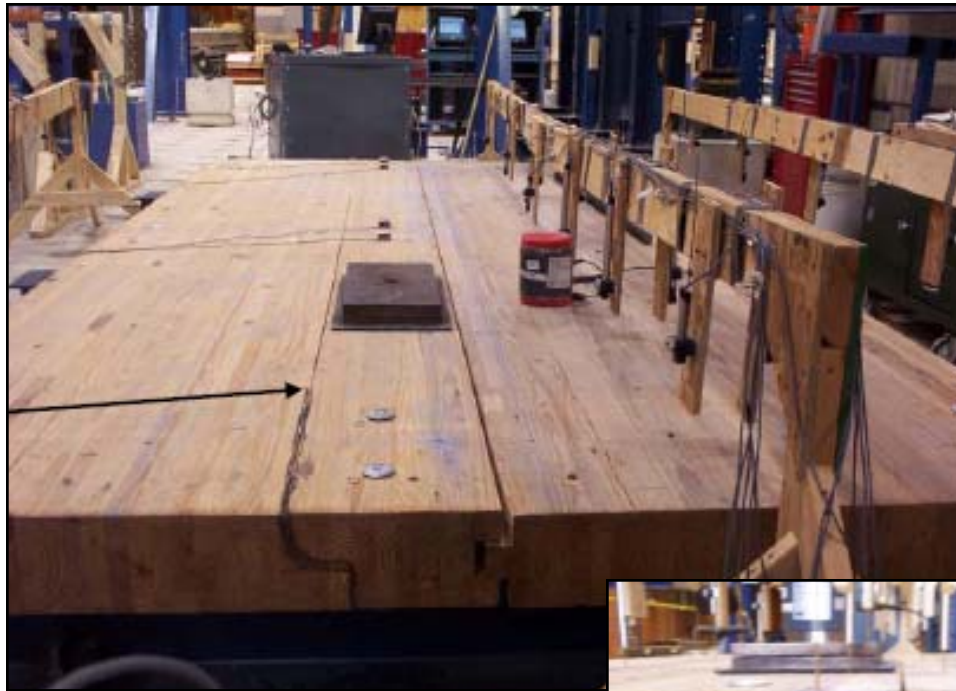


Temporary Bridge Deck Panels

Final Report
March 2009



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<p>The research described here has resulted in the development of a simplistic and innovative interlocking glulam bridge deck panel for use in temporary bridge deck applications. The key feature of this panel system is the interlocking tongue and groove joint. The joint provides both displacement compatibility and structural load transfer between panels. The unique structural tongue and groove design successfully addresses two common issues with existing transverse glulam bridge decks:</p> <ol style="list-style-type: none"> 1. It reduces differential movement between panels which is a very common problem with noninterconnected glulam deck systems in use today. 2. It allows the deck system to be installed completely from the top, eliminating the need for workers to install bolts from underneath the bridge deck. <p>Other design advantages of the panels include: prefabrication for rapid on site installation, reusability, and because they are made from wood, the panels are relatively lightweight and have a relatively high strength to weight ratio when compared to steel and concrete alternatives.</p>					
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Project 163**

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Executive Summary

With a large number of our nation bridges in need of replacement or refurbishment there will be a greater demand for temporary bridges in the near future. Most state agencies and individual State Departments of Transportation (DOTs) establish their own design guidelines for temporary bridges. Taylor et al. (1995) reports that there is little previous research that has been conducted on appropriate design procedures for portable bridges for low-volume roads. This indicates that continued research should be conducted on temporary bridges to develop nationally recognized design criteria for these types of structures.

Non-interconnected glulam decks are the most widely used type of glulam decks in modern timber construction. They require little fabrication, and are easy to install by unskilled labor and without special equipment, resulting in relatively low cost. Since the panels are not connected to one another, each panel acts separately to resist the stresses and deflection from applied loads. The design of these types of bridge deck systems is often controlled by the differential displacement between panels. If not limited, differential displacement between adjacent panels can lead to premature cracking of the asphalt wearing surface. One obvious solution to this problem is to interconnect the panels, however, to date most interconnected systems are very difficult and cumbersome to erect on site. The technology described here is an interconnected deck system that can be installed in essentially the same manner as a non-interconnected deck system.

