a database for information on more than 600,000 bridges on the Nation's Interstate Highways, U.S. Highways, State and County Roads, and other routes of national significance. This database constitutes bridge information that State officials of each state submits annually to the Federal Highway Administration (FHWA). The submitted data conforms to the format of Federal Report No. FHWA-PD-96-001, titled "Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges" (Coding Guide). The objective of this database is to determine the number, location, general condition and improvement cost of bridges in the Inventory.

FHWA requires that all States conduct periodic inspections of each bridge in the Inventory, and report the condition of these structures in conformance with the Coding Guide by the month of March every year. The guide has been prepared for use by the States, Federal and other agencies in recording and coding the data elements that comprise the National Bridge Inventory Database. A complete and thorough inventory of the Nation's bridges enables FHWA to make an accurate report to Congress on the number and condition of the Nation's bridges. The database also provides the information necessary for the FHWA and the Military Traffic Management Command to identify and classify the Strategic Highway Corridor Network (STRAHNET) and its connectors for defense purposes.

The NBI database can be obtained from FHWA on request. The database is divided into text files for each state which contain all the bridge status of the corresponding state. The database is distributed as a collection of generalized text files so that different researchers will

be able to integrate the information into their different computing environments. Each text file corresponding to a state comprises of a 432 character long text string for each bridge within the state. Each string contains coded information on over 100 individual items of information pertaining to each bridge on file, including location, structural appraisal, sufficiency rating, age, functional classification, etc. A complete list of these items as obtained from NBI 2004 is provided in Appendix A. Many States employ the use of a mainframe computer to generate the file through an edit/update program which compiles the data into the required format, while others use a Bridge Management System (BMS), to produce the file. After the file is submitted to FHWA, the data is subjected to post-processing to allow FHWA to extract the required information. The data is used to determine the number and condition of the Nation's bridges, and to determine funding eligibility for the States.

National Bridge Inventory is an invaluable source of information as it contains all the structural and physical parameters of bridges currently in-service throughout the United States. It also contains fields of information for individual bridge that quantifies the current and future traffic volume in terms of Annual Daily Truck Traffic (ADTT). All these information can serve as an important tool in the selection of bridge parameters for the current fatigue study. Consequently a copy of the NBI 2004 was obtained and the database text files for all the states were merged to form a single list for the whole country. This list was then fed to the Relational Database manipulation application, Microsoft Access. Various query tools were then applied to the database to produce different sub-tables that provide list of bridges specific to certain criteria.

3.1.3 Selection of Representative Bridge Span

As described earlier the span length of girder specimens for this study was limited to 20 ft for practical constraints. However an overview of the wood bridges within the NBI reveals that the span length of wooden bridges may vary from 10 feet to more than 100 feet. Figure 3-1 shows the distribution of the span lengths of all the bridges in US that are of wood construction. This distribution is based on the sub-list for the wooden bridges that was extracted from the master bridge list in NBI database. The criteria that were used in selecting the bridges were:

1. For single span bridges, the span length was considered; for multi-span bridges, the span length of the largest span was considered.
2. Bridges subjected to HS20, modified HS20 and HS25 were selected.
3. Highway bridges with and without pedestrian traffic, overpass structures at an interchange or second, third or fourth level of a multilevel interchange and other were considered.

Figure 3-1: Distribution of Span Length in Wooden Bridges

It was evident from the distribution that the average span length of such bridges was near 25 ft. A more accurate statistical analysis revealed that the average span was 26.46 ft with a standard deviation of 14.18 ft. The summation of these two measures produced a span length of 40.64 ft. Sources at Hughes Brothers also expressed that the average span length of SCL T-beam girders that they produced at that time was about 45ft.

The selected span length of 20 ft was close to the average span length of 26.46 ft. However if the spread of the distribution was considered the highest span length turned out to be significant was 40.64 ft. The prefabricated SCL girders of the bridge system being

discussed are generally installed as simply supported spans. It is known from the structural analysis stand point that the moment on a simply supported bridge girder is proportional to the square of its span length. Hence, due to the same wheel load configuration, a girder with span of 40.64 ft is likely to produce larger moment than that of a girder with 26.46 ft span. And since larger moment is expected to produce higher stresses within the SCL material, it was a rational choice to select 40.64 ft as the representative span length. This length was, however, much more than the selected span length of 20 ft. One approach to overcome this predicament was to simulate the truck load stresses that were induced in the 40.64 ft span girder on to the 20 ft girder. In other words, the stresses in a 40.64 ft bridge girder could be calculated due to certain wheel loadings; and then, loads could be applied in a laboratory setting on the 20 ft girder that produced the same level of stress. Such applied loads would be very excessive for the girder in addition to the fact that AASHTO LRFD truck loading was used when generally service loads are used to design wooden bridges. This conservative approach was thus adopted within the current research. The representative bridge span length was thus chosen to be 40 ft.

3.1.4 Determination of Girder Cross-Section

Hughes Brothers produces the prefabricated SCL girders in two shapes, box and T-beam configuration. While the box-beams are used as edge girders, the majority of the traffic load is carried out by the T-beams. Current research, therefore, was geared toward the study of the fatigue behavior of these T-beam girders. The cross-sectional dimensions of these girders are, however, pre-determined by Hughes Brothers. Currently, they sell T-beam girders up to 65 ft long with two sets of varying beam depths. The two sets of beam depths originate from the different moment of inertia requirements for HS-20 and HS-25 truck (Figure 3-2) loading. HS-25 truck has the same configuration as the HS-20 truck load except all the wheel loads are increased by 25%. HS-25 trucks, therefore, produces larger bending stresses which justifies the larger beam depth for these trucks. The depth of the beams also varies with different span lengths. This is attributed to the fact that larger span lengths induce larger bending moments which require correspondingly larger moment of inertias of the girders. Figure 3-3 shows the cross-sectional dimensions of the 20 ft and 40 ft long SCL T-beam Bridge girders.

Figure 3-2: Load Configuration of HS-20 and HS-25 Truck

Figure 3-3: Cross Section of SCL T-beam Bridge Girders

3.2 Loads on Bridge Girder

Wooden structures as a whole are designed on the premise of Allowable Stress Design (ASD) criteria. This is true due to the lower strength of wood material and the lack of uniformity of material properties due to strength reducing effects like, joints, knots, warping, insect attack etc. Although SCL material is much superior in terms of strength characteristics and strength uniformity than regular sawn lumber, it is also employed in structural applications following ASD formulations. However, the designer has the flexibility of choosing higher loads for a structure to be made using SCL material. The structural engineer may choose to maximize the load that is to be applied on the structure by using Load and Resistance Factor Design (LRFD). The LRFD method is supposed to produce much higher loads than the allowable stress design method. Once the maximum possible load is calculated, the designer can employ ASD method to design the SCL structure. This technique has the advantage of increased factor of safety over the traditional full featured allowable stress design. The current fatigue research acknowledged this idea. Therefore, for fatigue purposes, loads on the test girders were calculated using LRFD method. This ensured maximum possible stress on the SCL T-beam specimens.

3.2.1 AASHTO LRFD Loading

Currently, bridge design in the United States is dictated by the *AASHTO LRFD Bridge Design Specifications*. This is a comprehensive design guide that provides in-depth information on load calculation, design and maintenance of bridges made with concrete, steel, wood and other materials. According to the AASHTO specification live load on all bridges must conform to HL-93 loading. The HL-93 loading comprises of the following.

- An HS-20 truck, as shown in Figure 3-2
- A tandem truck that comprises of two axles of 25,000 lbs separated by a distance of 4 ft.
- A fatigue truck, which has the same loading as an HS-20 truck except the two rear axles are separated by a distance of 30 ft instead of 14 ft.

- A lane load of 640 lb/ft along the span, which is distributed in the transverse direction within a width of 10 ft

The HL-93 loading further specifies that different load cases have to be developed to select the appropriate live load on the bridge. Impact factors have to be applied to different truck loads and combined with the lane load. The impact factor is 1.33 for HS-20 and tandem truck; while for fatigue truck the value is 1.15. The governing live load will be the maximum of the following three load combinations.

- $(1.33 \times \text{HS-20 Truck Load}) + \text{Lane Load}$
- $(1.33 \times \text{Tandem Truck Load}) + \text{Lane Load}$
- $(1.15 \times \text{Fatigue Truck Load}) + \text{Lane Load}$

According to AASHTO LRFD specification, the dead load of the bridge will be calculated based on the self weight of the girder and deck system. The dead load will also constitute the roadway, sidewalks, guard rail, public utility services etc.

3.2.2 Load Calculation for Representative Girder

It was discussed previously that a representative bridge-span of 40 ft was used for stress calculation purposes. To achieve this, a hypothetical two lane bridge made out of 40 ft SCL T-beams was considered. The maximum live load moment on this beam was calculated using the AASHTO HL-93 loading. However, a slight modification was applied to the HL-93 loading as the HS-20 truck was replaced by an HS-25 truck. This was done due to the fact that for shorter spans with simple support construction, a single truck produces the maximum live load moment. And replacing the HS-20 with HS-25 ensured a higher moment value which was a more conservative estimate. Appendix B.1 shows the live load moment calculation in more detail. It is noted that the HS-25 truck combined with impact factor and the lane load produced the maximum live load moment on the bridge. Appendix B.2 shows the calculation of dead load moment of a single 40 ft girder. Proper distribution factor was applied to the live load moment to find out the effective live load moment on a single SCL T-beam girder. This value combined with the dead load produced the total LRFD moment on a girder which was

182.41 kip-ft (Appendix B-3). Maximum flexural stress due to this moment was produced at the bottom of the 40 ft girder. The maximum stress was found to be 2313 psi (Appendix B-4). This value was much smaller than the allowable stress of 3255 psi for the SCL material as published by the manufacturer. Also considering only the dead load moment, the minimum stress on the girder was estimated to be 195 psi. This was done on the premise that the minimum stress was obtained due to dead load only as it acted always; while the maximum stress only occurred due to dead and live load when a truck passed on the bridge.

3.2.3 Load Calculation for Test Girders

The maximum flexural stress obtained within the representative 40 ft span girder needed to be simulated in the 20 ft test specimens. The detailed calculation is shown in Appendix B.5. Due to its smaller length the 20 ft span girder had smaller dead load moment than the 40 ft span girder. The initial loading at each cycle of fatigue loading on the representative bridge was zero. This implies to the condition when there was no truck on the bridge. However to simulate the dead load stress of the representative bridge on the test girder, some additional loading must be applied. Calculation in Appendix B.5 suggests that an initial loading of 0.8 kip must be applied to simulate the no live load situation on the 40 ft bridge. This load had been calculated on the premise that the test girders would be subjected to four point loading tests and the distance of load head from each end of the girder was 8ft as shown in Figure 3-4. Using the same loading setup, the applied load required for simulating the full stress due to dead and live load on the representative bridge was 17.28 kip. The maximum deflection corresponding to the full live load and dead stress on the 20 ft girder was 1.26 in. This has been calculated using the manufacturer's published modulus of elasticity value of 2,000,000 psi.

Figure 3-4: Four Point Loading Setup with Dimensions

3.2.4 Stress Ratio for Test Girder

The stress ratio for the representative girder is the ratio of the initial stress to the final stress in each cycle of loading. For the in service girder, initial loading was only due to the self weight of the girder when no truck was on the bridge. The final load corresponded to a stress which is caused by both the dead load and the truck. Thus, for this girder the stress ratio was the ratio of dead load stress to the maximum stress. This value had been calculated to be 0.084. The stress ratio for the 20 ft test girder was also kept the same as the underlying concept of the test was to simulate the exact initial and final stresses in the representative girder. This must be noted here that a conventional fatigue test of a material involves repeatedly loading the material at different stress ratios and thereby producing a typical $S-N$ curve. For the current study, varying the stress ratio means variation of the live load on the representative girder. The load on this girder can only be varied by changing vehicle types. However, the truck load causes the most damage on any highway structure. Hence, variation of stress ratio may not produce the most damaging effect on the girders. Also, to identify the effect of stress variation, numerous number of test specimens must be subjected to fatigue, which was beyond the scope of this project. Therefore, the current tests were performed at a fixed stress ratio and hence eliminated the notion of producing $S-N$ curves for SCL T-beam bridge girders.

3.3 Number of Load Cycles Determination

It was proposed previously that the SCL girders would be subjected to two million cycles of load application cycles. This was decided following the works of Senthilnath et al (2001), Fieder et al (2003), Davids and Richie (2003) and the recommendations of steel bridge design procedure and wood handbook. AASHTO LRFD design method for bridges, however, does not specify any number of cycles for fatigue testing of bridge girders. For fatigue purposes, AASHTO suggests the inclusion of a fatigue truck during the evaluation of various load cases. The fatigue truck is similar to an HS20 truck with the rear two axles separated by a distance of 30 ft. Current analysis for stress included the fatigue truck. However, for fatigue testing purposes, a certain value must be determined to represent the number of load application cycles.

The most obvious approach to this problem was to directly identify the number of load cycles from actual existing wood bridges. Annual Daily Truck Traffic (ADTT) data was employed in this regard. Although, it may be argued that a bridge can expect smaller vehicles other than the heavy trucks, it is a safe option to consider the trucks only, which apply the most amounts of stresses on the girders. The ADTT data for all wooden bridges were readily available in NBI. It may be questioned whether it is a good idea to predict the future traffic on the bridge based on the current ADTT data. This problem was overcome by including projected future ADTT data within the calculation that was also available in the NBI.

For this investigation, the ADTT data for the current and future time was extracted from the NBI for existing wood bridges. Figure 3-5 shows the distribution of complete ADTT data from 2214 bridges selected using selection criteria described earlier. It is clear from the figure that the ADTT data for most of the bridges were substantially low. Figure 3-6 further demonstrates this fact and shows that most of the ADTT figures were within a range of zero to 25 which represented low volume rural traffic. Based on the current ADTT data, a mean of 44.15 was obtained. Comparing the current and future ADTT data, the rate of increase per year was calculated. The average of all the rates was estimated to be 0.41%. Based on these values the expected truck traffic on the girders for next 60 years was calculated to be 1093667. It is evident that this value is almost half of the previously proposed value of two

million. However, this can be justified considering the fact that while two million cycles is acceptable for steel and other stronger material, the calculated value was suitable for a relatively less stiff material as SCL. The project period of sixty years was obtained from life expectancy of SCL material as published in the manufacturer's website.

Figure 3-5: Complete Distribution of Current ADTT Data

Figure 3-6: Partial Distribution of Current ADTT Data

3.4 Loading Frequency Determination

Study of the previous researches has revealed that 1-5 Hz is a safe loading frequency for fatigue study of wood products. At this rate there is no possibility of moisture dissipation due to heat generation. Also this rate is fast enough so that the loading will not produce a creep effect within the specimen. For the current fatigue study of SCL T-beams a loading rate of 0.9 Hz was selected. At this rate the loading in each cycle was not considered to be sustained long enough to produce creep effects. The prime factor behind the selection of this loading rate was the capacity of the Hydraulic pump running the loading actuator at the FAMU-FSU College of Engineering Structures Lab. The pump had an operational rating of 21 gallons per minute. Also the fluid flow through the pump was dictated by the deflection of the specimen. Higher deflections required higher volumes of oil flow through the pump. If this exceeded the capacity of the pump certain peak loads were not achieved.

A few trial runs of the testing procedure were made to observe the performance of the hydraulic pump. From these trials it was evident that the pump was capable of comfortably producing the fatigue loadings at the rate of 0.9 Hz. An oscilloscope connected with the feedback device of the actuator produced perfect sinusoidal loading curves with well defined

peaks and values of the applied loads. It should be noted that running continuously at 0.9 Hz, time required to complete one million cycles of fatigue loading was about 12 days 21 hours.

3.5 Consideration of Other Parameters

Grain alignment in wooden structures controls fatigue strength. Sloping of grain alignment was found to reduce the failure strength due to fatigue. SCL is, however, manufactured out of wood veneers or strips with grain direction perfectly horizontal and aligned with the length of the specimen. Hence, there was no chance of fatigue strength variation in SCL girders due to the alignment of wood grains.

3.5.1 Moisture Content of Test Specimens

The current research on fatigue behavior analysis of SCL girders did not take into account the variation of moisture content of the specimens. However, during the cyclic loading of the specimens, their moisture content data were recorded for documentation purposes. It may be mentioned that the moisture content in LVL and PSL members typically range between 10-12% as they come out of the factory. Also the moisture content in an in-service SCL bridge girder may be significantly higher. While evaluating the performance of an existing bridge, Ritter, Lee, Kainz and Meyer (1996) found that the moisture content of LVL T-beam bridge girders increased up to 26% in two and a half years. Also SCL is expected to swell to some extent with an increase in moisture content. Since the SCL bridges are stress-laminated, the swelling in the SCL becomes beneficial as they help to maintain the post tensioning forces over time in the transverse tendons.

3.5.2 Selection of Preservative Treatment

Previous research has indicated that the static properties of preservative treated wood and wood laminates tend to decrease with fatigue. However, untreated SCL material, when used outdoors as transportation structures is susceptible to adverse environmental hazards like termite and other insect attacks. Hence, T-beam bridge girders made out of LVL and PSL must be treated thoroughly before they are installed. Current research studied the effect of fatigue on the following two different types of preservatives.

- Penta ChloroPhenol
- Copper Naphthenate

These preservatives are mostly used for outdoor applications. Most of the preservatives for exterior structures are copper based. Previously CCA was a major preservative that was used in SCL girders. This material, however, is being phased out for environmental concerns.

CHAPTER 4

POST TENSION IN SCL GIRDERS

4.1 Post Tension

A typical SCL bridge cross section consists of a number of SCL T-beam bridge girders placed side by side with two box girders on each end. All the girders thus placed are then post tensioned in the transverse direction using high strength steel bars with proper anchoring and produces a stress laminated deck system which acts as a single structural element. Within the current research the primary notion was to fatigue the specimens without the post tension applied. However, initial fatigue load applied to such an un-tensioned LVL girder revealed that severe de-lamination occurs at the SCL-epoxy interface. This de-lamination starts to occur just after application of only 300-400 cycles. Figure 4-1 shows such separation within the LVL beam cross-section. It is evident that these separations occurred starting from the web and all through the flange. Also such separation propagated through the entire length of the beam (Figure 4-2). This de-lamination phenomenon led to the decision that the girders needed to be post tensioned before they are fatigued. This also helps the fact that an in-service girder will never be stressed by traffic loading without being post tensioned and hence applying post tension to the specimens emulated the actual loading condition of a girder being used in a vehicular bridge.

4.2 Calculation of Post Tension

Before the girders could be stressed in the transverse direction a post tensioning force must be calculated. The post tensioning force depends on various geometric parameters of the bridge such as span length, deck width and deck thickness. It also depends on material properties such as compressive strength, modulus of elasticity, shear modulus, moisture

Figure 4-1: Severe De-lamination in the Web for the lack of Post Tension

Figure 4-2: Crack Propagation at SCL-Epoxy Interface along the length of Girder

content of wood material and tensile and bending stress of post tensioning material. For the current research, a 25 ft wide, two lane bridge deck was considered. The deck thickness was taken as 6 inch in accordance with the flange thickness of the SCL specimens obtained. SCL material properties were used as much as possible. In the absence of some of the material properties for SCL, the corresponding values for the constituent wood, which is Douglas Fir, were used. For post tensioning material, steel was considered. Dywidag bars were actually used as the post tensioning rod. These are very commonly used in wooden bridges for post tensioning and are named after the manufacturing company Dywidag-systems International. The Dywidag bars were made of high strength steel bars having an ultimate tensile strength of 150,000 psi. The post tension rod needed to be anchored properly. This was achieved using one base plate, one anchor plate and a nut on each end of the bar. These elements were made of regular stainless steel.

Post tension force was calculated following the procedures described in USDA Timber Bridge Design Manual. Details of the calculation are given in Appendix C. Based on the calculation, a 5/8 inch diameter Dywidag bar was selected with a post tensioning force of approximately 20,000 lbs. A 5/8 in diameter bar was used because it fit the hole that had been pre-drilled in these SCL girders as they came out of the manufacturing plant. For the base plate a 5 in $\times$ 6 in steel plate with 1/2 in thickness was chosen. This base plate distributes the compressive force from the post tension bar over a wide area of the SCL material and prevents localized crushing of the material. A 3 in $\times$ 3 in anchor plate with 3/4 in thickness was also chosen. Finally, for anchor nuts, some ready made nuts supplied by Dywidag-systems were used.

4.3 Post Tensioning Procedure

For applying load on the Dywidag bars a small actuator connected with a hand operated hydraulic pump was used as shown in Figure 4-3. Same figure also shows a stainless steel device that was used in conjunction with the loading actuator. To apply the post tension, the Dywidag bar was placed in the pre-built hole in the flanges. The bar was then attached to the anchor nut placed on top of the anchor plate and base plate at one end. On the other end the base plate and anchor plate were then placed and the anchor nut was tightened. The steel

device having slotted sides and the actuator were then placed as shown in Figure 4-4. On top of the actuator another anchor nut was placed which stretched by the Dywidag bar as the actuator exerted pressure on it. A pressure gauge connected to the hydraulic pump helped to determine the amount of force applied. Once the required force was attained the nut on top of the anchor plate was tightened by accessing it through the slotted sides of the steel cylinder. After it was properly tightened, hydraulic pressure from the actuator was withdrawn. At this point the Dywidag bar retracted and produced appropriate post tension with the help of two sets of base plates, anchor plates and nuts. Figure 4-5 shows these elements in detail.

Figure 4-3: Hydraulic Pump, Actuator and ancillaries for Post Tensioning

Figure 4-4: Application of Post Tension

Figure 4-5: Post Tension System - Dywidag Bar, Base & Anchor Plate and locking Nut

4.4 Loss of Post Tension

It was also expected that the post-tensioning applied would gradually decrease due to repetitive truck traffic loading of the SCL girders. It was, therefore, beneficial to understand how much post-tension the SCL girders lost due to repetitive loading over a long period of time. To estimate the loss due to fatigue loading, two load cells were used. These load cells were connected to the post-tensioning bars to measure the existing load in them. One of the load cells was placed on the post-tensioning bar near the support while the other was placed at mid-span. At the onset of fatigue testing, the readings on the load cells were noted which indicated the initial post-tension applied. Roughly after every 100,000 cycles, the post tensioning loads in the bars were recorded. At the end of fatigue testing, the load cell readings were noted once again. A plot of these recorded values provided an estimation of the loss of post-tension due to fatigue. Since the fatigue testing for one beam lasted 2-3 weeks, there was virtually no chance of time dependent losses being incorporated within the results. Figure 4-6 shows the location of post tension bars where the load cells were placed.

Figure 4-6: Locations of Load Cells for measuring Post Tension

CHAPTER 5

TEST SETUP AND PROCEDURE

5.1 Test matrix for SCL Girders

Commercially available SCL girders vary in their constituent materials as well as the types of preservatives based on their applications. Currently Hughes Brothers of Seward, Nebraska produces SCL T-beam girders made out of both LVL and PSL. To prevent insect attack and other environmental hazards the company sells these girders pre-treated with different preservatives. The preservatives vary depending on indoor and outdoor usages of the girders. For indoor use mostly water based preservatives are used while copper and oil based preservatives are preferred for outdoor applications. In recent years, leaching of potentially dangerous preservatives into the environment has become a major concern. Consequently, application of such harmful preservatives like CCA has been banned from the production process of pre-built SCL girders. During the current study, attempt was made to cover the whole range of SCL girders as made by the manufacturer. Based on this, twelve beams were selected for testing. Table 5-1 describes the test matrix in detail. Two material types and two kinds of preservatives were considered. One specimen of each type was chosen to be tested statically up to failure while two specimens were tested in fatigue. In total eight SCL girders were loaded in fatigue while a total four girders were only loaded up to their ultimate strength. As described in subsequent sections, once the fatigue loading had completed, the surviving girders were also loaded statically up to failure. The small number of sample size was attributed to the high cost of these pre-built girders.

Table 5-1: Test Matrix for SCL Girders

SCL Type	Preservative Type	Specimen Name	Type of Test
LVL	Penta Chlorophenol	LVL P1	Static
		LVL P2	Fatigue
		LVL P3	Fatigue
	Copper Naphthenate	LVL C1	Static
		LVL C2	Fatigue
		LVL C3	Fatigue
PSL	Penta Chlorophenol	PSL P1	Static
		PSL P2	Fatigue
		PSL P3	Fatigue
	Copper Naphthenate	PSL C1	Static
		PSL C2	Fatigue
		PSL C3	Fatigue

5.2 Four Point Loading Setup

All the fatigue and static tests on the SCL girders were carried out using a four point loading setup. Using this setup each specimen was simply supported at two ends. The load from the actuator was equally divided and applied at two locations on the specimen using a header beam. The header beam used in the test was chosen after proper deflection calculation. It was ensured that the steel section would not undergo large deflection due to the expected load and thereby would not influence the overall deflection of the SCL girder under heavy load. Two pieces of 2 in. by 1 in. steel tubing were attached at the end of the steel section which ensured that the loads were applied at two points on the specimen. Each of the tubing was 2 ft in length and spans through the whole width of the specimen. The steel tubings were

placed four feet apart on centers. Figure 5-1 shows the header beam attached with the steel tubings.

Figure 5-1: Four Point Loading Test Setup

The purpose of using the four point bending setup was to establish a constant moment region within the 4 ft region between the tubing. This is a very widely used method for testing specimens in flexure. This test setup also ensured that within the constant moment region there was no shear force and the specimen was being subjected to pure bending.

The specimens were placed on blocks on both ends. These blocks were made of 2 X 2 X 4 ft hollow box made out of steel and poured completely with concrete. These blocks were obtained from FDOT Structures Laboratory. Two blocks of this kind were placed on each end of the specimen. The specimens were placed on bearing pads resting on these blocks. Two bearing pads were used on each end having dimension of 8 in×12 in ×2 in.

5.3 Loading Apparatus

The tests were performed using the loading actuator available in the Structures Laboratory at FAMU-FSU College of Engineering. This actuator was built by MTS and had a loading capacity of 110 kips. The actuator was connected to a steel loading frame. The loading frame, made out of steel, is firm enough so that there was no additional deflection from the frame. The actuator had a closed loop system and was run by a 21 gpm hydraulic

pump. A 407 controller was used to setup the actuator as a load controlled device. The 407 controller had also built in sinusoidal loading function generator which was implemented to apply fatigue loading on the specimen. During static loading of the specimens up to failure, ramp loading function of the 407 controller was used. The controller also had automatic safety features which enabled the loading to stop in some unforeseen situations and thereby saved the specimen and avoided disastrous accidents. As mentioned earlier, the whole system was a close looped one where the 407 controller sent a command to the actuator and the actuator responded with a feedback to the controller. For example, if the controller sent a signal to the actuator to apply a load of 50 kips, upon attaining this load the actuator sent a feedback of 50 kips. In case that the difference between the command and feedback exceeded by certain value the controller was capable of shutting down the system. This situation might arise when the specimen had already been broken and the actuator could not apply the high load that the controller was sending as a command. Figure 5-2 shows the 407 controller.

Figure 5-2: Loading Actuator Operating Device – 407 Controller

5.4 Data Acquisition Equipment

In addition to controlling the actuator, the 407 controller also output some feedbacks from the actuator. It was capable of producing the load and deflection information from the actuator available at some of its output ports. These outputs were then fed into a LabJack data acquisition equipment (Figure 5-3). This was a 12 bit data acquisition system which was capable of recording data from 4 channels simultaneously through the USB port of a desktop computer. In addition to the load and deflection data for all the specimens, three strain data were obtained during static testing of some of the specimens. Consequently, the current research had to utilize two LabJacks. These devices were connected to a personal computer which recorded the data. Data was recorded at a rate of 4 samples per second which was considered more than adequate for the current testing purposes. A National Instrument Signal Conditioner (Figure 5-4) was also used to monitor strain gages. To measure the loss of post tension, a strain gauge box (Figure 5-5) made by Vishay Micromerement was used.

Figure 5-3: LabJack Data Acquisition System

Figure 5-4: Signal Conditioner from National Instruments

Figure 5-5: Strain Gauge Box from Vishay Micromeritics

5.5 Fatigue Loading

For fatigue loading the specimens were placed on bearing pads sitting on concrete blocks. Initially it was assumed that the 20 ft span girders would be loaded in fatigue to simulate the stresses on the 40 ft girder due to truck traffic. Since the beam was supported on 1 ft length of bearing pad, the effective span of the girder was 19 ft considering the distance between centers of supports. Based on this estimation the actuator must produce a moment of 70.267 kip-ft at the middle portion of the beam. With a 19 ft span the required load in this case was 17.28 kip and the maximum deflection was 1.259 in. However, this deflection was difficult to achieve when fatigue loading was applied at a relatively high rate of 1-2 Hz. One way to circumvent this situation was to reduce the frequency which would have required a much longer period of time to apply fatigue loading on each girder. The other alternative to reducing the frequency was to minimize the deflection. Now, the deflection of a simply supported beam varies proportionately to the square of the span length. Hence, reducing the span length will ultimately result in a lesser amount of deflection. The span length of the SCL girder specimen was thus reduced to 16 ft. Even though the span length was reduced, the applied moment had to be kept constant to attain the same stress level. This required that the applied load by the actuator had to be increased. Corresponding to the 16 ft span length the required load was 23.04 kip and the calculated deflection was 0.832 in. The frequency of the sinusoidal loading was kept at 0.9 Hz. At this rate the time required for each loading and unloading cycle was 1.11 second. According to calculation performed, considering the minimum length of an HS-25 truck and maximum speed of 60 mph for the truck traffic, it would take 1.10 second for a truck to cross the design girder. This calculation also assumed that the minimum distance between two consecutive trucks was the same as the length of an individual truck. Fatigue loading frequency of 0.9 Hz for all the girders was thus justified. At this rate, it would take approximately 12 days and 21 hours to apply one million cycles of fatigue loading on each specimen. In reality, though, it took more time than that because of intermittent stoppage of the hydraulic system due to voltage spikes and because of undergraduate laboratory classes held at the facility.

5.6 Static Loading

Surviving girders after fatigue loading were tested in static loading. The idea behind this was to find out the ultimate strengths of these girders after they had been fatigued. Also, some of the fresh girders were loaded statically to find out the ultimate strengths of fresh girders. Any static loading of the girders, therefore, required loading the specimens up to high load levels until they broke. The same four point bending setup was used for this purpose. However, much larger moment was required for each beam at the mid-section to fail it completely. For fatigue loading, the girders were placed on the concrete blocks with an effective span length of 16 ft. If the girders were to be statically loaded with the same effective span, the actuator needed to apply a very large amount of load to produce the large moment. Within a four point bending test setup, the moment produced by the actuator at the mid-span is a function of actuator load times the distance between the span and the first end of the header beam. When this distance is decreased, actuator load must be increased to produce the high moment. In fact, placing one of the specimens at 16 ft effective span, it was observed that the beam did not show any sign of failure at actuator load levels of 70 kips. Since the actuator at the laboratory had a maximum capacity of 110 kips, it was evident that with this setting, the ultimate load would be very close to or beyond the actuator capacity. To avoid this kind of scenario, it was decided that the effective span of the girders under static loading will be increased to provide a larger moment arm and hence produce a large moment at lower actuator load levels. For static loading, the girders were spaced at 19 ft effective span.

5.7 Data Collection Schedule

Test data for the SCL specimens were collected at regular intervals. The data collection procedure for the experiments can be divided into two categories.

5.7.1 Fatigue Test Data Collection

At the onset of fatigue testing of individual specimen, it was first loaded in static up to the level of peak load during each fatigue cycle. Thus the beam was loaded up to a static load of 23.4 kips. During this loading load data from actuator and the corresponding deflection at mid-span were recorded using the data acquisition system. Since these data come directly

from the 407 controller, no signal conditioner was used in between. Also data were collected at a rate of 4 samples per second. Once the applied load had reached the level of 23.4 kips, recording of data was stopped and the girder was then unloaded. This was done to obtain an estimation of the load-deflection characteristics and especially the modulus of elasticity of the girder before it experienced fatigue cycles.

At around every 100,000 cycles, the fatigue loading was stopped momentarily. The girders were then unloaded. After unloading, each of them was again statically loaded up to the peak load level of 23.04 kips. During this process continuous load-deflection data were recorded again. The purpose of this exercise was to evaluate the deterioration of modulus of elasticity if any due to fatigue loading. At the end of static loading the beam was again unloaded and the fatigue loading resumed. This process was continued for every 100,000 cycles up to 1 million.

Also during this momentary stoppage of fatigue loading two more data sets were recorded. Using a strain gauge box, readings from the two load cells attached to the Dywidag bars were collected. This was done when the beam was in an unloaded condition. These two readings helped to identify the loss of post tension in the SCL girders due to fatigue loading.

5.7.2 Static Loading Data Collection

During static loading, the fresh beams and the fatigued beams were loaded up to failure. During this loading the actuator load and deflection data were collected continuously. Most of the beams broke within ten minutes of the initiation of the static loading. Towards the end of the current research it was decided that strain gages would be attached to the fatigued beam while they were loaded up to failure. The strain gages were placed at the mid-span of the beam at three different locations as shown in Figure 5-6. Two of the strain gages measured tensile strains while the third one measured compressive strain. As mentioned before a signal conditioner was used in between them and the data acquisition equipment. The strain gage data were collected simultaneously along with the load-deflection data. The collected strain data was useful in validating finite element models of the SCL girders.

Figure 5-6: Location of Strain Gauges during Static Loading

CHAPTER 6

PERFORMANCE OF SCL GIRDERS

6.1 Performance of Fresh Girders

As per the test matrix described in chapter 5, four fresh beams were loaded statically to determine the ultimate breaking load of these girders. These girders were broken at different times over a period of one and a half years. Within this time they were stored in a field covered with plastic sheeting. Following subsections describe the behavior of these fresh girders under static loading.

6.1.1 Ultimate Failure Load

The statically loaded four beams varied in the SCL types. The ultimate breaking strengths for the four beams were different from each other. Table 6-1 summarizes the breaking strengths. It is evident that except the Copper Naphthenate treated LVL, the ultimate strengths of all other SCL girders are very similar.

Table 6-1: Ultimate Failure Strengths of Fresh Girders

SCL Type	Preservative Type	Specimen Name	Ultimate Strength (kips)
LVL	Penta Chlorophenol	LVLP1	37.94
LVL	Copper Naphthenate	LVLC1	60.75
PSL	Penta Chlorophenol	PSLP1	63.83
PSL	Copper Naphthenate	PSLC1	56.81

The largest ultimate strength value was obtained for a PSL beam. The reason for such result may be attributed to two facts. Firstly, owing to the manufacturing process PSL material seem to be denser than the LVL. Also PSL is more homogenous in nature because of the random packing of veneers compared to oriented lamination of LVL. Therefore, when the beams were loaded to high static load, the PSL material provided larger strength because of adjacent interlocking of material and better stress distribution. SCL manufacturer's published strength properties also corroborate this fact. Secondly, the higher strength of PSL beam was dictated by the manufacturing technique of the girder itself. As described in chapter 3, the LVL T-beam cross section was made up of a number of small pieces epoxied together where the web of the LVL beam itself comprised of three different parts. The PSL cross section, however, was made up of only three pieces (a solid web and two side flanges). As the beams were stored outside, they experienced lot of sunshine, rain and cold weather during their storage period prior to testing. Although the girders were covered with plastic sheeting, these environmental hazards had their marks on these girders. Most noticeably, the epoxy in between the SCL pieces within the web of LVL girders gave away very easily. This produced large cracks at the bottom of the web which were visible all through the length of the girders. These cracks severely deteriorated the strength of the LVL girders. Especially for the LVL girder treated with Copper Naphthenate this kind of environmental damage was extreme and consequently the beam broke at a relatively low ultimate load of 37.94 kips.

6.1.2 Constitutive Relationship

Figure 6-1 shows the load vs. deflection plot for Penta treated fresh PSL girder that was continuously loaded up to failure. This is a typical load-deflection plot for all fresh girders. Irrespective of their constituent SCL type, preservative treatment and damage characteristics due to environmental agents, all the fresh girders exhibited similar linear relationship between applied loads and displacements. This implied that the both LVL and PSL girders possessed linear elastic material properties up to failure. Although non linearity in wood is not prominent, the process of grading, removing defects, grain alignment, compacting and application of epoxy within the manufacturing process made the material

more homogenous so that it attained a linear constitutive relationship. Appendix D.1 shows the load vs. deflection plot for all fresh girders.

Figure 6-1: Typical Load Vs. Deflection Plot of Fresh SCL Girders

6.1.3 Flexural Stress and Modulus of Elasticity

Table 6-2 shows a list of ultimate stresses and moduli of elasticity of the girders. The ultimate flexural stresses for both LVL and PSL beams were much higher than the manufacturer's published allowable strength values of 2750 psi and 2900 psi respectively. The table also shows that except for the LVL girder with severe cracks, all other girders had a factor of safety more than 2 based on the advertised allowable flexural stress. The corresponding moduli of elasticity of the fresh girders are also presented in Table 6-2. The calculation of the modulus of elasticity was straight forward because of the linear behavior of the SCL materials. Equation 6-1 shows the formula that was used to calculate the modulus of elasticity from the load-deflection plot. Corresponding terms in the formula can be understood following the beam configuration as shown in Figure 6-2.

Figure 6-2: Loading Arrangement for MOE Calculation

$$E = \frac{0.5Pa(4a^2 - 3L^2)}{24I\Delta_{midspan}} \quad \text{Equation 6-1}$$

where,   E = Modulus of elasticity
                  P = Load applied
                  a = 7.5 ft
                  L = 19 ft
                  I = 3730 in⁴
                  Δ_{midspan} = Deflection at midspan

The modulus of elasticity varied from 1.482×10⁶ psi to 1.865×10⁶ psi. Also noticeable is that the moduli for PSL beams were slightly higher than those of LVL beams. This again proved the better compactness of the PSL material and gave it an edge over LVL material. The producer of SCL, Trusjoist company, markets LVL and PSL as “Microllam 2.0E” and “Parallam 2.0E” respectively. The term 2.0E stands for the modulus of elasticity of these materials which is advertised to be 2.0×10⁶ psi. Static failure tests of both LVL and PSL beams revealed that the longitudinal flexural moduli values are less than 2.0×10⁶ psi. In case of the LVL beam with Copper Naphthenate treatment, the modulus of elasticity was almost 25% less than the rated value. This implied that with physical deterioration of these girders due to environmental hazards or other damaging effects might lead to loss of modulus of elasticity.