The research described here has resulted in the development of a simplistic and innovative interlocking glulam bridge deck panel for use in temporary bridge deck applications. The key feature of this panel system is the interlocking tongue and groove joint. The joint provides both displacement compatibility and structural load transfer between panels. The unique structural tongue-and-groove design successfully addresses two common issues with existing transverse glulam bridge decks:

1. It reduces differential movement between panels which is a very common problem with non-interconnected glulam deck systems in use today.
2. It allows the deck system to be installed completely from the top, eliminating the need for workers to install bolts from underneath the bridge deck.

Other design advantages of the panels include: prefabrication for rapid on-site installation, reusability, and because they are made from wood, the panels are relatively lightweight and have a relatively high strength-to-weight ratio when compared to steel and concrete alternatives.

The overall goal of this experimental program was to evaluate the adequacy of the interlocking tongue and groove joint and the limited number of bolts/deck clips when subjected to AASHTO HS-25-44 live loading. The experimental program included three different sets of tests: Modulus of Elasticity (MOE) testing, full scale static testing and full scale fatigue testing.

The static test results show that:

- Differential panel deflection is within acceptable limits for HS-25 Loading
- Uplift force on bolts are minimal and can be neglected in design
- The Ultimate load range from 3.2 to 4.5 times the AASHTO HS-25 Loading. These Safety Factors are comparable to essential structure, such as hospitals, schools and high volume traffic bridges.
- Design sizing of the panels will likely be controlled by either differential deflection limits between panels or overall panel deflection limits rather than strength.
- When tested to failure the deck panels exhibit a progressive pseudo-ductile failure.

The fatigue Results show:

- No significant stiffness loss or hysteretic damage accumulation after 2 million design load cycles (50 yr life span)
- Residual strength and stiffness of fatigued panels is approximately the same as unfatigued strength and stiffness (no strength or stiffness degradation).
- Failure modes of fatigue panels are the same as those of unfatigued panels indicating that there are no fatigue sensitive design details.

Recommendations for future work are follows:

- Investigate the environmental durability of the decking system
- Conduct a demonstration project
- Investigate a wearing surface and a railing system for the decks
- Investigate the possibility for this decking system to be used for permanent bridge deck applications
- Develop a simplistic user friendly design tool which may be in the form of a model or design tables.
- Work with industry to develop an automated manufacturing process for the tongue and groove system.

Introduction

This project focused on the development and experimental evaluation of interlocking glulam panels for use as temporary bridge deck panels. The innovative panels feature an interlocking tongue and groove joint which is intended to provide both displacement compatibility and structural load transfer between panels. The unique structural tongue-and-groove addresses two common issues with existing transverse glulam bridge decks:

1. It reduces or eliminates differential movement between panels which is a very common problem with non-interconnected glulam deck systems in use today.
2. It allows the deck system to be installed completely from the top, eliminating the need for workers to install bolts from underneath the bridge deck.

Other design advantages of the panels include: prefabrication for rapid on-site installation, reusability, and because they are made from wood, the panels are relatively lightweight and have a relatively high strength-to-weight ratio when compared to steel and concrete alternatives.

Evaluation of the deck system feasibility focused on both the behavior of the panel-to-panel connection (tongue and groove connection) and the panel-to-girder connection through static and fatigue load testing. An FEA model was also developed to more fully understand panel behavior and aid in future design models.

Background

The nation's 587,964 bridges are critical to our transportation system, allowing people and goods to move around the country in a safe and efficient manner (TRIP, 2002). During the 1960's bridge construction was at its' peak, thus indicating that bridges in the United States are getting older, with average age reaching 40 years. Based on The Road Information Program (TRIP), 28 percent of the nation's bridges are rated as structurally deficient or functionally obsolete. Of those, 14 percent are rated as structural deficient and another 14 percent are rated as functionally obsolete. A bridge is structurally deficient if there is significant deterioration of the bridge deck, supports or other major components. Bridges that are functionally obsolete no longer meet current highway design standards, often because of narrow lanes, inadequate under clearance or poor alignment, all of which reduce highway safety (TRIP, 2002).