Table 6-2: Comparison of Ultimate Stresses and Moduli of Elasticity for Fresh Girders

SCL Type	Specimen Name	Ultimate Flexural Stress (psi)	Allowable Flexural Stress (psi)	Comparison	Maximum Deflection (in)	Modulus of Elasticity (psi)	Published Modulus of Elasticity (psi)	Comparison
LVL	LVLC1	4569	2750	166%	1.590	1.482×10^6	2.0×10^6	74.10%
LVL	LVLP1	7317	2750	266%	2.307	1.636×10^6	2.0×10^6	81.79%
PSL	PSLC1	7687	2900	265%	2.331	1.701×10^6	2.0×10^6	85.03%
PSL	PSLP1	6842	2900	236%	1.892	1.865×10^6	2.0×10^6	93.23%

6.2 Fatigue Performance

The following subsections describe the various aspects of the fatigue performance of SCL girders under repetitive load.

6.2.1 General Physical Behavior

Due to one million cycles of fatigue loading no obvious physical damage was introduced in any of the beams. There was no significantly visible crack or degradation in the SCL material of both LVL and PSL girders (Figure 6-3). However, the LVL beams made out of SCL pieces epoxied together, suffered some damages in terms of further deterioration of the cracks that were present before fatigue loading. These cracks at the SCL-epoxy interface, caused by weathering effects, increased in size. In case of the cracks at the bottom of the web, they propagated further along the length of the beam (Figure 6-4). Some of the LVL beams had cracks in the web. These cracks disappeared when the girders were post tensioned. However after fatiguing, when the post tension was relieved, these cracks grew larger in size. Figure 6-5 shows the enlargement such cracks after fatigue.

Figure 6-3: Typical PSL Cross Section after Fatigue Loading

Figure 6-4: Propagation of Cracks at the Bottom of LVL after Fatigue Loading

Figure 6-5: Enlarged Cracks at SCL-Epoxy Interface due to Cyclic Loading

6.2.2 Loss of Post Tension

As described before, two load cells were placed at mid-span and at the end of the girders. These load cells were placed to monitor the gradual loss of post tension in the Dywidag bars. At approximately every 100,000 cycles the fatigue loading was stopped and the girders were unloaded. At this time the load cell readings were recorded using a strain gauge box. It is to be noted that the decision of putting load cells on the Dywidag bars was taken at a later time after fatigue testing of some of the beams had already completed. As such, loss of post tension data was not available for all the beams.

The general trend obtained from the plots of post tension data with time (number of load application cycles) revealed that the girders tend to lose post tensioning force gradually. Figure 6-6 shows one such typical plot. It is observed that the loss of post tension for both locations was drastic in the beginning. This may be attributed to the fact that the SCL material itself compresses to a large extent as the high post tensioning force is applied (Figure 6-7 and Figure 6-8). The phenomenon of compression continued for a certain amount of time after fatigue loading had been applied. This could be further verified from the fact that although each Dywidag bar was stressed with a force of 20,000 lbs, most of the plot showed the initial load cell reading to be around 18,000 lbs. This meant that by the time fatigue loading started after application of post tension the girders had lost roughly 2000 lbs of tension in each bar. This rapid decrease in post tensioning force may also be attributed to the possible relaxation of the post tension bar. However, there was no provision to measure such quantity.

After the initial rapid decrease in post tension force, the rate of loss with time slowed down considerably. This slow gradual loss may be attributed to the consecutive loading and unloading of the girder in each fatigue cycle. At this stage, it was assumed that the SCL material has compressed to its fullest extent. However, due to repetitive bending and flexing of the girders the Dywidag bars and the adjoining anchorage system tended to relax and hence caused slower decrease in the post tensioning force. As far as the location of post tension bar on the girder is concerned, there was no definite conclusion about which bar location suffers the most amount of post tension loss. In some cases the load in the bar at the end was higher than the one in the middle. In other situations the case was completely opposite. Although

adequate efforts had been made, it was not possible to maintain the same level of applied load in all the Dywidag bar locations. In addition, different sections along the length of the girders might have compressed at different rates due to variation in moisture contents. This might have caused the forces to be dissimilar to each other.

Figure 6-6: Post Tension Stress Vs. Load Cycles for LVLC3 Specimen

Figure 6-7: Compression of SCL due to Post Tension Force Applied

Figure 6-8: Permanent Deformation in SCL due to Post Tension Force Applied

6.2.3 Loss of Stiffness

At the very beginning of any fatigue testing all the girders were loaded statically up to a load level of 23.4 kips which was the maximum load in each fatigue loading cycle. Continuous load-deflection data were recorded at a rate of four samples per second during this loading. The slope of the load-deflection curve provided a measure of the modulus of elasticity of SCL material. During the time of fatigue testing, the actuator was stopped at every 100,000 cycles. At this point the girders were unloaded completely. Similar static load was then applied on the girders gradually up to 23.4 kip and corresponding load-deflection data were collected. Thus the slopes calculated from subsequent load-deflection data sets would provide a measure of the change in modulus of elasticity of the girders. This change MOE values over time, if any, dictated the effect of fatigue within the SCL material.

Subsequent load-deflection data gathered over a period of one million cycles of fatigue loading did not reveal any noticeable change in the modulus of elasticity of the girders. In some of the samples there were very minute degradations of the moduli (Figure 6-9). In other samples, the moduli remained almost the same as the initial value. This behavior exhibited that one million cycles of fatigue from AASHTO loading did not change any the SCL material properties considerably.

Figure 6-9: Typical Load Vs. Deflection Plot before and after Fatigue Loading

6.2.4 Loss of Static Strength

A material subjected to fatigue should experience changes in its material properties. Due to repetitive application and removal of stresses the material becomes compliant over time. When a specimen of a certain material has been stressed for a large number of cycles, its ultimate strength should be smaller than the failure strength of a fresh and un-fatigued specimen of the same material. This reduction in ultimate strength due to fatigue loading has been characterized as loss of static strength within the current research. The term “residual strength” may also be used to describe this effect. In this regard, the static loading required to fail a fatigued specimen was denoted as its residual strength.

It was described in the previous section that there was no substantial change in the stiffness of the LVL and PSL beams due to one million cycles of truck loading. Consequently, the static failure strengths of the beams were not reduced at all due to fatigue loading. Figure 6-10 shows a comparison of the residual static strength of the fatigue girders and the ultimate breaking strength of fresh girders. It is obvious that in case of PSL girders the ultimate strengths of fresh and fatigued girders were very similar. The slight dissimilarity in this

quantity was attributed to the variability of the SCL material itself. Same observation could also be applied to the case of Penta treated LVLs. The Copper Naphthenate treated LVLs exhibited erratic behavior in terms of their breaking strength. In this case the strength of the fresh girder was less than both the fatigued girders. Also the strength was much smaller than the Penta treated LVLs. In addition to that, the residual strength of the fatigued girders varied considerably from each other. However, this kind of variability might be attributed to the cracks existing in LVL girders even before the initiation of fatigue loading. Environmental changes had produced large cracks at the SCL epoxy interface which significantly reduced the static strength of the material. The fact that these cracks occurred at the bottom of the web, also helped to reduce the flexural strength of the girders. In fact the fresh beam which only registered 37.9 kip as its ultimate strength had the worst case of de-lamination at the web.

Figure 6-10: Comparison of Ultimate Strengths of Fatigued and Fresh Girders

6.3 Performance Under Static Loading

Table 6-3 shows the ultimate strengths of the fatigued girders in detail. It also expresses the strength values as percentages of the ultimate strengths of fresh girder made with the same SCL material and having similar preservative treatment. The ultimate strengths of the fatigued girders were denoted as their residual strengths.

Table 6-3: Residual Strength of Fatigued Girders

SCL Type	Preservative Treatment	Specimen Name	Ultimate Strength (kip)	Ultimate Strength of Fresh Girder (kip)	Percentage
LVL	Penta Chlorophenol	LVLP2	64.73	60.75	107%
LVL	Penta Chlorophenol	LVLP3	53.73	60.75	88%
LVL	Copper Naphthenate	LVLC2	53.39	37.94	141%
LVL	Copper Naphthenate	LVLC3	43.49	37.94	115%
PSL	Penta Chlorophenol	PSLP2	65.63	56.81	116%
PSL	Penta Chlorophenol	PSLP3	55.49	56.81	98%
PSL	Copper Naphthenate	PSLC2	62.12	63.83	97%
PSL	Copper Naphthenate	PSLC3	66.85	63.83	105%

Table 6-4 describes the average ultimate strengths and corresponding standard deviation values of the fatigued and un-fatigued girders. The values are given for both LVLs

and PSLs. Also average values of strengths for two types of girders are also shown. The last row within the table describes the average ultimate strengths of all the girders and the corresponding standard deviation. It is observed that the LVL girders had much higher standard deviation than the PSL girders. This was due to the fact the LVL girders had largely varying ultimate strengths as they were more affected by the weathering phenomenon leading to different magnitudes of large visible cracks in them. As described in Table 6-3 their strengths varied from a low value of 37.94 kips to as high as 64.73 kips. The ultimate strengths of the PSL girders were more consistent and the average values were slightly larger than 60 kips. This again proved that as a material PSL is superior to LVL. Finally, the combined average breaking strength of all the girders were 57.06 kips and the standard deviation was 4.67 kips

Table 6-4: Average values and standard deviation of ultimate strengths

Girder Type	No. of Girders	Average Ultimate Strength (kips)	Standard Deviation
Fatigue Loaded LVLs	4	53.84	8.68
Static Loaded LVLs	2	49.35	16.13
Average of LVLs	6	52.34	10.13
Fatigue Loaded PSLs	4	62.52	5.10
Static Loaded PSLs	2	60.32	4.96
Average of PSLs	6	61.79	4.67
LVLs & PSLs Combined	12	57.06	8.99

6.4 Modulus of Elasticity & Deflection

During the static loading of all the SCL girders up to failure, two sets of data were recorded continuously at a rate of four samples per seconds. These were the increasing load data (P) and the travel distance of the actuator head. The distance traveled by the actuator can

safely be assumed as the deflection (Δ) at the mid span of the girders based on the test setup geometry. These load-deflection data for all the girders were plotted in excel. Since the girders behaved more or less elastically up to the point of failure, the P- Δ plot in most cases were completely linear. Based on these linear plots the modulus of elasticity values for each girder was calculated using Equation 6-1.

Table 6-5 shows the moduli of elasticity values thus obtained from the girders. Also it tabulates the maximum deflection at midspan just before failure. It should be noted that except one, none of the MOE values matches the advertised value of 2×10^6 psi for SCL material. The deflection for LVL varied from 1.570 in. to 2.307 in. while the ones for PSL varied from 1.892 in. to 2.331 in. PSL girders experienced larger overall deflection than the LVL girders. This was due to the fact that PSL girders actually sustained much higher strength before impending failure. Probably the more homogenous nature of PSL material imparted this property to these girders and established a preference over LVL.

Table 6-5: Comparison of moduli of elasticity and deflections

Specimen Name	Load (P), lbs	Deflection (Δ_{midspan}), in	Modulus of Elasticity, psi
LVLC1	37936	1.590	1.46×10^6
LVLC2	53392	2.029	1.59×10^6
LVLC3	43494	1.970	1.36×10^6
LVL P1	60755	2.307	1.63×10^6
LVL P2	65729	1.570	2.60×10^6
LVL P3	53734	2.004	1.65×10^6
PSLC1	63827	2.331	1.74×10^6
PSLC2	62120	2.199	1.73×10^6
PSLC3	66850	2.263	1.81×10^6
PSLP1	56806	1.892	1.84×10^6
PSLP2	65631	2.302	1.77×10^6
PSLP3	55489	2.331	1.52×10^6

CHAPTER 7

HIGH STRESS FATIGUE TESTING

7.1 Introduction

Fatigue testing performed on SCL girders were carried out at very low stress levels. Load levels were kept similar to in-service condition based on AASHTO LRFD estimation. Each of the girders was cycled from a minimum load of 2 kip to a maximum load of 23.4 kip. Chapter 6, however, revealed that the ultimate strength of these girders were much higher than the load applied in each cycle. The resulting mean stress in each fatigue cycle was thus very small compared to the ultimate strength. Now, if a specimen was fatigued at a very low stress levels, there might not be any significant damage to the sample due to fatigue. The results obtained so far actually corroborated this fact. However, if it was possible to fatigue the samples at a much higher stresses, substantial deterioration might be evident. At much higher stress levels, the deflection would be much higher in each cycle. The cumulative damage would be more sustained and would propagate through out the stressed region, ensuring impending failure. A decision was, therefore, taken to perform some high strength fatigue testing on some samples that were left unused after the initial cyclic testing. The results obtained from these tests provided insight into the fatigue characteristics of SCL girders at higher load levels. Also performing these tests at different stress levels introduced a scope of building a rudimentary S-N curve for these bridge girders.

7.2 Test Setup

For the high strength fatigue loading, the same four point bending test setup was used. This setup ensured a pure moment section in the middle part of the girder producing only flexural stresses and free from any shear stress. For the high strength fatigue loadings, both

load and deflection data were recorded continuously. The data acquisition computer and equipment was set up to record 4 data points per second. The continuous stream of data was recorded instantly on the computer hard drive which ultimately resulted in huge volume of data files. Deflection data was recorded for the midpoint of the girders. Both load and deflection data were obtained from the MTS hydraulic actuator through the 407 control device. Figure 7-1 reveals a brief segment of the data that were recorded during the process. The horizontal axis in this figure represents time while the vertical axis shows both load and deflection. The figure shows the application of cyclic load over time in a consistent sinusoidal pattern. In addition to these two measurements, strain gauges were placed on top of the flange and at the bottom. The loading frequency for the high strength fatigue testing was somewhat reduced compared to the frequency maintained in regular fatigue testing. Cyclic load during these tests was applied at rates that varied from 0.5 Hz to 0.7 Hz. The loading rate had to be reduced because much higher loads were being applied in this case. Increasing the loading rate while maintaining comparatively large load levels, would have produced tremendous pressure on the hydraulic actuator. This would have introduced the possibility of the desired load levels not being applied to the specimens.

Figure 7-1: Fraction of Sinusoidal Cyclic loading

During the application of high levels of fatigue loads, another problem with the test setup became evident. Initial test setup as developed during the low strength fatigue testing applied load with a four point bending configuration. In this test setup, the loads from the actuator at the middle two points were applied as line load spanning the entire width of the flange. This was achieved by attaching two rectangular steel tubing at the bottom of the header beam (Figure 7-2). However, most of the girders did not have a plain horizontal surface at the web. Due to environmental effects, while they were lying outside in the field, cracks were introduced at the web-flange interface of the some of the girders. This also caused the girders to have curvilinear surface on top of the flange (Figure 7-3). When the actuator applied load on the girders, the curvature at the top of the flange caused differential load application throughout the width of the flange. At extremely high loads, the difference in load becomes too large and produces a net torsional effect on the girder. This created a tendency of twisting in the girders which is very dangerous and might turn out to be disastrous. To avoid this phenomenon, additional steel plates were placed in between the girder flange and the steel tubing as shown in Figure 7-4. This caused an equal load distribution on the two sides of the flange and thereby reduced the possibility of twisting.

Figure 7-2: Header beam with Steel Tubing

Figure 7-3: Typical curvature of Flange top

Figure 7-4: Loading Arrangement including Steel Plates

7.3 Test Specimens

After the initial fatigue testing, two PSL T-beam bridge girders were left. These two specimens were used for high stress fatigue testing. Cyclic loading was applied to these two girders at two different mean stress levels. Detailed discussion about these two stress levels are provided in the following section. Although testing of only two samples was not enough to draw conclusion about the high strength fatigue characteristics of SCL bridge girders, it at least shed some light on the prospect of future endeavors involving these girders, being repeatedly loaded at a higher strength. The results from this study may also be referenced to obtain some knowledge of the SCL material behavior when subjected to high levels of cyclic loading.

7.4 Loading for High Strength Fatigue Tests

Chapter 6 revealed that the average ultimate breaking strength of the PSL girders was around 61.79 kips with a standard deviation of 4.67 kips. Considering a normal distribution of the ultimate stresses and 5% exclusion, the maximum ultimate strength was 69.48 kips. During fatigue loading using the repetitive truck loads, the maximum load in each cycle was 23.4 kips while the minimum was kept at 12 kips. This signified that the mean strength in each load application cycle was 17.7 kips. Compared to the maximum ultimate strengths of the PSL girders this strength was only 26%. Figure 7-5 shows a typical S-N curve for different materials. The S-N curve provides an estimate of the time required for a material to collapse due to fatigue at different stress levels. The ‘S’ terms represents the average of the maximum and minimum stress in each cycle and also is known as mean stress. However, in an S-N diagram, the mean stress is expressed as a percentage of the ultimate strength. On the other hand, the term ‘N’ signifies the time required for fatigue failure and is expressed as the number of cycles of loading the material can withstand. The S-N diagram in Figure 7-5 shows that different material has dissimilar capacities in terms of resisting fatigue loading. Also materials behave differently at various mean stress levels. It is evident that generally ductile materials such as metals tend to have lower fatigue lives at high mean stress levels. Also at lower stress levels the time required for fatigue failure in metals becomes infinite. This leads to an inverted “S” shape of the S-N plot which is characteristic of the metals. On the other

hand brittle materials such as wood and concrete tend to have uniformly varying fatigue qualities where such materials tend to withstand very high fatigue cycles at lower mean stresses and vice versa. This is manifested by the linear shape of the S-N plot for such materials.

¹ Mild steel; ² Aluminum alloy; ³ Concrete; ⁴ Wood

Figure 7-5: Typical S-N diagram for different materials

As observed from the typical S-N diagram (Figure 7-5), fatigue life gets extended at lower mean stress levels while samples tend to fail earlier when they are repetitively stressed at higher mean stress levels. This is true because at lower mean stress levels the cumulative damage accumulation occurs for a much longer period until it reaches the critical failure point. Following this fact, we can safely conclude that, two millions cycles of loading at a mean stress of 26% was not enough to produce any noticeable fatigue damage in SCL girders.

Determination of a high mean strength level that would produce pronounced fatigue effect was very important. This might not be as critical in the current case where the SCL girders were being used to carry only vehicular or truck traffic. However, this information might become desirable if the potential of these SCL girders to carry repeated loads from armored vehicles and other military equipment was investigated. Without a certain guideline, identification of such a threshold fatigue load level became very difficult. For laboratory

testing purposes, one choice in such a case was to arbitrarily choose higher load levels and keep on testing in fatigue until failure was reached. Corresponding high stress levels and fatigue life data then could be recorded to produce a presentable S-N diagram. This process, however, was very time consuming and requires a large number of samples. In the current study, only two of these expensive girders samples were available after the initial fatigue testing. Therefore, adoption of the aforementioned testing procedure was not feasible. Another approach to solve this problem was to estimate high strength fatigue load levels beforehand, based on available fatigue data for SCL material and perform the fatigue testing on the girders using those values. But, the fact that fatigue data for SCL material was almost non-existent, introduced a hurdle in this approach. However, in the absence of any fatigue data for standard SCL material, alternative sources of such information for similar materials were sought. One such option was to employ the fatigue failure data for clear wood species. Although SCL differed from clear wood in terms of material composition, conservative estimates was derived which could be used in high strength fatigue testing of the girders.

The wood handbook published by USDA is the de facto standard for obtaining material properties of clear wood. Information about fatigue characteristics of wood are presented in section 4 of the manual. The wood handbook, however, does not provide an S-N diagram for clear wood species. It provides a brief summary of fatigue life (in millions of cycles) of clear wood for different loading conditions as shown in Table 7-1. The table also describes the variation of fatigue life in clear wood specimens due to different grain alignment as well as due to the presence of knots. Most of the fatigue data specified in the table indicates a stress ratio of 0.1. This means that the ratio of maximum stress to minimum stress is 0.1. Corresponding text also suggest that for a particular sample, the maximum loading stress in each cycle was kept at 60% of ultimate strength of wood. Since the stress ratio is 0.1, minimum stress in each cycle is 6% of failure strength. An average of the two stresses produces a mean stress of 33% of ultimate strength. Also, the table shows different loading frequencies for different loading conditions. For the current study, attaining these ranges of loading frequency was out of the scope because of the size of the specimens and the limitations of the hydraulic actuator. The wood handbook, however, does not mention any

dimensions of the specimens which were employed to tabulate the afore-mentioned fatigue characteristics.

Table 7-1: Reported results on Cyclic Fatigue of Clear Wood

Property	Range ratio	Cyclic frequency (Hz)	Maximum stress per cycle ^a (%)	Approximate fatigue life (million cycles)
Bending, clear, straight grain				
Cantilever	0.45	30	45	30
Cantilever	0	30	40	30
Cantilever	-1.0	30	30	30
Center-point	-1.0	40	3C	4
Rotational	-1.0	—	28	30
Third-point	0.1	8-1/3	60	2
Bending, third-point				
Small knots	0.1	8-1/3	50	2
Clear. 1:12 slope of grain	0.1	8-1/3	50	2
Small knots. 1:12 slope	0.1	8-1/3	40	2
Tension parallel to grain				
Clear, straight grain	0.1	15	50	30
Scarf joint	0.1	15	50	30
Finger joint	0.1	15	40	30
Compression parallel to grain				
Clear, straight grain	0.1	40	75	3.5
Shear parallel to grain				
Glue-laminated	0.1	15	45	30

^a Percentage of estimated static strength

A close look at the table shows that for clear wood specimens with straight grain alignment, fatigue life is 2 million cycles in third point flexure loading. This leads to the

observation that when stressed at a mean stress of 33%, the clear wood specimen in third point bending will fail due to fatigue after 2 million cycles of loading. Also it is obvious that at a mean stress of 100%, the fatigue life of the same sample will be one. Although this assumption is not clearly stated in the table, a look at the typical S-N diagram of wood in Figure 7-5 establishes this idea as fact. In essence, fatigue loading at 100% means stress level will imply that in the very first cycle the specimen will be loaded up to the ultimate strength of the material. Except for some variances, in most cases this will cause failure without the chance of any cycling of the applied load.

Theoretically, a mean stress of 100% signifies that the maximum stress in each cycle may be higher than the ultimate strength, which is not possible to attain in real life testing. Loading a sample up to 110% of static load and then unloading to 90% to achieve a mean stress of 100% does not seem to be a feasible idea. Any loading above the ultimate strength is likely to permanently damage the specimen. For the same reason mean stress less than but close to 100% is also difficult to achieve. Even if it is possible to apply loads at a high mean stress level, the cyclic loading will feature very small amplitudes. In other words the difference between minimum and maximum stress in each cycle will be minuscule. As for example, if it is required that a mean stress of 95% of ultimate strength is to be achieved in each loading cycle, the minimum stress might be 92 % while the maximum stress might be kept at 98%. However, loading the sample to 98% of ultimate strength may turn out to be a risky proposition. Due to statistical variation from sample to sample, some specimen may fail well below the average ultimate strength and thereby fail the test. On the other hand, cycling the load at a lower mean stress level may provide some flexibility. If the mean stress is to be maintained at 70%, each sample may be stressed from 50% to 90% of ultimate strength in each cycle.

From previous discussion of fatigue life of clear wood, two sets values were obtained as shown in the Table 7-2 below. Based on these two instances, a simplistic linear S-N plot was developed for clear wood in flexural fatigue. The resultant plot is shown in Figure 7-6. It is clearly evident from the above discussion and the plot that the 26% of mean stress applied for 2 million cycles was not adequate to produce failure in clear wood samples. Since SCL

girders are stronger than clear wood, similar loading rate and duration was not expected to introduce any desired fatigue effect. Also typical S-N curves for other materials show that the fatigue life tends to be infinite at low mean stress level and hence the S-N curve becomes asymptotic. Therefore to attain fatigue failure of the SCL girders at 26% mean stress the cyclic loading might have to be continued for loading cycles well over 2 million. Based on the tentative S-N diagram shown in Figure 7-6 mean stress levels for high strength fatigue testing might be identified. Determination of a certain mean stress using this graph provided a tentative estimate of the time required to achieve fatigue failure for SCL girders.

Table 7-2: Estimated fatigue life of Clear Wood in Third Point Bending

Mean Stress Level (% of Static Strength)	Fatigue Life (No. of Load Cycles)
100	1
33	2,000,000

Figure 7-6: Typical S-N plot for Clear Wood in Third Point Bending

For the first PSL girder that was tested in high strength fatigue, the maximum load in each cycle was set to 62 kip. Loading the girder at this load level was potentially dangerous, since it was very close to the average failure strength. The minimum strength for each cycle was set at 56 kip. Resulting mean strength was, therefore, 59 kip which was almost 84.91% of the average failure strength of PSL girders as obtained from previous static tests. Observation of the S-N diagram of clear wood revealed that for mean strength of 84.91% of failure load, the fatigue life was 450,342 cycles.

The second PSL girder that was tested in high strength fatigue had a maximum load of 56 kip applied in each cycle. Corresponding minimum load was 50 kip, thus providing a mean load of 53 kip. The mean strength was 76.28% of the average failure strength.

Table 7-3 describes the expected fatigue life of the specimens at the tested mean stress levels. This data was obtained from the rudimentary S-N diagram (Figure 7-6) and interpolation of Table 7-2. It has to be mentioned here again that, this information was provided for clear wood specimens and not particularly for SCL material. The outcome might vary for SCL which is a better, denser and structurally sound material than regular wood. Also it has to be remembered that fatigue life is denoted by the number of load cycles required to completely fail the SCL girder in fatigue.

Table 7-3: Expected Fatigue Life of Clear Wood samples at different mean stresses

Mean Stress Levels (Percentage of average ultimate strength)	Fatigue Life (No. of Load Cycles)
84.91 %	450,342
76.28 %	708,111

7.5 Results from High Strength Fatigue Test

Cyclic loading at higher mean stress levels produced desired fatigue failure of the two SCL bridge girders. The girders suffered complete structural failure at numbers of loading cycles well below two million. The failure types were catastrophic where the beams were almost broken into two pieces. The failures occurred during the night while the beams were unattended. The safety feature in the 407 controller connected with the actuator, however, stopped the loading once failure was identified. Figure 7-7 and Figure 7-8 shows one of the girders failed in high strength fatigue. It is evident that the damage due to fatigue failure was very devastating. The girder had multiple brittle failures at different places within the section where pure flexural loading was applied. Such failure of the girders justifies the outcome from previous low strength fatigue tests which dictated that the mean stress caused by the AASHTO LRFD truck loading was relatively low to produce any fatigue failure of these girders within the period of their service life.

Figure 7-7: Complete failure of SCL Girder under Fatigue Loading

Figure 7-8: Close up shot of Failure due to High Strength Fatigue

As stated earlier, during the high strength fatigue testing of PSL girders, load deflection data were recorded continuously. Along with this information the timestamp data was also recorded which helped to identify the exact time at which a certain loading or deflection occurred. Thus the loading and deflection data when plotted individually with respect to time produces sinusoidal curve as shown in Figure 7-1. This huge amount of data that spanned hundreds of megabytes of disk space was fed into an MS Access database. Using relational query within Access, deflection data points relating to each peak loading were filtered and was fed into a separate database table. Thus this table contained deflections which correspond to highest loading in each cycle with their appropriate timestamp. When these deflection values for the two PSL girders were plotted against time, Figure 7-9 and Figure 7-10 were produced. It is observed that the deflection of the girders increased gradually as more cycles of loading was being applied. This phenomenon almost resembles the envelope in hysteresis plot obtained when metals are subjected to loading and unloading in fatigue cycles. The gradual increase signified cumulative fatigue damage effect within the girders. Almost similar to metal and other materials, the deflections increased rapidly at the beginning.

Within the middle portion of the fatigue life deflections tended to be stable over a period of time. Close to the end of fatigue life, the deflections started to take off more rapidly. As the fatigue loading continued, the girders became pliable gradually and started to suffer more deflections for the same amount of loading. For example, the beam stressed at 84.91% of means strength had an initial deflection of 2.06 in. while just before failure it suffered a deflection of 2.55 in. Thus the amount of accumulated damage caused an increased deflection in the amount of almost half an inch. In the case of the girder stressed at 76.28% of mean strength the fatigue damage caused an additional deflection of 0.59 in before it failed catastrophically.

Figure 7-9 and Figure 7-10 show a few breaks within each plot. These breakages actually signify stoppage of the fatigue loading cycles. Due to unavoidable circumstances it was not possible to fatigue these girders continuously over the whole extent of their fatigue lives. After any stoppage, there was always a delay of 1-2 days before load cycling could be started again. Within the plots, this time delay had been eliminated to bring continuity of the pots. Also, with time gaps in the plot, interpretation of the overall nature of the fatigue loading would not have been appropriate. It was observed that after each stoppage when fatigue loading had started again, the deflection was little bit less than the maximum deflection obtained before stoppage. This implies that within the stoppage time the girders recuperated some of the damage accumulated due to fatigue. But, the girders were also very quick on gaining back the maximum deflection achieved in the previous loading cycles. This behavior also reminded of loading, unloading and reloading effect in metals within the elastic limit.

Figure 7-9: Cumulative Deflection in PSL Girder Fatigued at 84.91%

Figure 7-10: Cumulative Deflection in PSL Girder Fatigued at 76.28%

Table 7-4 shows a comparison of the fatigue lives as obtained from the high strength tests to the ones calculated as per the wood handbook. The beam loaded at a higher mean stress actually failed earlier than its expected fatigue life. This might be due to some variance in that particular PSL girder itself. The only concrete conclusion about its failure could be drawn if it was possible to test many more of the same PSL girder at the same mean stress level. This judgment was also true for the other PSL girder which was tested at 76.28% mean stress level. In this case however, the fatigue life exceeded the expected value by 20.04%.

Table 7-4: Comparison of obtained and predicted Fatigue Lives

Mean Stress Level (%)	Fatigue Life		Difference (%)
	Clear Wood	SCL Girder	
84.91 %	450,342	370,000	-17.84 %
76.28 %	708,111	850,000	+20.04 %

Figure 7-11 shows the S-N plots of both clear wood and SCL girder; more specifically PSL girder. The S-N plot for clear wood is represented by the dotted line and is completely linear as shown in the figure. This is due to the fact that only two fatigue data points were identified using the wood handbook. The solid plot represents the S-N data of the PSL girder based on the high mean stress fatigue loading. It is observed that the S-N plot for the SCL girder closely followed the plot for clear wood. Again, it should be mentioned that this plot is based on fatigue testing of only 2 samples, which is an extremely inadequate sample size to draw any justifiable conclusion.

Figure 7-11: Comparison of S-N plots for Clear Wood and SCL Girder

CHAPTER 8

FINITE ELEMENT MODELING OF SCL GIRDERS

8.1 Introduction

Finite element modeling provides a convenient method to predict the behavior of structural elements under different loading conditions. It can be used as a preliminary tool to estimate the acceptable limits of parametric quantities involved in a physical structural laboratory testing or real-life in-service loading scenario of a particular structure. For the current fatigue research, finite element modeling can be used to predict future behavior of SCL girders after repetitive application of wheel loads. The approach taken might be to model the SCL material itself using suitable finite elements which best represents the material behavior obtained from rigorous laboratory testing. Using the elements selected, a complete model of the girder can be prepared. The final model will then be subjected to cyclic loading within the final element analysis program, with appropriate fatigue parameters built into it. The fatigue performance of the FE model can then be verified using the physical fatigue testing results obtained in the laboratory. If the two results coincide within an acceptable limit, the finite element model developed can be used as a standard for subsequent fatigue investigation of other structures made of SCL material. With the emergence of enormous computing prowess of modern era, the finite element model can be further tweaked to attain greater accuracy and convenience. Such computational tools can greatly reduce the cost involved in actual fatigue testing of SCL girders.

8.2 Finite Element Modeling Tool

During the current fatigue research, the finite element analysis program ANSYS was selected as it was readily available. Currently at version 11, ANSYS has gathered much

respect and recommendations as one of the leading application software of this kind. Within ANSYS, there is a complex fatigue module that can be implemented to produce repetitive loads and corresponding material behavior. For proper implementation of the fatigue module the following data must be input within the program.

- S-N Curve – As described in the previous chapter, this is a plot of alternating stress intensity, $(S_{\max} - S_{\min})/2$ versus allowable number of cycles within the fatigue loading. Here $S_{\max}$ the maximum loading stress in each cycle while $S_{\min}$ signifies the minimum stress intensity.
- S_m -T Curve – This is a curve of design stress-intensity value versus temperature. This curve is needed by the program to detect whether or not the nominal stress range has gone plastic.
- M and N – Elastic-plastic material parameters (strain hardening exponents). These parameters are required if simplified elastic-plastic code calculations are required. Values for these parameters can be obtained from Boiler Pressure Vessel Code published by the American Society of Mechanical Engineers (ASME).

8.3 Limitations of Fatigue Modeling for Current Research

The ASME Boiler & Pressure Vessel Code, which is the basis for the fatigue module in ANSYS, was primarily designed to be used with ductile materials like metal and not wood. It has been previously established that the fatigue loading for the SCL girders will be limited to a level which will produce stress intensity much less than the yield stress or plastic stress. In other words the mean stress in each cycle is much less than failure strength. It is also evident from the static failure loading tests that the stress-strain behavior for the SCL girders is primarily linear. No plastic range is readily observed before the material suffers almost brittle failure. Absence of any plasticity within the material impedes the use S_m -T curve as required in fatigue analysis using ANSYS. S_m -T curves for other material describes the stresses within the plastic range which was not identified in the previous static failure tests. In addition to that, such curves are not available for wood or SCL as the strengths of these materials are never implemented in design beyond their allowable limits. In fact, there is not

much information about plastic behavior in wood altogether. For the same argument, the elastic-plastic material parameters M and N also cannot be implemented within the fatigue modeling using ANSYS.

The only parameter that can be input within ANSYS for the current fatigue modeling is an S-N curve. For the current fatigue finite element modeling, the rudimentary S-N plot for clear wood specimen as developed in the previous chapter might be used. However, in the absence of any S_m -T data or other elastic-plastic material parameters, the fatigue module within ANSYS works as an interpreter of the S-N data only. In such a case the fatigue module calculates the stresses and usage factors of the fatigued specimen based on an interpolation of the S-N plot. Such results or outcomes are, however, very simplistic and can be obtained hand calculation. In addition, since the whole analysis is performed within the elastic limit, the associated stresses, strains and deflections will also remain linear. Implementation of finite element modeling for such fatigue analysis is thus not only unworthy but also is superfluous.

8.4 Revised Finite Element Modeling Approach

Due to the aforementioned shortcomings of the finite element modeling approach, the fatigue modeling of SCL bridge girders using ANSYS had to be abandoned. Instead, a methodology was adopted where the main focus of finite element analysis would be to define a model using ANSYS which closely resembled the characteristics of the actual SCL bridge girder. The validity of the model would then be checked against the results obtained from actual laboratory testing of the girders. For the current study mainly the deflections from various static loading cases were compared to the finite element results of the developed model. A close proximity of the results then justified the model as the one best representing the SCL girders. Such a model would then act as a guiding standard for future finite element modeling of these types of girders. On the other hand, large dissimilarities in the results obtained from static testing and those calculated from finite element analysis deemed the model as an unsuitable one. Such a model should not be used as a representative one of SCL bridge girders and further investigation or tweaking of the model might be required to obtain desired objective.

8.5 Finite Element Selection

The first step in any finite element modeling involves selection of an element type that best suits the characteristics of the material under investigation. Since the current study involved Structural Composite Lumber, an engineered wood product, special care needed to be taken in element selection so as to best represent the laminated effect of the material strengthened with epoxy. Also wood itself being an orthotropic material, the element selected should be compatible with this type of material behavior.

Previous efforts in modeling SCL using finite element techniques are extremely rare. Modeling of full scale SCL girder is non-existent. Some researchers used computational models to simulate damage accumulation and other fatigue characteristics.

Finite element analyses of solid sawn lumber and glulam were carried out by a few researchers. Cofer et al. (1997) modeled different sizes of solid sawn lumbers (SSL) to predict shear strength. FEA modeling produced good results for small member sizes. For larger members, the reduction in shear strength as found in experimental results was not matched by the FEA results. Finite element analysis was performed by Gupta et al. (2002b) on full-size SSL under torsion using isotropic material properties. The study concluded that uniform shear stress occurs within the shear span, which begins and ends at a distance of approximately two times the depth plus the grip distance away from each end of the specimen. Also it was verified using finite element analysis that the shear failure plane in torsion is similar to the known shear failure mode in specimens subjected to transverse loads.

In recent times, Gupta and Siller (2005) have actually modeled SCL using ANSYS. Their research objective was to determine the shear strength of SCL using orthotropic torsion theory and compare the results to the experimental findings. A three dimensional, eight-node brick element was used to simulate the SCL material. The analysis revealed that SCL is more orthotropic in nature than regular lumber. However, the use of an eight-node brick element to model SCL material, which essentially has a laminated construction, was overly simplistic. For a long time now, ANSYS had boasted a number of good shell elements that could be utilized to obtain a more realistic laminar aspect of the SCL material. Also the ability to

define different material properties to different layers within shell element provided an appropriate environment where the interaction between the epoxy and wood laminates could be modeled more accurately.

For the current finite element analysis an ANSYS element SOLID186 was used. SOLID186 is 3-D 20-node layered solid element that exhibits quadratic displacement behavior. The element is defined by 20 nodes having three degrees of freedom per node: translations in the nodal x, y, and z directions. This element supports plasticity, hyper-elasticity, creep, stress stiffening, large deflection, and large strain capabilities. It also has mixed formulation capability for simulating deformations of nearly incompressible elasto-plastic materials, and fully incompressible hyper-elastic materials.

8.5.1 Element Description

SOLID186 is a convenient element to model layered thick shells or solids. In each of these elements a total of 250 different material layers can be accommodated. To model composites with more than 250 layers the element may be stacked on top of each other. This type of stacking also helps in improving the accuracy of the solution. The geometry, node locations, and the coordinate system for this element are shown in Figure 8-1. The figure also reveals that if required this element can be shaped to a prism form.

Figure 8-1: SOLID186 Layered Solid Geometry

In addition to the nodes, SOLID186 input data includes the anisotropic material properties. Anisotropic material directions correspond to the element coordinate directions. The solution output associated with the element is in two forms:

- Nodal displacements included in the overall nodal solution
- Additional element output as shown in Figure 8-2

Figure 8-2: SOLID186 Layered Solid Stress Output

8.5.2 Element Assumptions and Restrictions

The SOLID186 element may be used in finite element modeling provided the following restrictions are met.