With a large number of our nation bridges in need of replacement or refurbishment there will be a greater demand for temporary bridges in the future. Most state agencies and individual State

Departments of Transportation (DOTs) establish their own design guidelines for temporary bridges. Taylor et al. (1995) reports that there is little previous research that has been conducted on appropriate design procedures for portable bridges for low-volume roads. This indicates that continued research should be conducted on temporary bridges to develop nationally recognized design criteria for these types of structures.

Non-interconnected glulam decks are the most widely used type of glulam decks in modern timber construction. They require little fabrication, and are easy to install by unskilled labor and without special equipment, resulting in relatively low cost. Since the panels are not connected to one another, each panel acts separately to resist the stresses and deflection from applied loads. The design of these types of bridge deck systems is often controlled by the differential displacement between panels. If not limited, differential displacement between adjacent panels can lead to premature cracking of the asphalt wearing surface. One obviously solution to this problem is to interconnect the panels, however, to date most interconnected systems are very difficult and cumbersome to erect on site. The technology described here is an interconnected deck system that can be installed in essentially the same manner as a non-interconnected deck system.

Preliminary Design

The transverse interconnected glulam deck was designed in accordance with AASHTO HS25-44 truck loading using traditional glulam deck design methodology. The magnitude of the HS25 truck wheel load is 89 kN (20 kips). The AASHTO bridge code, Article 3.30 specifies that the wheel load is to be a patch load distributed over a tire contact area of 508 mm by 254 mm (20" by 10"). The long side of the tire contact area is kept parallel to the direction of laminates (transverse direction) and the short side is kept parallel to the direction of the girders (longitudinal) to simulate traffic flow for this type of decking system.

For preliminary design of the panel-to-panel connection it was assumed that the connection would fail in pure shear. Thus, each panel's thickness was divided into three equal parts to distribute the shear stress equally between the thicknesses of the tongue and groove, as illustrated in Figure 1.

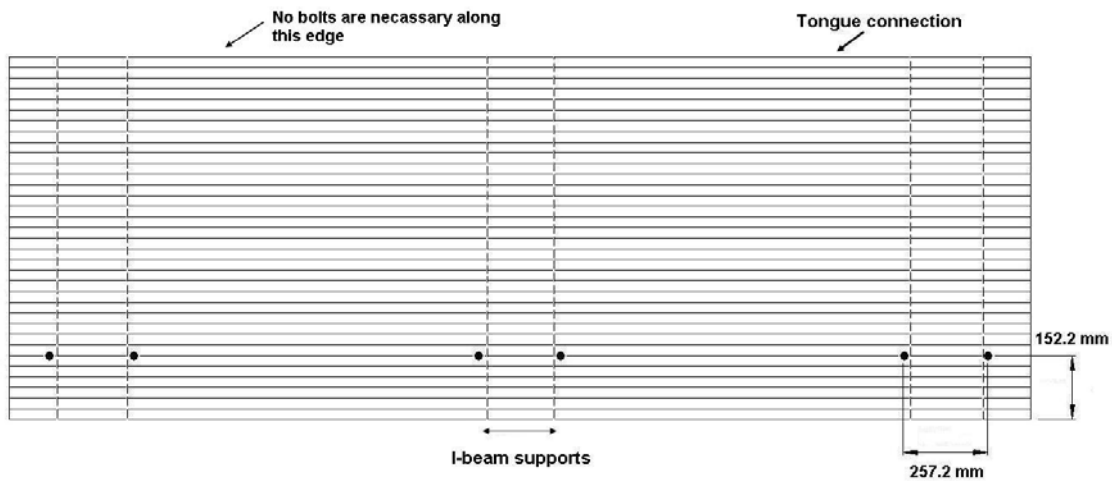
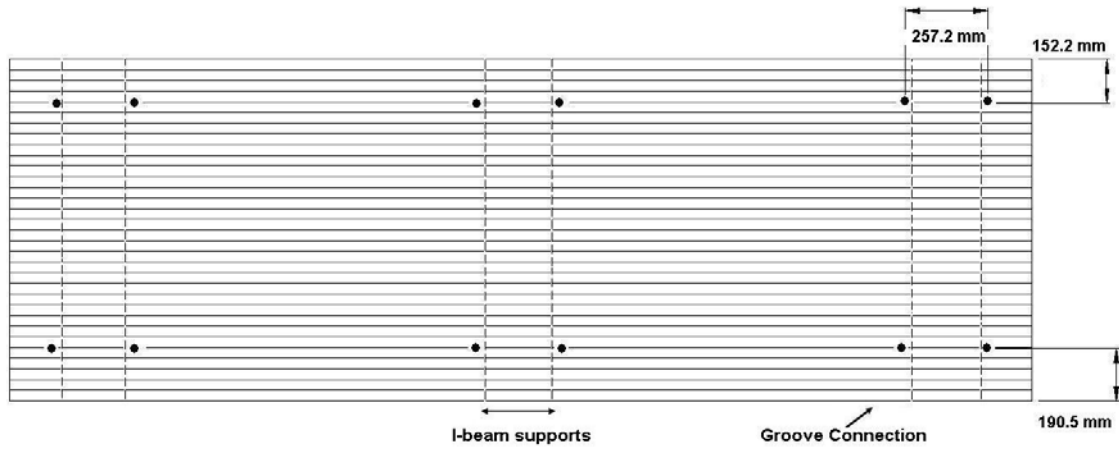