- The element can not be used to model a zero volume. Also, twisting of the element so as to create two separate volumes is not allowed. This type of situation may occur frequently if care is not taken in proper numbering of the element.
- If a middle node along one of the edge of the element is removed, it is assumed that the displacement along that edge varies linearly rather than parabolically.
- At least two elements should be used in each direction to avoid hourglass mode.

- If the element is degenerated into a wedge element shape such as a Prism or Tetrahedral Element, the corresponding degenerated shape functions are used. The sizes of the degenerated elements should be kept small to minimize the stress gradients.
- Stress stiffening is always included in geometrically nonlinear analyses.
- The number of layers in each element will never exceed 250.
- The layer orientation angle has no effect in case one of the material layer is hyper-elastic.

8.5.3 Specifics of Current Implementation

The finite element analysis at hand used the SOLID186 element for its ability to prescribe different material properties of the different layers. This was beneficial because rather than using a solid element to define SCL material, both epoxy and wood veneers were simulated in their laminar form. Also the anisotropy allowed in each layer of the element permitted more realistic materialization of the orthotropic behavior of wood laminates.

The veneers used in both LVL and PSL generally had a width of 1/8 inch. The individual layers within SOLID186 were thus kept at that thickness. In reality the thickness was little less than 1/8 inch. This was done to accommodate an even thinner layer of epoxy in between two layers of laminates. Thus within a single SOLID186 element there were alternate layers of wood laminate of 0.120 inch and epoxy layer of 0.005 inch. The combination of these two layers produced the desired thickness of 1/8 inch. When compared to the total volume of the SCL material, the volume of epoxy simulated in this arrangement will be 4%. Correspondence with engineers at Trusjoist have led to the information that in actual production of SCL the volume of epoxy is generally less than 5%. Thus this kind of formulation of SCL material using the layers of SOLID186 was considered reasonably acceptable.

Once the element was selected in a finite element analysis, the size of the element became important. Considering the fact that, within the current analysis, the full length SCL girder was modeled, there was not much scope of diminishing the element size to attain

improved accuracy. If it was desired that the element sizes would be quarter of an inch while modeling a 20 ft long girder, the number of resulting nodes and elements would be extremely large. One finite element analysis of such a model usually took tremendous amount of time and large computing prowess. On the other hand, if the element sizes were kept very high i.e. 6 inches or 1 ft, the number of elements became too low which led to inaccuracy in results and misinterpretation of the actual scenario. The best policy in this case was to conduct a convergence study. In this technique models with different element sizes ranging from very high to very low values were prepared. Each model was then analyzed to obtain some key quantities like specific displacements or stresses. Due to the varying sizes of the element the obtained values differed from each other. However, as the element sizes tended to become smaller the values of these quantities started to converge. And at one point it was realized that reducing the element sizes further did not produce any significant differences within the results. This point or specifically this size of the element was said to have produced convergence. It was desired that the future finite element analysis of such structure should be performed using that particular size of the elements.

For the current research an element size of 2 inches was chosen. This value was not big enough compared to the 20 ft length of the girders and the same time it was not minuscule enough to warrant a lot of computer processing time and resources. Also for such a large scale modeling the accuracy provided by this model is considered sufficient. The SOLID186 element in this modeling was taken as more or less a cube with each side being 2 inch long. The element size perfectly matched the dimensions of the entire girder so that no element was defined with a length to width to height ratio in excess of 1.5. In some cases the ANSYS program assumes different dimensions of the elements to match the actual physical size of the structure. However, even in those cases the dimension ratios were closed to 1 which was desirable for good outcome from any finite element analysis. Figure 8-3 shows the actual SOLID186 element as used in the current analysis. In the same figure an enlargement of a corner of the element shows alternate layers of epoxy and wood laminates. Figure 8-4 shows the cross section of the SCL girder after it had been modeled in ANSYS using the 2 inch long elements. It is evident that even though different elements are given different element sizes, none of these violated the dimension ratio of 1.5 and no irregular shaped or non-consistent

element had been produced. Figure 8-5 presents an isometric view of the modeled full length girder.

Figure 8-3: Details of the Layered SOLID186 Element

Figure 8-4: Cross-section of SCL Girder displaying elements

Figure 8-5: Finite Element Model of full length SCL Girder

It should be mentioned here that the orientation of the SCL material and epoxy layers within the SOLID186 element represented the exact laminar characteristics of Laminated Veneer Lumber. This type of arrangement of layers did not match Parallel Strand Lumber where the veneers are stacked more randomly. However, a close look at the cross-section of a PSL section will reveal that even the veneers are laid in a disorderly fashion, on a macro scale the wood laminates are mostly placed in a fashion that will closely resemble their orientation in LVL. Hence with some degree of allowance, the chosen element was used to model PSL as well as LVL. In fact, the randomness of the wood strips in PSL makes it very difficult to achieve an exact representation using finite elements. One very conservative solution to this dilemma might be to adopt a solid element that would disregard the composite effect of the PSL and treat the material as a homogenous one. In such a case the SOLID45 element of ANSYS, used by Gupta and Siller (2005), might be utilized to model the material.

8.6 Material Properties in Modeling

After the selection of an element, the next step in any finite element modeling is to prescribe material properties to the elements. SOLID186 element, used in current modeling, is composed of two constituent materials – wood laminates and epoxy. It has been discussed that wood is an anisotropic material. The best reference for the properties of this material is wood handbook. It provides every possible material properties of wood. It also publishes different material properties of different types of wood. Since the LVL and PSL used in the SCL girders tested are made out of Douglas Fir, the properties that will be chosen for the finite element analysis will be confined to that particular species.

Previous static testing of the SCL girders revealed that the constitutive relationship of the SCL material remains linear even up to the point of ultimate failure. The finite element analysis carried out, therefore, will be restricted to elastic behavior only. No plasticity of both epoxy and wood will be defined. As a general rule, there is not enough data available about the plasticity of wood laminates. The epoxy used in SCL may possess some plastic behavior while loaded near failure. However, it will be assumed that the failure strength of epoxy is much higher compared to that of wood and within the load range at which wood laminates fail, the behavior of the epoxy will remain perfectly elastic.

8.6.1 Material Properties for Wood Laminates

According to the wood handbook, wood is an orthotropic material. This implies that along the three mutually perpendicular axes, it has unique and independent material properties. The three longitudinal axes may be defined as below (Figure 8-6).

- Longitudinal – This is parallel to the fiber (grain) and is denoted as L
- Radial – This is normal to the growth rings (perpendicular to the grain in the radial direction) and is denoted by R
- Tangential – This is perpendicular to the grains but tangent to the growth rings; denoted by T

Figure 8-6: Three Directions for Material Properties in Wood

The elastic material properties of wood vary along these three axes. In total nine independent properties in three directions are required to completely describe the elastic behavior of wood. These properties are described below.

- Modulus of elasticity – E_L , E_R and E_T are the three moduli of elasticity along the longitudinal, radial, and tangential axes of wood. The values of these properties are

generally obtained from compression tests of wood samples. The longitudinal modulus of elasticity value E_L for different species may be readily available from flexural bending test, rather than axial test. The wood handbook describes average E_L values for different hardwoods and softwoods as obtained from bending test. Data for other two moduli of elasticity, E_R and E_T , are not extensively available. However, the wood handbook publishes a table (Table 8-1) which relates the E_R and E_T values with the value of E_L for a certain species of wood. The E_L values obtained from wood handbook tables need to be increased by 10% to use the relationships described in Table 8-1. This increase is required to remove the effect of shear deflection within the value of E_L . The elastic ratios, as well as the elastic constants themselves, vary within and between species and with moisture content and specific gravity. The relationships described in Table 8-1 are applicable to various species at approximately 12% moisture content.

- **Modulus of rigidity** – This is also known as shear modulus. This represents the resistance to deflection of a material when it is loaded in shear. The three moduli of rigidity denoted by G_{LR} , G_{LT} and G_{RT} are the elastic constants in the LR, LT, and RT planes, respectively. For example, G_{LR} is the shear modulus that depends on the shear stresses in LT and RT planes and also on shear strain in the LR plane. Moduli of rigidity values of three directions are also published as ratios of elastic modulus E_L as indicated in Table 8-1. As in the case with moduli of elasticity, the moduli of rigidity vary within and between species and with moisture content and specific gravity.
- **Poisson's ratio** – This is the ratio of the transverse strain to axial strain. The Poisson's ratios in wood are denoted by μ_{LR} , μ_{RL} , μ_{LT} , μ_{TL} , μ_{RT} and μ_{TR} . Within the subscript, the first letter refers to the direction of applied stress and the second letter to the direction of lateral deformation. As for example, the Poisson's ratio for deformation along the radial axis caused by stress along longitudinal axis is denoted by μ_{LR} . Wood handbook publishes a table of average Poisson's ratio values for a number of wood species (Table 8-2). The Poisson's ratios vary between species and are also subject to change due to variation of specific gravity and moisture content of the wood.

8.6.2 Material Properties of Epoxy

The epoxy layer within SOLID186 is assumed to be made of a homogenous isotropic material. Since all the analyses are restricted within the elastic range, no plasticity quantity for the epoxy is defined. Thus for the epoxy material only two material properties are required.

- Modulus of elasticity – It is assumed that the epoxy used in SCL has much superior material properties than the wood laminates. Thus in case of a failure, the wood veneers should give in before the epoxy suffers any damage. This effect might also be substantiated by the fact that wood constitutes the sheer volume (95%) of the SCL material. Based on this assumption and for the sake of simplicity it can be assumed that within the modeling the highest modulus of elasticity of constituent wood can be assigned to the epoxy. Now the advertised modulus of elasticity value for SCL is 2,000,000 psi. Thus in the current analysis, the epoxy layer have a modulus of elasticity value of 2×10^6 psi.

Table 8-1: Elastic Ratios for various Species at 12% Moisture Content

Species	E_T/E_L	E_R/E_L	G_{LR}/E_L	G_{LT}/E_L	G_{RT}/E_L
Hardwoods					
Ash, white	0.080	0.125	0.109	0.077	—
Balsa	0.015	0.046	0.054	0.037	0.005
Basswood	0.027	0.066	0.056	0.046	—
Birch, yellow	0.050	0.078	0.074	0.068	0.017
Cherry, black	0.086	0.197	0.147	0.097	—
Cottonwood, eastern	0.047	0.083	0.076	0.052	—
Mahogany, African	0.050	0.111	0.088	0.059	0.021
Mahogany, Honduras	0.064	0.107	0.066	0.086	0.028
Maple, sugar	0.065	0.132	0.111	0.063	—
Maple, red	0.067	0.140	0.133	0.074	—

Table 8-1 – Continued

Species	E_T/E_L	E_R/E_L	G_{LR}/E_L	G_{LT}/E_L	G_{RT}/E_L
Oak, red	0.082	0.154	0.089	0.081	—
Oak, white	0.072	0.163	0.086	—	—
Sweet gum	0.050	0.115	0.089	0.061	0.021
Walnut, black	0.056	0.106	0.085	0.062	0.021
Yellow-poplar	0.043	0.092	0.075	0.069	0.011
Softwoods					
Baldcypress	0.039	0.084	0.063	0.054	0.007
Cedar, northern white	0.081	0.183	0.210	0.187	0.015
Cedar, western red	0.055	0.081	0.087	0.086	0.005
Douglas-fir	0.050	0.068	0.064	0.078	0.007
Fir, subalpine	0.039	0.102	0.070	0.058	0.006
Hemlock, western	0.031	0.058	0.038	0.032	0.003
Larch, western	0.065	0.079	0.063	0.069	0.007
Pine					
Loblolly	0.078	0.113	0.082	0.081	0.013
Lodgepole	0.068	0.102	0.049	0.046	0.005
Longleaf	0.055	0.102	0.071	0.060	0.012
Pond	0.041	0.071	0.050	0.045	0.009
Ponderosa	0.083	0.122	0.138	0.115	0.017
Red	0.044	0.088	0.096	0.081	0.011
Slash	0.045	0.074	0.055	0.053	0.010
Sugar	0.087	0.131	0.124	0.113	0.019
Western white	0.038	0.078	0.052	0.048	0.005
Redwood	0.089	0.087	0.066	0.077	0.011
Spruce, Sitka	0.043	0.078	0.064	0.061	0.003
Spruce, Engelmann	0.059	0.128	0.124	0.120	0.010

Table 8-2: Poisson's Ratio for various Species at 12% Moisture Content

Species	μ_{LR}	μ_{LT}	μ_{RT}	μ_{TR}	μ_{RL}	μ_{TL}
Hardwoods						
Ash, white	0.371	0.440	0.684	0.360	0.059	0.051
Aspen, quaking	0.489	0.374	—	0.496	0.054	0.022
Balsa	0.229	0.488	0.665	0.231	0.018	0.009
Basswood	0.364	0.406	0.912	0.346	0.034	0.022
Birch, yellow	0.426	0.451	0.697	0.426	0.043	0.024
Cherry, black	0.392	0.428	0.695	0.282	0.086	0.048
Cottonwood, eastern	0.344	0.420	0.875	0.292	0.043	0.018
Mahogany, African	0.297	0.641	0.604	0.264	0.033	0.032
Mahogany, Honduras	0.314	0.533	0.600	0.326	0.033	0.034
Maple, sugar	0.424	0.476	0.774	0.349	0.065	0.037
Maple, red	0.434	0.509	0.762	0.354	0.063	0.044
Oak, red	0.350	0.448	0.560	0.292	0.064	0.033
Oak, white	0.369	0.428	0.618	0.300	0.074	0.036
Sweet gum	0.325	0.403	0.682	0.309	0.044	0.023
Walnut, black	0.495	0.632	0.718	0.378	0.052	0.035
Yellow-poplar	0.318	0.392	0.703	0.329	0.030	0.019
Softwoods						
Baldcypress	0.338	0.326	0.411	0.356	—	—
Cedar, northern white	0.337	0.340	0.458	0.345	—	—
Cedar, western red	0.378	0.296	0.484	0.403	—	—
Douglas-fir	0.292	0.449	0.390	0.374	0.036	0.029
Fir, subalpine	0.341	0.332	0.437	0.336	—	—

Table 8-2 – Continued

Species	μ_{LR}	μ_{LT}	μ_{RT}	μ_{TR}	μ_{RL}	μ_{TL}
Hemlock, western	0.485	0.423	0.442	0.382	—	—
Larch, western	0.355	0.276	0.389	0.352	—	—
Pine						
Loblolly	0.328	0.292	0.382	0.362	—	—
Lodgepole	0.316	0.347	0.469	0.381	—	—
Longleaf	0.332	0.365	0.384	0.342	—	—
Pond	0.280	0.364	0.389	0.320	—	—
Ponderosa	0.337	0.400	0.426	0.359	—	—
Red	0.347	0.315	0.408	0.308	—	—
Slash	0.392	0.444	0.447	0.387	—	—
Sugar	0.356	0.349	0.428	0.358	—	—
Western white	0.329	0.344	0.410	0.334	—	—
Redwood	0.360	0.346	0.373	0.400	—	—
Spruce, Sitka	0.372	0.467	0.435	0.245	0.040	0.025
Spruce, Engelmann	0.422	0.462	0.530	0.255	0.083	0.058

- Poisson's ratio – The Poisson's ratio for the epoxy layer was kept at 0.3.

8.6.3 Modification to Material Properties Specific to SCL

Through laboratory experiments Hindman et al (2005) has investigated the material properties of three different SCL materials – LVL, PSL and Laminated Strand Lumber (LSL). They performed five-point bending tests (FPBT) to simultaneously investigate modulus of elasticity and modulus of rigidity. They also performed torsional stiffness measurement test (TSMT) to identify both the in-plane and through-the-thickness shear moduli from a single test scheme. They also determined longitudinal and transverse elastic moduli combined with in-plane Poisson's Ratio through axial compression and tension test. Experimental data obtained from their study indicated that SCL exhibits orthotropic properties that are different from the elastic material properties of wood. Based on their study, they defined new elastic ratios that are quite different than those published in the wood handbook. The elastic ratios

that they described are shown in Table 8-3. It is evident that they used various SCL materials of different wood species. Gupta and Siller (2005) had used the elastic ratios obtained by Hindman within their study to obtain stress distribution in full size SCL under torsion. Their results from the finite element modeling were found to be consistent with the ones obtained from experiments.

Table 8-3: Elastic Ratios published by Hindman et al. (2001)

Wood Material¹	E_L:E_R	G_{LR}:G_{LT}	G_{LR}:G_{RT}	G_{LT}:G_{RT}	E_L:G_{LR}
<i>P. menziesii</i>	17.7	0.93	6.55	7.01	22.4
<i>P. palustris</i>	8.8	1.01	6.34	6.31	12.3
<i>P. tremuloides</i>	10.6	1.35	7.24	5.38	13.1
<i>L. tuliperifera</i>	10.9	1.07	6.50	6.06	13.5
Average:	12.0	1.09	6.66	6.19	15.3
SCL Material	E₁:E₂	G₁₂:G₁₃	G₁₂:G₂₃	G₁₃:G₂₃	E₁:G₁₂
2.0E SP LVL	28.4	1.34	7.41	5.52	34.7
2.2E SP LVL	24.9	1.21	6.27	5.17	37.7
2.0E DF1 LVL	36.4	1.22	8.84	7.22	45.1
2.0E DF2 LVL	32.7	1.59	14.39	9.06	29.2
2.0E YP LVL	34.2	0.79	2.52	3.20	61.7
Average:	31.3	1.23	7.89	6.03	41.7
2.0E SP PSL	31.3	1.24	5.91	4.77	30.0
2.0E YP PSL	36.3	0.68	2.56	3.77	57.4
Average:	33.8	0.96	4.24	4.27	43.7
1.5E A LSL	9.04	2.71	6.54	2.41	19.8
1.5E YP LSL	9.05	1.21	3.67	3.04	33.4
Average:	9.05	1.96	5.11	2.73	26.6

It has to be mentioned that, within the study carried out by Hindman et al (2001), the SCL material is treated as a single homogenous material although it is made of two constituent materials. The current finite element modeling, however, takes the composite effect of SCL into consideration. The use of the Hindman elastic ratios in the current modeling may not be appropriate. However, decision was made to carry out two sets of analyses using the two different elastic ratios.

8.6.4 Summary of Wood Laminate Properties

Previous section discussed two sets of elastic ratios for SCL material that can be adopted in the current finite element modeling. It was decided that the analyses will be carried out using both of these ratios. Two sets of results will be obtained from the computation using this methodology which will be compared side-by-side the results obtained from actual structural testing of the girders.