Glulam panel manufacture processes use primarily dimensional lumber, with typical moisture contents ranging from 12-14 percent at the time of manufacture. Moisture content can increase the dimensions in glulam panels due to low temperature and high humidity conditions occurring at the jobsite, which can be critical in glulam timber connections (APA, 1998). Therefore, it was assumed that the jobsite conditions could produce a high EMC of 25 percent in the panels. The American Plywood Association (APA) (1998) concluded that grain orientations of laminations within a glulam panel will be random for a given lay-up and that the net change in dimension of a typical glulam member results from the composite behavior of the shrinkage and swelling effect in radial and tangential directions of the laminations, or a net effect in the respective laminations making up the member. However, the dimensional change of the laminations is the largest in tangential direction for a typical glulam member (APA, 1998). From the Wood Handbook (1999), the dimensional change coefficient for Southern Pine in the tangential direction is approximately 0.0003. Based on this factor, a gap of a 1.59 mm (0.0625 inches) was calculated between the tongue and groove connection for each of the two panels thicknesses based on the tangential shrinkage and swelling effects occurring in the laminations between 12 and 25 percent moisture content.

Standard 5/8" galvanized dome head bolt and C-clip connectors (Figure 2) commonly used for timber deck to steel girder bridges were selected for use in this experimental evaluation. The attachment design only uses bolts along the grooved panel edge. No bolts are used on the tongue side of the panel, thus this edge of the panel is supported only by the interlocking joint. This attachment design allows the deck system to be installed complete from the top down, with no need to access the underside of the bridge deck to tighten bolts. Minimum edge and end distances of 6 inches (8d) were selected for this evaluation.



Figure 2. Deck-to-Girder Connection



Bolt Placement Plan

Experimental Program

The overall goal of this testing program was to evaluate the adequacy of the interlocking tongue and groove joint and the limited number of bolts/deck clips when subjected to AASHTO HS-25-44 live loading. The experimental program included three different sets of tests: Modulus of Elasticity (MOE) testing, full scale static testing and full scale fatigue testing. The overall test matrix is shown in Table 1.

Table 1. Experimental Test Matrix

Type of Test	Number of Specimens	Dimensions	Bridge Configuration	Design requirement
MOE Testing	12	(5) at 4 ft wide, 12 ft long, 5 1/8 in deep; (6) at 4 ft wide, 12 ft long, 6 3/4 in deep	N/A	N/A
Static Testing	8	(4) at 4 ft wide, 12 ft long, 5 1/8 in deep; (4) at 4 ft wide, 12 ft long, 6 3/4 in deep	(2 setups) 5 1/8" panels are transverse to 3 I-beam 5 ft O.C.; (2 setups) 6 3/4" panels are transverse to 3 I-beam 7 ft O.C.	HS-25 Loading
Fatigue Testing	4	(2) at 4 ft wide, 12 ft long, 5 1/8 in deep; (2) at 4 ft wide, 12 ft long, 6 3/4 in deep	(1 setups) 5 1/8" panels are transverse to 3 I-beam 5 ft O.C.; (1 setups) 6 3/4" panels are transverse to 3 I-beam 7 ft O.C.	HS-25 Loading

Modulus of Elasticity Testing

The modulus of elasticity of each panel was determined experimentally and the data was used 1) to compare the experimental values with published NDS values for the specific glulam combination type 49, and 2) be used as material input parameter for a finite element model. The MOE test set-up and procedure followed ASTM D198 (ASTM 2004) with one modification: a 3-point bend set-up was used instead of a 4 point bend.

Test panels were placed on (2) two W10x15 support beams 152.4 mm (6 ft) long with pin and roller support placed on top of the I-beams. The supports were placed 254 mm 10 ft apart for the 130.2 mm (5 1/8") thick panels and placed 355.6 mm (12 ft) apart for the 171.5 mm (6 3/4") thick panels. The loading frame consisted of two 35 mm (1 3/8") diameter DYWDAG threaded steel rods mounted vertically to a spreader beam, consisting of 152.4 mm (6ft) long C10x30 double channel steel beam. The testing frame

was then mounted to the laboratory reaction floor. A 222.5 kN (50 kip) load cell was used to record the load applied to each panel. A W10x49 I-beam and neoprene pad were placed underneath the load cell to distribute load across the panel width. A 102 mm by 102 mm (4" by 4") steel plate and a 178 kN (20-ton) hand pump hydraulic jack were placed on top of the W10x49 I-beam and underneath the load cell to apply the load to the panels. Four linearly-variable displacement transformers (LVDT's) were used to measure deflections of the panel at selected points. Two LVDT's were installed on both sides of the panel at midspan and two were mounted at opposite ends of each support. All of the data from the load cell and the LVDT's were recorded using a National Instrument data acquisition system. Before testing, each specimen's dimensions and moisture content was recorded at four different locations across the panels and then averaged. The experimental setup is shown in Figures 3 and 4.



Figure 3. MOE Test Schematic



The maximum load applied to the panels was 10% percent of the allowable bending stress or approximately 4.45 kN to 8.90 kN (1000 lbs to 2000 lbs). The panels were loaded in load control mode using a hand pump hydraulic jack. The maximum deflection, Δ , was computed by averaging the midspan deflections and subtracting the average support settlement. The modulus of elasticity was then calculated using linear elastic beam equations. The modulus of elasticity was multiplied by the wet-use factor (C_m) of 0.833, if the panels had average moisture content of 16 percent or greater during testing.

The results are summarized in Tables 2 and 3 below. The average MOE for the 130.2 mm (5 1/8") thick panels was 10.9 GPa (1581 ksi), which is within 8% of the published NDS value of 11.7 GPa (1700 ksi). Similarly, the average modulus for the 171.5 mm (6 3/4") panels was 11.2 GPa (1697 ksi) which is within 10% of the published NDS value.

Table 2. Summary of MOE results of the 130 mm (5-1/8") Panels

Summary of Modulus of Elasticity for 130.2 mm Glulam Panels					
Specimen #	Avg MC (%)	Max Load (kN)	Total Defl. (mm)	E' (GPa)	% Diff.(%)
130.2 mm-1					
130.2 mm-2	18.6	11.3	3.05	10.1	3.60
130.2 mm-3	13.9	12.4	2.97	10.2	13.1
130.2 mm-4	14.3	12.3	2.87	11.1	5.26
130.2 mm-5	11.8	12.7	2.67	12.5	7.00
130.2 mm-6	14.3	12.4	3.73	10.7	9.08
Average	14.6	12.2	3.06	10.9	7.61
SDV	2.48	0.538	0.403	0.983	3.68
COV (%)	17.0	4.40	13.2	8.99	48.4

Table 3. Summary of MOE Results for 171.5 (6-3/4") Panels

Summary of Modulus of Elasticity for 171.5 mm Glulam Panels					
Specimen #	Avg MC (%)	Max Load (kN)	Total Defl. (mm)	E' (GPa)	% Diff. (%)
171.5 mm-1	14.1	12.4	3.72	11.8	0.54
171.5 mm-2	14.1	12.9	3.72	11.0	5.76
171.5 mm-3	14.3	12.5	4.09	9.5	19.0
171.5 mm-4	16.6	12.9	3.68	11.1	13.9
171.5 mm-5	15.2	12.8	3.34	12.9	10.1
171.5 mm-6	14.3	12.4	3.73	10.7	9.08
Average	14.8	12.7	3.7	11.2	9.73
SDV	0.987	0.237	0.241	1.14	6.39
COV (%)	6.69	1.87	6.49	10.2	65.7