The elastic ratios for orthotropic material properties for the wood are defined in both cases with respect to the longitudinal modulus of elasticity. Poisson's ratio, however, is an exception, which is given directly in the wood handbook. The longitudinal modulus of elasticity of each SCL girder is obtained from the laboratory testing. It was mentioned in chapter 6 that continuous load (P) data and midspan deflection (Δ) data were recorded during static testing. From these data the modulus of elasticity was calculated which, understandably, varies from girder to girder. Therefore, the E_L values for all the girders tested were readily available. All other moduli for a particular girder were then calculated based on the elastic ratios defined in Table 8-4. The first row of the table defines E_R , E_T , G_{LR} , G_{RT} and G_{LT} in terms of E_L based on the elastic ratios found in wood handbook, while the other row defines the same using the ratios dictated by Hindman. The Hindman ratios only define $E_1:E_2$. Assuming E_1 is E_L , there is no separate ratio defined for E_R and E_T . Since there is no distinct way to measure E_T and E_R during the test, both E_R and E_T are assumed equal for the current case. Douglas fir values from the wood handbook have been selected based on the fact the constituent material for all the SCL girder was the same. For the Hindman ratios, values for 2.0E DF1 LVL were chosen. These ratios, although given for Douglas Fir LVL, were equally applied for PSL girders for two reasons. Firstly, the PSL girders tested during this study were

made of Douglas Fir and no particular ratios are defined by Hindman for this combination material and wood type. Secondly, it was assumed that the SOLID186 element with layers resembling the LVL construction will be used to simulate PSL.

Table 8-4: Elastic Ratios for Finite Element Analysis

Source	$E_L : E_T$	$E_L : E_R$	$E_L : G_{LR}$	$E_L : G_{LT}$	$E_L : G_{RT}$
Wood Handbook	20.00	14.71	15.63	12.82	142.86
Hindman	36.40	36.40	45.10	55.02	398.68

Table 8-5 summarizes the material properties for the finite element model based on the wood handbook elastic ratios. Table 8-6 describes the same properties of the model which were calculated using the Hindman elastic ratios. As expressed before, the E_L values are obtained from the test while other values were calculated using elastic ratios in Excel.

Table 8-5: Laminate Properties using Wood Handbook Ratios

Specimen Name	E_L (psi)	E_T (psi)	E_R (psi)	G_{LR} (psi)	G_{LT} (psi)	G_{RT} (psi)
LVLC1	1460000	73000	99252	93410	113885	10220
LVLC2	1590000	79500	108090	101727	124025	11130
LVLC3	1360000	68000	92454	87012	106084	9520
LVLP1	1630000	81500	110809	104287	127145	11410
LVLP2	2600000	130000	176751	166347	202808	18200
LVLP3	1650000	82500	112169	105566	128705	11550
PSLC1	1740000	87000	118287	111324	135725	12180
PSLC2	1730000	86500	117607	110685	134945	12110
PSLC3	1810000	90500	123046	115803	141186	12670
PLSP1	1840000	92000	125085	117722	143526	12880

Table 8-5 - Continued

Specimen Name	E_L (psi)	E_T (psi)	E_R (psi)	G_{LR} (psi)	G_{LT} (psi)	G_{RT} (psi)
PSLP2	1770000	88500	120326	113244	138066	12390
PSLP3	1520000	76000	103331	97249	118565	10640

The table shows the Poisson's ratio values that were used in modeling. These values have been picked from the wood handbook without any modification. Poisson's ratio values were same for all girders since it is not influenced by other mechanical properties. Also their values were kept the same for both sets of analyses.

Table 8-6: Laminate Properties using Hindman Ratios

Specimen Name	E_L (psi)	E_T (psi)	E_R (psi)	G_{LR} (psi)	G_{LT} (psi)	G_{RT} (psi)
LVLC1	1460000	40110	40110	32373	26536	3662
LVLC2	1590000	43681	43681	35255	28899	3988
LVLC3	1360000	37363	37363	30155	24718	3411
LVLP1	1630000	44780	44780	36142	29626	4088
LVLP2	2600000	71429	71429	57650	47256	6522
LVLP3	1650000	45330	45330	36585	29989	4139
PSLC1	1740000	47802	47802	38581	31625	4364
PSLC2	1730000	47527	47527	38359	31443	4339
PSLC3	1810000	49725	49725	40133	32897	4540
PLSP1	1840000	50549	50549	40798	33442	4615
PSLP2	1770000	48626	48626	39246	32170	4440
PSLP3	1520000	41758	41758	33703	27626	3813

Table 8-7: Poisson's Ratio for Finite Element Modeling

Analysis Type	μ_{RT}	μ_{RL}	μ_{TL}
Wood Handbook	0.390	0.036	0.029
Hindman	0.390	0.036	0.029

8.7 Finite Element Loading

Once the element and its properties are defined, the next step in the process is to set up the degrees of freedom to simulate support. Appropriate nodes within the modeled girder were treated with different boundary conditions so as to simulate a girder which is simply supported. Here it should be remembered that in actual testing the girder was sitting on bearing pad on both ends. These bearing pads had a length of 12 inches which was in full contact with the girder. However, during finite element modeling the supports were located along lines across the width of the web. This was done to achieve simplicity within the modeling. There is a possibility that the deflection values at mid-span during finite element analysis may differ from the actual testing due to this idealization.

The load from the actuator on the girder was also applied along two lines across the width of the flange. The nodes along these particular lines were first selected. The applied load was then equally distributed on top of these nodes as individual point loads.

Finally the post tension effect from the Dywidag bars was also simulated within the analysis. The post tension load was applied as point loads at regular intervals along the sides. The spacing between the point loads were kept similar to the distances between individual post tensioning holes within the actual girder. Figure 8-7 shows the load on the finite element model. The marks at the bottom of the web are the supports. The lines on top of the flange are simulating the actuator load while the loads acting in the transverse direction from both sides are the post tensioning forces.

Figure 8-8 shows the deflected shape of the modeled SCL T-beam bridge girder after the finite element analysis. The figure also shows the deflection along the length as contours. As expected the deflection is the highest at midspan. The indentations along the side of the flanges are due to the applied post tension loads.

Figure 8-7: Loading and Boundary Conditions

Figure 8-8: Deflected Model of SCL T-beam Girder

8.8 Results from Finite Element Modeling

As described above the primary purpose of the finite element analysis will be to compare the deflections obtained in the analysis with those obtained from static tests of SCL girders. Based on the longitudinal modulus of elasticity obtained from laboratory tests, the girders were loaded in ANSYS up to their ultimate breaking strength and corresponding highest deflections were noted. The two sets of deflection values were obtained using elastic ratios obtained from wood handbook and the ones published by Hindman. Table 8-8 describes the ultimate load and maximum deflection of each girder as obtained during testing. The next column calculates the longitudinal modulus of elasticity based on the maximum load and deflection data. The fifth and seventh column describes the deflection at midspan for the girders as obtained from finite element analysis using the two elastic ratios. The sixth and eighth column calculates the difference between the experimental result with analytical result expressed as a percentage based on the laboratory-tested value. It is observed from the table that the deflection values obtained using wood handbook ratios are roughly 40% less than those obtained during experiments. Using Hindman ratios, however, the deflections obtained using finite element analyses were mostly within 3% of the experimental values with the exception of two girders which deflected up to 5% higher. Based on such close proximity of results it needs to be emphasized that the finite element model of the SCL girder developed in the current study very closely resembles the exact structural behavior of the actual structure.

Another interesting point that needs to be noted, based on Table 8-8, is that the deflections values for both LVL and PSL follow almost the same margin of deviation from the original laboratory tested values. In one case they differ by 40% and in the other case they differ by almost 3%. This information leads one to believe that the finite element model developed works equally well for modeling PSL and LVL girders. In other words, on a macro scale, when LVL or PSL material is turned into a larger structural element, the difference in their inherent constitutive relationship becomes non-existent and their interaction between various material properties start to show a certain uniqueness. Their ultimate capacity, however, remains different as it is observed from the table that PSLs tend to have higher modulus of elasticity and tend to sustain higher failure loads than LVLs.

Table 8-8: Comparison of Deflections obtained from Finite Element Modeling

Samples	Load (P), lb	Deflection (Δ), in.	Modulus of Elasticity (E), psi	Deflection (Δ) based on wood handbook ratios, in	Difference %	Deflection (Δ) based on Hindman ratios, in	Difference %
LVLC1	37936	1.590	1.46E+06	0.944	-41%	1.659	4%
LVLC2	53392	2.029	1.59E+06	1.231	-39%	2.127	5%
LVLC3	43494	1.970	1.36E+06	1.175	-40%	2.035	3%
LVL P1	60755	2.307	1.63E+08	1.364	-41%	2.355	2%
LVL P2	65729	1.570	2.60E+06	0.926	-41%	1.597	2%
LVL P3	53734	2.004	1.65E+06	1.194	-40%	2.062	3%
PSLC1	63827	2.331	1.74E+06	1.343	-42%	2.317	-1%
PSLC2	62120	2.199	1.73E+06	1.314	-40%	2.268	3%
PSLC3	66850	2.263	1.81E+06	1.352	-40%	2.334	3%
PLSP1	56806	1.892	1.84E+06	1.131	-40%	1.953	3%
PSLP2	65631	2.302	1.77E+06	1.358	-41%	2.343	2%
PSLP3	55489	2.331	1.52E+06	1.338	-43%	2.310	-1%

Based on Table 8-8, it is evident that the Hindman relationships between material properties should be used in finite element modeling of SCL material. The findings of this study thus corroborate the results obtained by Gupta and Siller (2005). Based on these elastic ratios, further analysis was carried out using the same finite element model. It was decided to find out the deflections of the girders assuming the longitudinal modulus of elasticity to the advertised value of 2,000,000 psi. Table 8-9 describes the deflection values obtained within those analyses. The deflections were then compared with the actual values obtained during laboratory experiments. Since, the structural testing of the girders yielded lesser values of longitudinal moment of elasticity, the deflection values obtained in the analyses were much less than the actual girder in most of the cases. The deflections from the analyses were 16-29% less than the deflections obtained in structural testing. This proves that the advertised value of modulus of elasticity is questionable and should be investigated further.

Further analyses were carried out to study the effect of post tension on the performance of the SCL girders. Table 8-10 shows the deflections that were obtained from finite element analysis by varying the post tensioning force from 10,000lbs to 1lb at decrements of 2500lbs. The analyses were carried out for both PSL and LVL girders. It was found that the deflections increase due to a decrease in post tension force. However, the rise in deflection is very small. In case of a 1 lb lateral load which signifies almost no post tension force, the deflection increased only 0.8% from the deflection value that had been obtained using a 10,000lbs force applied transversely.

Table 8-9: Deflections based on Published MOE and Comparison to Actual tests

Samples	Deflection from Tests, in	Deflection based on $E = 2 \times 10^6$ psi, in	Difference, %
LVLC1	1.590	1.227	-23%
LVLC2	2.029	1.702	-16%
LVLC3	1.970	1.398	-29%
LVL P1	2.307	1.930	-16%
LVL P2	1.570	2.083	33%
LVL P3	2.004	1.713	-15%
PSLC1	2.331	1.741	-25%
PSLC2	2.199	1.694	-23%
PSLC3	2.263	1.824	-19%
PLSP1	1.892	1.547	-18%
PSLP2	2.302	1.790	-22%
PSLP3	2.331	1.511	-35%

Table 8-10: Comparison of Deflection for various Post Tension Forces

Samples	Deflections due to Post Tension, in				
	10000 lb	7500 lb	5000 lb	2500 lb	1 lb
LVL P1	2.355	2.358	2.363	2.369	2.374
PSLP2	2.343	2.347	2.352	2.357	2.361

CHAPTER 9

CONCLUSIONS

9.1 Introduction

The objective of the current research was to evaluate how the SCL T-beam bridge girders perform when they are subjected to one million cycles of truck loads during their projected in-service life of 60 years. The intent was to investigate the fatigue performance of this family of ready-made girders varying in SCL material types and preservative treatment types. Due to their relatively expensive price in the context of available resources for the project, a limited number of specimens were tested under fatigue to simulate the desired effect. In addition, two SCL girders were also tested in fatigue using loads much higher than that generating from truck traffic. Supplemental finite element modeling and analysis were also undertaken to verify material properties and constitutive relationships. The following subsections describe a brief summary of the tests and provide conclusive remarks based on the findings. Effects of high stress fatigue on SCL girders are also discussed. Recommendations are also provided for future investigations towards the fatigue characteristics of these girders and SCL material itself.

9.2 Summary of the Test

During this research fatigue loading was applied to pre-built T-beam bridge girders made out of both PSL and LVL. In total eight beams were tested in fatigue while four beams were statically loaded up to their failure point. The beams had preservative treatments applied to them using two environmentally friendly agents – Penta Chlorophenol and Copper Naphthenate. For load calculation purposes, a design bridge having two lanes and 40 ft span was selected after a statistical analysis of data obtained from the National Bridge Inventory (NBI). Maximum flexural stress calculated for the 40 ft bridge following the AASHTO LRFD

specification, was simulated on 20 ft long girders. Corresponding maximum and minimum loads were calculated for each cycle of loading which ultimately simulates a truck passing on this girder. The total number of truck loads expected on such a bridge was calculated from statistical analysis of ADTT and future ADTT data from the NBI. In accordance to the truck traffic volume, fatigue loading was applied on each girder for one million cycles at a frequency of 0.9Hz. None of the girders suffered any collapse or serious damage due to this cyclic loading. At the end of fatigue loading of these surviving girders, each of them was loaded continuously until they suffered flexural failure. The intent of this was to identify the extent of damage due to the applied fatigue loading.

Additionally, two more PSL girders were loaded in fatigue at much higher mean stress levels than the previous stresses. While the initial tests were confined to a mean stress level of 20% of average ultimate strength, the stress levels for these two beams were kept at 84.91% and 76.28%. The same laboratory test setup was used for these girders. Close proximity of the mean stress level to their ultimate stresses caused these girders to fail under cyclic loading.

9.3 Conclusions

Fatigue behavior of the SCL girders observed during the current study and relevant concluding remarks are listed below.

- The SCL girders purchased for this project were stored in an open field with plastic sheeting covering each specimen. Because of the long time required for fatigue testing and other delays due to technical difficulties, these girders suffered from cycles of hot and cold weather as well as rainy conditions. Due to repetitive expansion and contraction, the LVL girders experienced cracks at both their web and flange. These cracks occurred where two pieces of LVL were joined together using epoxy as a part of the manufacturing process. Basically, the epoxy was very thinly applied and it vanished at places within a year and a half. However, some six to year old beams of the same construction, lying in the open field without covering, did not have any of these cracks. This signifies that care should be taken in manufacturing these LVL girders so that appropriate amount of glue can be applied. Another possibility may be

to use a different epoxy which is more weather resistant. In a real life bridge, however, such kinds of cracks are less likely to occur. Due to transverse post tensioning the flanges forms a laterally compressed deck system which will not allow the formation of crack due to weathering effects. Relaxation of post tension with time and repetitive loading will, however, produce avenues for this type of crack formation. Cracks at the bottom of the web are also less likely. This is due to the fact that most of the web which lies under the deck, will seldom come into contact with direct sunlight and moisture.

- The cracks introduced at the bottom of the LVL girders deteriorated the ultimate strength of these girders. Test result revealed that the ultimate flexural strength in a severely cracked LVL girder was almost 40% less than a girder of similar build with no cracks. Therefore, care must be taken to prevent these LVL crack formations in an outdoor environment. In fact, this situation may raise a serious serviceability concern and render bridges made out of this type of girder unusable.
- Crack formations at the bottom of the LVL created another phenomenon where the adjacent pieces of LVLs tended to get separated from each other causing a rounded surface at the end of the web. In such a case, the girder became unstable when seated on top of a bearing pad. Repetitive load applications on such girders caused a rotating tendency against the longitudinal axis. This effect may be offset in a bridge as adjacent girders are post tensioned and arranged side by side so as to resist such rotation. However, this scenario may become a concern if one of these cracked girders is subjected to loading in a stand-alone situation. Lateral post tensioning of these girders is also mandatory. Fatigue testing of LVL girders without post tensioning revealed that large vertical cracks had formed at the SCL-epoxy interface within the web.
- For post tensioning purposes of the SCL T-beam girders, commercially available Dywidag bars are adequate. However, there was a significant amount of loss in post tension initially due to gradual compression of the SCL material itself. This again depended on the existing moisture contents within the girder. Higher moisture

contents led to more compression and hence resulted in greater post tension losses. Due to fatigue loading, the post tensioning forces in the bars also got reduced. This reduction was almost linear and was proportional to the number of fatigue loading cycles. For an in-service bridge made with SCL girders, mechanism should be provided to identify periodic loss of post tension in the Dywidag bars. It is also recommended that after certain cycles of truck loading the post tensioning bars should be re-tensioned to regain the original strength of the bridge.

- Except for one severely cracked girder, the ultimate flexural strength of all the LVL and PSL were quite similar. The PSL beams were, however, slightly better because of their inherent density and randomness of fiber originating from the manufacturing process. The ultimate flexural stress of both PSL and LVL were very high with a factor of safety more than double compared to the published allowable design values. This implies that SCL material is very strong and may be able to handle loads larger than those of allowable stress design method. Also because of the decreased variability of the SCL material compared to regular solid sawn lumber attempts may be made to incorporate LRFD loads within the design for this kind of material.
- The longitudinal modulus of elasticity of all the girders was less than the advertised value of 2.0×10^6 psi. Especially with the girders having cracks at the bottom, the value was even lower. Hence, it can be concluded that even though the modulus of elasticity (MOE) of SCL material itself is rated at two million psi, the overall MOE values for the girders are somewhat less.
- In terms of material properties, the two types of SCL girders were not influenced by the cyclic loads originating from the truck traffic. Fatigued girders of both PSL and LVL types did not exhibit any degradation in their residual strength. Actually some of the girders performed better than some fresh girders when loaded up to failure. This implies that these girders with their specific cross-sections were not fatigued at all due to one million cycles of load originating from semi-trailer trucks. It is a possibility that they may be somewhat affected by 10 or 15 million of cycles of such loading. But that

scenario is an unlikely one in real life. It can be safely concluded that within their expected life span these T-beam bridge girders will not be affected by repetitive loading.

- Both PSL and LVL girders exhibited perfectly linear constitutive relationship up to the point of static failure. The failure at ultimate load was brittle. There was no sign of any plastic behavior throughout static loading. Any consideration of non-linear relationship in design or analysis of these girders is unrealistic and should be avoided. Also the brittle tendency of these girders at extremely high loads prohibits the use of these girders in such scenario.
- Overall, the PSL girders seemed to perform better than those made out of LVL. Because of the construction technique adopted in PSL, these girders had higher average moduli of elasticity and ultimate strengths. LVLs, because of their more laminar nature, tended to separate along the layers. In PSL, however, cracks appeared at the cross sections which propagated along the epoxy seams.
- Due to one million cycles of fatigue loading from vehicular trucks there was no physical damage to both PSL and LVL girders. They performed very well even though the applied load was maximized by adopting AASHTO LRFD specifications instead of allowable stress design values. However, the cracks already present in LVL before fatigue loading, grew in size. The cracks at the bottom of LVL girders propagated further along the length.
- There was no pronounced effect in fatigue performances and ultimate strengths of SCL girders due to variation in preservative treatment. The two types of preservatives – Penta Chlorophenol and Copper Naphthenate did not produce any significant effect in results that could be clearly identified to their presence within the SCL girders.
- High strength fatigue loading definitely produced detrimental effect on these girders. As expected in any other material, fatigue lives of the SCL girders were shortened significantly when higher repetitive loads were applied. The girders suffered from

gradual increase in their deflection and ultimately failed in fatigue. A plot of progressive damage accumulation roughly resembled similar effects seen in metals and other materials. Brief amount of time allowed between successive loading and unloading enabled the SCL girders to recuperate some of the damage as in any elastic material. Deflection was low at the beginning and sharply rose towards the end of fatigue life. Complete fatigue failures were generally catastrophic and can cause complete breakage of the cross section. Based on the study the AASHSTO LRFD truck loading is too small to produce this kind of high fatigue loading.

9.4 Finite Element Analysis Outcome

The layered shell element in ANSYS, SOLID186, performed excellently when alternate layers of wood laminates and epoxy were defined. The two distinct layers had their own properties with orthotropic behavior. When the longitudinal modulus of elasticity of the girders was known the element produced good results using the Hindman elastic ratios. The published MOE value of 2 million psi should not be used in the modeling as the overall value of this quantity seemed to be much less for built-up members. Based on the recommended modeling procedure, the SOLID186 element produced deflection results that match mostly within 3% of actual values. Also the same model worked equally well for both LVL and PSL girders. Because of the nature of wood itself, it was not possible to perform fatigue analysis of the SCL girders.

9.5 Recommendation for Future Works

A full fledged S-N curve for any SCL material is not present at this time. Future studies can investigate both LVL and PSL to introduce such curves. A large number of specimens need to be tested at different mean strength levels for this purpose. This can be done on small specimens. Once the full S-N plot is developed an analytical prediction may be made about the fatigue life of SCL T-beam Bridge girders. Experimental fatigue loading of such beam may corroborate the validity of applying such S-N curve in predicting the fatigue life of these girders.

The linear elastic material properties and high ultimate strength of these girders open up the opportunity for further optimization of the design process. Research can be carried out to investigate the viability of regular LRFD based load applications on such girders. Also the application of very large vehicular loads such as tanks and other armored vehicles may be investigated.

For design purposes, the finite element model proposed in this research can be used. The only requirement for this procedure is to identify the longitudinal MOE of the girders. However, this MOE value varies from girder to girder. An extensive testing program may be undertaken where a large number of these girders will be tested to identify a standard MOE value. The girders that need to be tested should vary in span length, cross-section, material type etc. Based on such parametric study a standard equation may be formed that expresses the longitudinal MOE of these girders in terms of these parameters.

Investigations should be carried out to find the extent of damage to LVL girders due to weathering effects. A large number of LVL girders can be weathered for different amount of time and then tested for their static strengths. The type of cracks, de-lamination and reduction of strength can be noted to quantify the damage. Methods should be developed to prevent the cracks that form at the SCL-epoxy interface. One approach towards this goal will be to identify a stronger epoxy that can stand numerous cycles of sun, rain and cold weather before it gives up.

APPENDIX A

DETAILS OF NATIONAL BRIDGE INVENTORY

A.1 National Bridge Inventory Record Format

Table A-1: Format of record in the National Bridge Inventory database

Item No.	Item Name	Item Position	Item Length/Type
1	State Code	1-3	3/N
8	Structure Number	4-18	15/AN
5	Inventory Route	19 - 27	9,/AN
5A	Record Type	19	1/AN
5B	Route Signing Prefix	20	1/N
5C	Designated Level of Service	21	1/N
5D	Route Number	22 - 26	5/AN
5E	Directional Suffix	2.7	. 1/N
2	Highway Agency District	28-29	2/AN
3	County (Parish) Code	30-32	3/N
4	Place Code	33-37	5/N
6	Features Intersected	38 - 62	25/AN
6A	Features Intersected	38 - 61	24/AN
6B	Critical Facility Indicator	62	1/AN
7	Facility Carried By Structure	63 - 80	18/AN
9	Location	81 - 105	25/AN
10	Inventory Rte, Min. Vertical Clearance	106 - 109	4/N
11	Kilometer point	110 - 116	7/N
12	Base Highway Network	117	1/N
13	Inventory Route, Subroute Number	118 - 129	12/AN
13A	LRS Inventory Route	118 - 127	10/AN
13B	Subroute Number	128 - 129	2/N
16	Latitude	130 - 137	8/N
17	Longitude	138 - 146	9/N
19	Bypass/Detour Length	147 - 149	3/N
20	Toll	150	1/N
21	Maintenance Responsibility	151 - 152	2/N

Table A-1 – Continued

Item No.	Item Name	Item Position	Item Length/Type
22	Owner	153 - 154	2/N
26	Functional Class Of Inventory Rte.	155 - 156	2/N
27	Year Built	157 - 160	4/N
28	Lanes On/Under Structure	161 - 164	4/N
28A	Lanes On Structure	161 - 162	2/N
28B	Lanes Under Structure	163 - .164	2/N
29	Average Daily Traffic	165 - 170	6/N
30	Year Of Average Daily Traffic	171 - 174	4/N
31	Design Load	175	1/N
32	Approach Roadway Width	176 - 179	4/N
33	Bridge Median	180	1/N
34	Skew	181 - 182	2/N
35	Structure Flared	183	1/N
3 6	Traffic Safety Features	184 -187	4/AN
36A	Bridge Railings	184	1/AN
3 6B	Transitions	185	1/AN
36C	Approach Guardrail	186	1/AN
3 6D	Approach Guardrail Ends	187	1/AN
37	Historical significance	188.	1/N
38	Navigation Control	189	1/AN
39	Navigation Vertical Clearance	190 - 193	4/N
40	Navigation Horizontal Clearance	194 - 198	5/N
41	Structure Open/Posted/Closed	199	1/AN
42	Type Of Service	200 - 201	2/N
42A	Type of Service On Bridge	200	1/N
42B	Type of Service Under Bridge	201	1/N
43	Structure Type, Main	202 - 204	3/N
43A	Kind of Material/Design	202	1/N
43B	Type of Design/Construction	203 - 204	2/N

Table A-1 – Continued

Item No.	Item Name	Item Position	Item Length/Type
44	Structure Type, Approach Spans	205 - 207	3/N
44A	Kind of Material/Design	205	1/N
44B	Type of Design/Construction	206 - 207	2/N
45	Number Of Spans In Main Unit	208 - 210	3/N
46	Number Of Approach Spans	211 - 214	4/N
47	Inventory Rte Total Horizontal Clearance	215 - 217	3/N
48	Length Of Maximum Span	218 - 222	5/N
49	Structure Length	223 - 228	6/N
50	Curb/Sidewalk Widths	229 - 234	6/N
50A	Left Curb/Sidewalk Width	229 - 231	3/N
50B	Right Curb/Sidewalk Width	232 - 234	3/N
51	Bridge Roadway Width Curb-To-Curb	235 - 238	4/N
52	Deck Width, Out-To-Out	239 - 242	4/N
53	Min Vertical Clear Over Bridge Roadway	243 - 246	4/N
54	Minimum Vertical Underclearance	247 - 251	5/AN
54A	Reference Feature	247	1/AN
54B	Minimum Vertical Underclearance	248 - 251	4/N
55	Min Lateral Underclear On Right	252 - 255	4/AN
55A	Reference Feature	252	1/AN
55B	Minimum Lateral Underclearance	253 - 255	3/N
56	Min Lateral Underclear On Left	256 - 258	3/N
58	Deck	259	1/AN
59	Superstructure	260	1/AN
60	Substructure	261	1/AN
61	Channel/Channel Protection	262	1/AN
62	Culverts	263	1/AN
63	Method Used To Determine Operating Rating	264	1/N
64	Operating Rating	265 - 267	3/N
65	Method Used To Determine Inventory Rating	268	1/N

Table A-1 – Continued

Item No.	Item Name	Item Position	Item Length/Type
66	Inventory. Rating	269 – 271	3/N
61	Structural Evaluation	272	1/AN
68	Deck Geometry	273	1/AN
69	Underclear, Vertical & Horizontal	274	1/AN
70	Bridge Posting	275	1/N
71	Waterway Adequacy	276	1/AN
72	Approach Roadway Alignment	277	1/AN
75	Type of Work	278 - 280	3/N
75A	Type of Work Proposed	278 - 279	2/N
75B	Work Done By	280	1/AN
76	Length Of Structure Improvement	281 - 286	6/N
90	Inspection Date	287 - 290	4/N
91	Designated Inspection Frequency	291 - 292	2/N
92	Critical Feature Inspection	293 - 301	9/AN
92A	Fracture Critical Details	293 - 295	3/AN
92B	Underwater Inspection	296 - 298	3/AN
92C	Other Special Inspection	299 - 301	3/AN
93	Critical Feature Inspection Dates	302 - 313	12/AN
93A	Fracture Critical Details Date	302 - 305	4/AN
93B	Underwater Inspection Date	306 - 309	4/AN
93C	Other Specia.1 Inspection Date	310 - 313	4/AN
94	Bridge Improvement Cost	314 - 319	6/N
95	Roadway Improvement Cost	320 - 325	6/N
96	Total Project Cost	326 - 331	6/N
97	Year Of Improvement Cost Estimate	332 - 335	4/N
98	Border Bridge	336 - 340	5/AN
98A	Neighboring State Code	336 - 338	3/AN
98B	Percent Responsibility	339 - 340	2/N
99	Border Bridge Structure Number	341 - 355	15/AN

Table A-1 – Continued

Item No.	Item Name	Item Position	Item Length/Type
100	STRAHNET Highway Designation	356	1/N
101	Parallel Structure Designation	357	1/AN
102	Direction Of Traffic	358	1/N
103	Temporary Structure Designation	359	1/AN
104	Highway System Of Inventory Route	360	1/N
105	Federal Lands Highways	361	1/N
106	Year Reconstructed	362 - 365	4/N
107	Deck Structure Type	366	1/AN
108	Wearing Surface/Protective System	367 - 369	3/AN
108A	Type of Wearing Surface	367	1/AN
108B	Type of Membrane	368	1/AN
108C	Deck Protection	369	1/AN
109	Average Daily Truck Traffic	370 - 371	2/N
110	Designated National Network	372	1/N
111	Pier/Abutment Protection	373	1/N
112	NBIS Bridge Length	374	1/AN
113	Scour Critical Bridges	375	1/AN
114	Future Average Daily Truck Traffic	376 - 381	6/N
115	Year of Future Average Daily Truck Traffic	382 - 385	4/N
116	Min. Navigation Vertical Clearance	386 - 389	4/N
	Funded Indicator	390	
	Federal Agency Indicator	391	
---	Washington Headquarters Use	392 -427	
	Status	426	
n/a	Asterisk Field in SR	428	1/AN
SR	Sufficiency Rating	429 - 432	4/N

Status Field: 1 = Structurally Deficient; 2 = Functionally Obsolete; 0 = Not Deficient; N = Not Applicable.

APPENDIX B

FATIGUE LOAD CALCULATIONS

B.1 Live Load Moment Calculation for 40ft Girder

Determination of section properties of 40ft girder:

Figure B-1: Cross-section of 40ft SCL Girder

Cross-section of the beam,

$$A_{g\_40} := (2 \cdot 6 \cdot 7.125 + 25 \cdot 7.125) \cdot \text{in}^2$$

$$A_{g\_40} = 263.625 \cdot \text{in}^2$$

Center of gravity of the beam from flange top,

$$c_{t\_40} := \frac{2 \cdot 6 \cdot 7.125 \cdot 3 + 25 \cdot 7.125 \cdot 12.5}{2 \cdot 6 \cdot 7.125 + 25 \cdot 7.125} \cdot \text{in}$$

$$c_{t\_40} = 9.419 \cdot \text{in}$$

Center of gravity of the beam from the bottom,

$$c_{b\_40} := 25 \cdot \text{in} - c_{t\_40}$$

$$c_{b\_40} = 15.581 \cdot \text{in}$$

Moment of inertia of the 40 ft girder

$$I_{g\_40} := \left[2 \cdot \frac{7.125 \cdot 6^3}{12} + 2 \cdot 7.125 \cdot 6 \cdot (9.419 - 3)^2 + \frac{7.125 \cdot 25^3}{12} + 7.125 \cdot 25 \cdot (12.5 - 9.419)^2 \right] \cdot \text{in}^4$$

$$I_{g\_40} = 14748 \cdot \text{in}^4$$

Live Load Moment Calculation:

Span

$$L_{40} := 40 \cdot \text{ft}$$

Figure B-2: HS25 Wheel Loading

Center of gravity of the wheels from the small wheel is, $cg := \frac{40 \cdot \text{kip} \cdot 14 \cdot \text{ft} + 40 \cdot \text{kip} \cdot 28 \cdot \text{ft}}{(10 + 40 + 40) \cdot \text{kip}}$

$$cg = 18.667 \cdot \text{ft}$$

Now the c.g. and the nearest wheel would be equidistant from center line of the girder to produce the maximum moment.

Figure B-3: Placement of Wheel for Maximum Moment

Therefore, the distance $x := \frac{cg - 14 \cdot \text{ft}}{2}$

$$x = 2.333 \cdot \text{ft}$$

At the location where the first 40 kip wheel sits, the ordinate is
$$o_1 := \frac{0.25 \cdot \left(\frac{L_{40}}{2} - x \right) \cdot L_{40}}{\frac{L_{40}}{2}}$$

At the location where the 10 kip wheel sits the ordinate is
$$o_2 := \frac{o_1}{\left(\frac{L_{40}}{2} - x \right)} \cdot \left(\frac{L_{40}}{2} - x - 14 \cdot \text{ft} \right)$$

At the location where the second 40 kip wheel sits, the ordinate is

$$o_3 := \frac{o_1}{\left(\frac{L_{40}}{2} - x \right)} \cdot \left(\frac{L_{40}}{2} - 14 \cdot \text{ft} + x \right)$$

Maximum moment due to truck, $M_{\text{truck}} := o_2 \cdot 10 \cdot \text{kip} + o_1 \cdot 40 \cdot \text{kip} + o_3 \cdot 40 \cdot \text{kip}$

$$M_{\text{truck}} = 538 \cdot \text{kip} \cdot \text{ft}$$

Figure B-4: Placement of Wheels for Tandem Truck

$$\text{Moment due to tandem, } M_{\text{tandem}} := 2 \cdot \left[\frac{0.25}{\frac{L_{40}}{2}} \cdot \left(\frac{L_{40}}{2} - 2 \cdot \text{ft} \right) \cdot L_{40} \cdot 25 \cdot \text{kip} \right]$$

$$M_{\text{tandem}} = 450 \cdot \text{kip} \cdot \text{ft}$$

Figure B-5: AASHTO LRFD Fatigue Truck

$$\text{Moment due to fatigue truck, } M_{\text{fatigue}} := \left[40 \cdot \text{kip} \cdot 0.25 + \frac{0.25}{\frac{L_{40}}{2}} \cdot \left[\left(\frac{L_{40}}{2} - 14 \cdot \text{ft} \right) \cdot 10 \cdot \text{kip} \right] \right] \cdot L_{40}$$

$$M_{\text{fatigue}} = 430 \cdot \text{kip} \cdot \text{ft}$$

$$\text{Moment due to lane load, } M_{\text{lane}} := \frac{640 \cdot \frac{\text{lb}}{\text{ft}} \cdot L_{40}^2}{8} \quad M_{\text{lane}} = 128 \cdot \text{kip} \cdot \text{ft}$$

Live load Moment Combinations:

$$\text{Single Truck + Lane Load, } M_{\text{S\_L}} := 1.33 \cdot M_{\text{truck}} + M_{\text{lane}} \quad M_{\text{S\_L}} = 844 \cdot \text{kip} \cdot \text{ft}$$

$$\text{Tandem Truck + Lane Load, } M_{\text{T\_L}} := M_{\text{tandem}} \cdot 1.33 + M_{\text{lane}} \quad M_{\text{T\_L}} = 727 \cdot \text{kip} \cdot \text{ft}$$

$$\text{Fatigue Truck + Lane Load, } M_{\text{F\_L}} := M_{\text{fatigue}} \cdot 1.15 + M_{\text{lane}} \quad M_{\text{F\_L}} = 623 \cdot \text{kip} \cdot \text{ft}$$

$$\text{Governing live load moment, } M_{\text{LL\_40}} := \max(M_{\text{S\_L}}, M_{\text{T\_L}}, M_{\text{F\_L}})$$

$$M_{\text{LL\_40}} = 844 \cdot \text{kip} \cdot \text{ft}$$

B.2 Dead Load Moment Calculation for 40ft Girder

Self weight of the beam,

$$w_{\text{DL\_40}} := 0.292 \cdot \frac{\text{lb}}{\text{ft} \cdot \text{in}^2} \cdot A_{\text{g\_40}}$$

$$w_{\text{DL\_40}} = 76.978 \cdot \frac{\text{lb}}{\text{ft}}$$

Dead load moment,

~

$$M_{DL\_40} := \frac{w_{DL\_40} \cdot L_{40}^2}{8}$$

$$M_{DL\_40} = 15.396 \cdot \text{kip} \cdot \text{ft}$$

B.3 Total Moment on 40ft Girder

Distribution factor for live load moment is

For one design lane loaded,

$$DFM_1 := \frac{1.781}{9.2}$$

$$DFM_1 = 0.194$$

For tow or more design lane loaded,

$$DFM_2 := \frac{1.781}{9.0}$$

$$DFM_2 = 0.198$$

Therefore distribution factor is,

$$DFM := \max(DFM_1, DFM_2)$$

$$DFM = 0.198$$

Total maximum moment

$$M_{40} := M_{DL\_40} + DFM \cdot M_{LL\_40}$$

$$M_{40} = 182.411 \cdot \text{kip} \cdot \text{ft}$$

B.4 Calculation of stresses in 40ft Girder

Stress at the bottom due to dead load only,

$$\sigma_{DL\_40} := \frac{M_{DL\_40} \cdot c_{b\_40}}{I_{\sigma\_40}}$$

$$\sigma_{DL\_40} = 195 \cdot \text{psi}$$

Maximum stress due to total moment,

$$\sigma_{\max\_40} := \frac{M_{40} \cdot c_{b\_40}}{I_{\sigma\_40}}$$

$$\sigma_{\max\_40} = 2313 \cdot \text{psi}$$

Stress ratio is

$$R_{40} := \frac{\sigma_{DL\_40}}{\sigma_{\max\_40}}$$

$$R_{40} = 0.084$$

B.5 Fatigue Load Calculation for 20 ft Girder

Figure B-6: Cross-section of 20 ft Span Girder

Cross-section of the beam,

$$A_{g\_20} := (2 \cdot 6 \cdot 7.125 + 15.75 \cdot 7.125) \cdot \text{in}^2$$

$$A_{g\_20} = 197.719 \cdot \text{in}^2$$

Center of gravity of the beam from flange top,

$$c_{t\_20} := \frac{2 \cdot 6 \cdot 7.125 \cdot 3 + 15.75 \cdot 7.125 \cdot 7.875}{2 \cdot 6 \cdot 7.125 + 15.75 \cdot 7.125} \cdot \text{in}$$

$$c_{t\_20} = 5.767 \cdot \text{in}$$

Center of gravity of the beam from the bottom,

$$c_{b\_20} := 15.75 \cdot \text{in} - c_{t\_20}$$

$$c_{b\_20} = 9.983 \cdot \text{in}$$

Moment of inertia of the 20 ft girder

$$I_{g\_20} := \left[2 \cdot \frac{7.125 \cdot 6^3}{12} + 2 \cdot 7.125 \cdot 6 \cdot (5.767 - 3)^2 + \frac{7.125 \cdot 15.75^3}{12} + 7.125 \cdot 15.75 \cdot (7.875 - 5.767)^2 \right] \cdot \text{in}^4$$

$$I_{g\_20} = 3730 \cdot \text{in}^4$$

Span of 20 ft girder is

$$L_{20} := 20 \cdot \text{ft}$$

Self weight of the beam,

$$w_{DL\_20} := 0.292 \cdot \frac{\text{lb}}{\text{ft} \cdot \text{in}^2} \cdot A_{g\_20}$$

$$w_{DL\_20} = 57.734 \cdot \frac{\text{lb}}{\text{ft}}$$

Dead load moment,

$$M_{DL\_20} := \frac{w_{DL\_20} \cdot L_{20}^2}{8}$$

$$M_{DL\_20} = 2.887 \cdot \text{kip} \cdot \text{ft}$$

Tensile stress in 20 ft girder due to dead load only,

$$\sigma_{DL\_20} := \frac{M_{DL\_20} \cdot c_{b\_20}}{I_{g\_20}}$$

$$\sigma_{DL\_20} = 92.724 \cdot \text{psi}$$

To simulate the maximum stress of 40 ft girder, additional stress required is,

$$\sigma_{\text{required}} := \sigma_{\text{max\_40}} - \sigma_{DL\_20}$$

$$\sigma_{\text{required}} = 2220 \cdot \text{psi}$$

Corresponding additional moment requirement is,

$$M_{\text{required}} := \frac{\sigma_{\text{required}} \cdot I_{g\_20}}{c_{b\_20}}$$

$$M_{\text{required}} = 69.111 \cdot \text{kip} \cdot \text{ft}$$

An arrangement similar to third point loading will be used to simulate the moment. Assuming a total span length of 20 ft with the two loading point at the middle will be 4 ft apart, the actuator must apply a load P, where

Load required

$$P_{\text{req}} := \frac{M_{\text{required}}}{4 \cdot \text{ft}}$$

$$P_{\text{req}} = 17.278 \cdot \text{kip}$$

Therefore, in each fatigue cycle the actuator must apply a load of 17.28 kip to simulate the effect of an HS-25 truck passing on top of a 40 ft span SCL Girder.

Since the 40 ft girder has higher dead load, additional loading may be required on top the 20 ft girder to simulate the dead load only moment on the larger girder. This loading is denoted as the lowest peak of each loading cycle.

$$M_{DL\_required} := \frac{(\sigma_{DL\_40} - \sigma_{DL\_20}) \cdot I_{g\_20}}{c_{b\_20}}$$

$$M_{DL\_required} = 3.19 \cdot \text{kip} \cdot \text{ft}$$

$$\text{Load required, } P_{\text{req\_for\_DL\_moment}} := \frac{M_{\text{DL\_required}}}{4 \cdot \text{ft}} \quad P_{\text{req\_for\_DL\_moment}} = 0.8 \cdot \text{kip}$$

Expected maximum deflection in each cycle is,

$$\Delta := \frac{P_{\text{req}} \cdot 8 \cdot \text{ft} \cdot \left[3 \cdot L_{20}^2 - 4 \cdot (8 \cdot \text{ft})^2 \right]}{24 \cdot 2000000 \cdot \text{psi} \cdot I_{g\_20}} \quad \Delta = 1.259 \cdot \text{in}$$

During actual testing, it was found out that for fatigue loading maximum span can only be 16 ft. The low and high peak in each cycle was required to be modified.

$$\text{Highest load in each cycle, } P_{\text{High}} := \frac{M_{\text{required}}}{3 \cdot \text{ft}} \quad P_{\text{High}} = 23.037 \cdot \text{kip}$$

$$\text{Lowest load in each cycle, } P_{\text{Low}} := \frac{M_{\text{DL\_required}}}{3 \cdot \text{ft}} \quad P_{\text{Low}} = 1.063 \cdot \text{kip}$$

Corresponding deflection is,

$$\Delta_{\text{high}} := \frac{P_{\text{High}} \cdot 6 \cdot \text{ft} \cdot \left[3 \cdot L_{20}^2 - 4 \cdot (6 \cdot \text{ft})^2 \right]}{24 \cdot 2000000 \cdot \text{psi} \cdot I_{g\_20}} \quad \Delta_{\text{high}} = 0.832 \cdot \text{in}$$

APPENDIX C

POST TENSION CALCULATIONS

C.1 Calculation of Post Tension Forcd and Bar Diameter

Moisture content factor,

$$C_M := 1.00$$

Modulus of elasticity,

$$E' := C_M \cdot 1600 \cdot \text{ksi}$$

$$E' = 1.6 \times 10^3 \cdot \text{ksi}$$

For Douglas Fir, Transverse modulus of elasticity,

$$E_{TS} := 0.013 \cdot E'$$

$$E_{TS} = 20.8 \cdot \text{ksi}$$

Transverse shear modulus,

$$G_{TS} := 0.03 \cdot E'$$

$$G_{TS} = 48 \cdot \text{ksi}$$

For no butt joints, butt joint factor,

$$C_B := 1.00$$

Deck width,

$$b := 25 \cdot \text{ft}$$

Deck span,

$$L := 19 \cdot \text{ft}$$

Deck thickness, $t := 6 \cdot \text{in}$

Minimum ultimate tensile stress of steel, $f_{pu} := 150000 \cdot \text{psi}$

Coefficient of friction for surfaced lumber,

$$\mu := 0.35$$

SCL compressive strength,

$$f_c := 750 \cdot \text{psi}$$

Allowable bending stress of steel plate,

$$F_b := 0.55 \cdot 36 \cdot \text{ksi}$$

$$\alpha := \frac{2 \cdot G_{TS}}{\sqrt{E' \cdot C_B \cdot E_{TS}}} \quad \alpha = 0.526$$

$$\theta := \frac{b}{2 \cdot L} \cdot \left(\frac{E' \cdot C_B}{E_{TS}} \right)^{0.25} \quad \theta = 1.948$$

$$\beta := \pi \cdot \left(\frac{b}{L} \right) \cdot \sqrt{\frac{E' \cdot C_B}{2 \cdot G_{TS}}} \quad \beta = 16.876$$

From fig 9-16. transverse moment,

$$M_T := 500 \cdot 1.25 \cdot \text{in} \cdot \frac{\text{lb}}{\text{in}}$$

$$M_T = 625 \cdot \text{in} \cdot \frac{\text{lb}}{\text{in}}$$

From fig 9-17. transverse shear,

$$V_T := 84 \cdot 1.25 \cdot \frac{\text{lb}}{\text{in}}$$

$$V_T = 105 \cdot \frac{\text{lb}}{\text{in}}$$

Minimum uniform compressive prestress in service,

$$N := \max \left[(40) \cdot \frac{\text{lb}}{\text{in}^2}, \frac{6 \cdot M_T}{t^2}, \frac{1.5 \cdot V_T}{t \cdot \mu} \right]$$

$$N = 104.167 \cdot \frac{\text{lb}}{\text{in}^2}$$

$$A_s := \frac{N \cdot 2.67 \cdot \text{ft} \cdot t}{0.70 \cdot f_{pu}}$$

$$A_s = 0.191 \cdot \text{in}^2$$

We shall use @5/8 rod.

$$A_s := 0.28 \cdot \text{in}^2$$

$$\frac{A_s}{2.67 \cdot \text{ft} \cdot t} = 0.0015 < 0.0016 \text{ OK}$$

Prestressing force,

$$F_{ps} := N \cdot 2.67 \cdot \text{ft} \cdot t$$

$$F_{ps} = 20.025 \cdot \text{kip}$$

C.2 Design of Base Plate

Minimum bearing plate area,

$$A_p := \frac{F_{ps}}{f_c}$$

$$A_p = 26.7 \cdot \text{in}^2$$

Considering 5 inch plate width, plate length,

$$L_p := \frac{A_p}{5 \cdot \text{in}}$$

$$L_p = 5.34 \cdot \text{in}$$

Assuming a plate size of 6 in. by 5 in.

$$L_p := 6 \cdot \text{in}$$

$$\frac{L_p}{5 \cdot \text{in}} = 1.2 \text{ OK}$$

Final size for anchorage plate is 6 in. by 5 in. by 0.5 in.

C.3 Design of Anchorage Plate

Assuming anchorage plate size is 3 in. by 3 in. by 0.75 in

$$W_A := 3 \cdot \text{in} \quad L_A := 3 \cdot \text{in} \quad t_A := 0.75 \cdot \text{in}$$

$$W_p := 5 \cdot \text{in} \quad L_p = 6 \cdot \text{in} \quad [\text{Base plate dimensions}]$$

$$k := \max\left(\frac{W_p - W_A}{2}, \frac{L_p - L_A}{2}\right) \quad k = 1.5 \cdot \text{in}$$

$$f_{c\_actual} := \frac{F_{ps}}{W_p \cdot L_p} \quad f_{c\_actual} = 667.5 \cdot \text{psi}$$

$$t_p := \sqrt{\frac{3 \cdot f_{c\_actual} \cdot k^2}{F_b}} \quad t_p = 0.477 \cdot \text{in} < 0.75 \text{ in.} \quad \text{OK}$$

Therefore, Final size of anchorage plate is 3 in. by 3 in. by 3/4 in.

APPENDIX D

RESULTS AND PLOTS

D.1 Load Vs. Deflection Plots for Fresh SCL Girders

Figure D-1: Load vs. deflection Plot for LVLC1

Figure D-2: Load Vs. Deflection Plot for LVLP1

Figure D-3: Load vs. Deflection Plot for PSLC1

Figure D-4: Load vs. deflection Plot for PSLP1

D.2 Load Vs. Deflection Plot for Fatigued Girders

Figure D-5: Load vs. Deflection Plot for LVLC2

Figure D-6: Load vs. Deflection Plot for LVLC3

Figure D-7: Load vs. Deflection Plot for LVLP2

Figure D-8: Load vs. Deflection Plot for LVLP3

Figure D-9: Load vs. Deflection Plot for PSLC2

Figure D-10: Load vs. Deflection Plot for PSLC3

Figure D-11: Load vs. Deflection Plot for PSLP2

Figure D-12: Load vs. Deflection Plot for PSLP3

D.3 Loss of Post Tension Force Plots

Figure D-13: Loss of Post Tension Force in LVLC3

Figure D-14: Loss of Post Tension Force in LVLP3

Figure D-15: Loss of Post Tension Force in PSLP2

Figure D-16: Loss of Post Tension Force in PSLP3

D.4 Various Pictures of SCL Girders

Figure D-17: Severe De-lamination of LVLC1 due to Weathering

Figure D-18: Cracks at the Web of LVLC1 after Static Failure

Figure D-19: Crack propagation at SCL-epoxy interface of LVLC2

Figure D-20: Failure crack at the side of LVLC2 Web

Figure D-21: Localizing crushing of LVLC3 at flange due to Post Tension Force

Figure D-22: Longitudinal Cracks at the Bottom of Web of LVLC3

Figure D-23: Length-wise Crack in LVLP2 due to Fatigue Loading without Post Tension

Figure D-24: Crack Through out the Whole Section of LVLP2

Figure D-25: Localized Failure of LVLP3 web due to Static Loading

Figure D-26: Failure due to Static Load on PSLP2

Figure D-27: Minor Crack on Flange of PSLC1 due to weathering

Figure D-28: Irregular Crack at PSLC1 Web due to Static Loading

Figure D-29: Failure Crack at the Web of PSLC2

Figure D-30: Solid Cross-section of PSLC2 despite weathering

Figure D-31: Localized Static Crack at the Bottom of PSLC3

Figure D-32: Close Shot of Crack at the Web of PSLC3

APPENDIX E

FINITE ELEMENT ANALYSIS RESULTS

E.1 Script for Finite Element Analysis in ANSYS

```
!*****!  
!  
! FATIGUE EVALUATION OF STRUCTURAL COMPOSITE LUMBER (SCL) !  
! T-BEAM BRIDGE GIRDERS !  
!  
! By !  
! Muhammad Badre Enam !  
!  
!*****!  
!  
!This is the ANSYS script file that was used to produce the !  
!finite element model that was used in chapter 8. This script!  
!also generates the boundary conditions, applies the load and!  
!post tesnioning force. Once load is applied to the girder, !  
!the script solves it and automatically produces a contour !  
!plot for vertical deflections. !  
!  
!*****!  
!  
!USAGE: In ANSYS click File > Read Input From... and then !  
! select the text file which includes this code. !  
!  
!*****!  
  
!*** Initial setup code, Clears previous run results  
  
FINISH  
/CLEAR, START  
/PREP7  
/UNITS, BIN  
  
!*** Setting up of paramaters, mainly E-longitudinal and  
!*** elastic ratios.  
  
*SET, PTENSION, 10000 !*** Post Tension Load  
*SET, STATICLOAD, 60755 !*** Applied Load  
*SET, ELONGITUDINAL, 1.79e6 !*** Longitudinal MOE  
*SET, ERADIAL, ELONGITUDINAL/36.4  
*SET, ETANGENTIAL, ELONGITUDINAL/36.4  
*SET, GRADIALTANGENTIAL, ELONGITUDINAL/398.68  
*SET, GLONGITUDIANLTANGENTIAL, ELONGITUDINAL/55.02  
*SET, GRADIALLONGITUDINAL, ELONGITUDINAL/45.1  
  
!*** Creating the first cross-section, a Tee  
  
K,, 0, 15.75  
K,, 0, 9.75  
K,, 7.125, 9.75  
K,, 7.125, 0  
K,, 14.25, 0  
K,, 14.25, 9.75  
K,, 21.375, 9.75
```

```

K,,21.375,15.75
A,1,2,3,4,5,6,7,8

!*** Creating second cross-section at 20 ft away

VOFFST,1,240

!*** Defining element for SCL, layered solid

ET,1,SOLID186

KEYOPT,1,2,0
KEYOPT,1,6,0
KEYOPT,1,10,0
keyopt,1,3,1

!*** Direction Z - Longitudinal
!*** Direction X - Radial
!*** Direction Y - Tangential

!*** Defining material properties for wood laminates

MPTEMP,1,0
MPDATA,EX,1,,ERADIAL
MPDATA,EY,1,,ETANGENTIAL
MPDATA,EZ,1,,ELONGITUDINAL
MPDATA,PRXY,1,,0.39
MPDATA,PRYZ,1,,0.036
MPDATA,PRXZ,1,,0.029
MPDATA,GXY,1,,GRADIALTANGENTIAL
MPDATA,GYZ,1,,GLONGITUDIANLTANGENTIAL
MPDATA,GXZ,1,,GRADIALLONGITUDINAL

!*** Defining material properties for epoxy

MP,EX,2,2e6
MP,PRXY,0.3

!FC,1,EPEL,XTEN,0.0032
!FC,1,EPEL,YTEN,0.0044
!FC,1,EPEL,ZTEN,0.0087
!FC,1,EPEL,XCMP,-0.0063
!FC,1,EPEL,YCMP,-0.0086
!FC,1,EPEL,ZCMP,-0.0039
!FC,1,EPEL,XY,0.1117
!FC,1,EPEL,YZ,0.1117
!FC,1,EPEL,XZ,0.1117

!*** Defining alternate layers in shell element as
!*** wood laminates and epoxy

sect,1,shell,,

```

```

secdata, .06,1,0.0,3
secdata, .005,1,0.0,3
secdata, .12,1,0.0,3
secdata, .005,1,0.0,3
secdata, .12,1,0.0,3
secdata, .005,1,0.0,3
secdata, .12,1,0.0,3
secdata, .005,1,0.0,3
secdata, .12,1,0.0,3
secdata, .005,1,0.0,3
secdata, .12,1,0.0,3
secdata, .005,1,0.0,3
secdata, .12,1,0.0,3
secdata, .005,1,0.0,3
secdata, .12,1,0.0,3
secdata, .005,1,0.0,3
secdata, .12,1,0.0,3
secdata, .005,1,0.0,3
secdata, .12,1,0.0,3
secdata, .005,1,0.0,3
secdata, .12,1,0.0,3
secdata, .005,1,0.0,3
secdata, .12,1,0.0,3
secdata, .005,1,0.0,3
secdata, .12,1,0.0,3
secdata, .005,1,0.0,3
secdata, .12,1,0.0,3
secdata, .005,1,0.0,3
secdata, .12,1,0.0,3
secdata, .005,1,0.0,3
secdata, .06,1,0.0,3
secoffset,MID
seccontrol,,,, , , ,
SAVE,'cantilever','db','C:\DOCUME~1\ADMINI~1\'

```

```

!*** Generating the volume of SCL girder
ESIZE,3,0,
VSWEEP,1

```

```

!*** Applying support boundary conditions
NSEL,S,LOC,Z,6
NSEL,R,LOC,Y,0

```

```

D,ALL,ALL

```

```

NSEL,S,LOC,Z,234
NSEL,R,LOC,Y,0

```

```

D,ALL,ALL

```

```

!*** Applying the static load from the actuator

```

```

NSEL,S,LOC,Z,96
NSEL,A,LOC,Z,144
NSEL,R,LOC,Y,15.75
*GET,NODECOUNT,NODE,0,COUNT

F,ALL,FY,-STATICLOAD/NODECOUNT

!*** Applying post tension force at selected nodes

NSEL,ALL

*DO,I,15,225,30
NSEL,S,LOC,X,0
NSEL,R,LOC,Y,12.75
NSEL,R,LOC,Z,I
F,ALL,FX,1*PTENSION
NSEL,ALL
*ENDDO

*DO,I,15,225,30
NSEL,S,LOC,X,21.375
NSEL,R,LOC,Y,12.75
NSEL,R,LOC,Z,I
F,ALL,FX,-1*PTENSION
NSEL,ALL
*ENDDO

!*** Solve for the applied loads

/SOLU
SOLVE

!*** Enter post processor

/POST1
SET, LAST

!*** Plot the contour diagram for vertical deflections

PLNSOL, U,Y, 0,1.0

!*** End of Script

!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!

```

E.2 Deflection Contour Plots

Figure E-1: Deflection Contour Plot for LVLC1

Figure E-2: Deflection Contour Plot for LVLC2

Figure E-3: Deflection Contour Plot for LVLC3

Figure E-4: Deflection Contour Plot for LVL P1

Figure E-5: Deflection Contour Plot for LVLP2

Figure E-6: Deflection Contour Plot for LVLP3

Figure E-7: Deflection Contour Plot for PSLC1

Figure E-8: Deflection Contour Plot for PSLC2

Figure E-9: Deflection Contour Plot for PSLC3

Figure E-10: Deflection Contour Plot for PSLP1

Figure E-11: Deflection Contour Plot for PSLP2

Figure E-12: Deflection Contour Plot for PSLP3

E.3 Strain Contour Plots

Figure E-13: Strain Contour Plot for LVLC1

Figure E-14: Strain Contour Plot for LVLC2

Figure E-15: Strain Contour Plot for LVLC3

Figure E-16: Strain Contour Plot for LVL P1

Figure E-17: Strain Contour Plot for LVLP2

Figure E-18: Strain Contour Plot for LVLP3

Figure E-19: Strain Contour Plot for PSLC1

Figure E-20: Strain Contour Plot for PSLC2

Figure E-21: Strain Contour Plot for PSLC3

Figure E-22: Strain Contour Plot for PSLP1

Figure E-23: Strain Contour Plot for PSLP2

Figure E-24: Strain Contour Plot for PSLP3

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BIOGRAPHICAL SKETCH

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