Static Testing

Four mock bridge decks (8 panels total; 4 at 130.2 mm (5 1/8") thickness, 4 at 171.5 mm (6 3/4") thickness) were tested statically first in 22 kN (5 kip) load increments and then to failure. The bridge deck panels were placed transversely over three (3) W12 x 53 I-beams. For each set-up two adjacent panels with a tongue and groove connection were bolted down to the I-beams with standard deck C-clip and 15.9 mm (5/8") diameter domed head bolts. Deflections and bolt forces were recorded at each 22 kN (5 kip) load step. Static tests to failure were also conducted for each bridge configuration to determine the ultimate strength and failure modes. Each bridge was tested under two load cases, one loaded on the tongue side and the other on the groove side. To simulate the tire loading of the HS25 design truck, a 508x254 mm (20 x 10 in) rectangular patch load was applied which is roughly equivalent to the contact area of one design truck wheel.

Two bridge configurations were setup for the full-scale static testing (illustrated in Figure 5). The first bridge configuration consisted of two 130.2 mm (5 1/8") panels supported by longitudinal I-beams spaced at 1524 mm (5ft) on center. The deck panels were installed transversely (perpendicular to the direction of traffic) over the I-beams. Each panel was 3657.6 mm (12) long, 1219.2 mm (4 ft) wide with a 177.8 mm (7") overhang on each side of the bridge.

The second bridge configuration was comprised of two 171.5 mm (6 3/4") thick glulam panels with a tongue and groove connection installed transversely to the three I-beam supports. For this configuration the I-beams were spaced 2133.6 mm (7 ft) on center. Each panel was 4876.8 mm (16ft) long, 1219.2 mm (4 ft) wide and had a 177.8 mm (7") overhang on each side of the bridge.

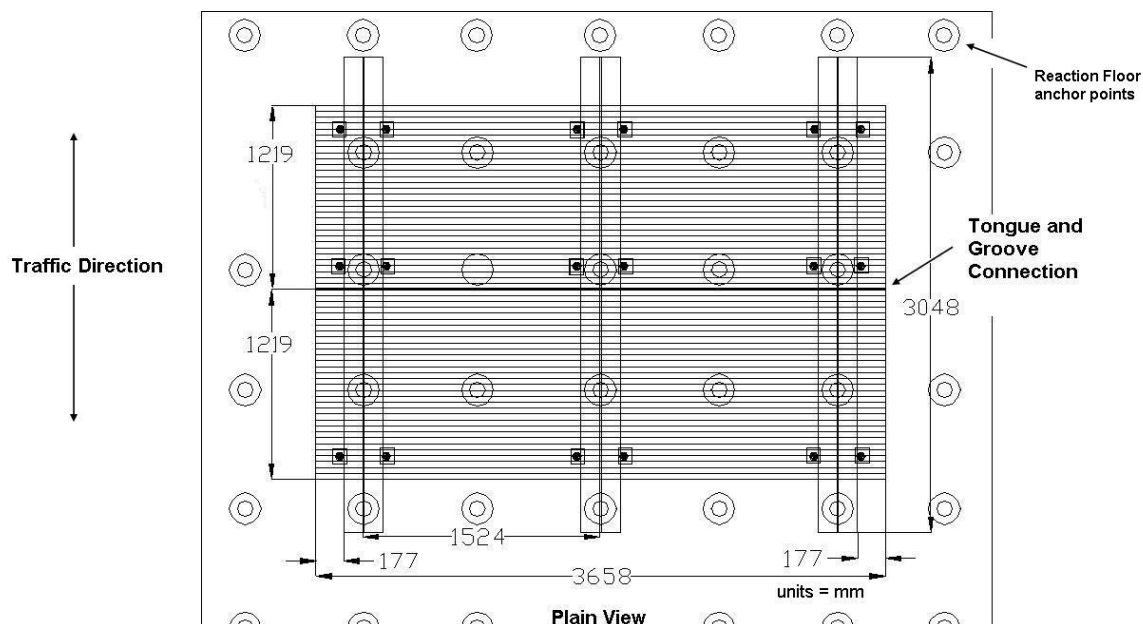


Figure 5a. 130.2mm (5-1/8") Thick Panel Bridge Configuration

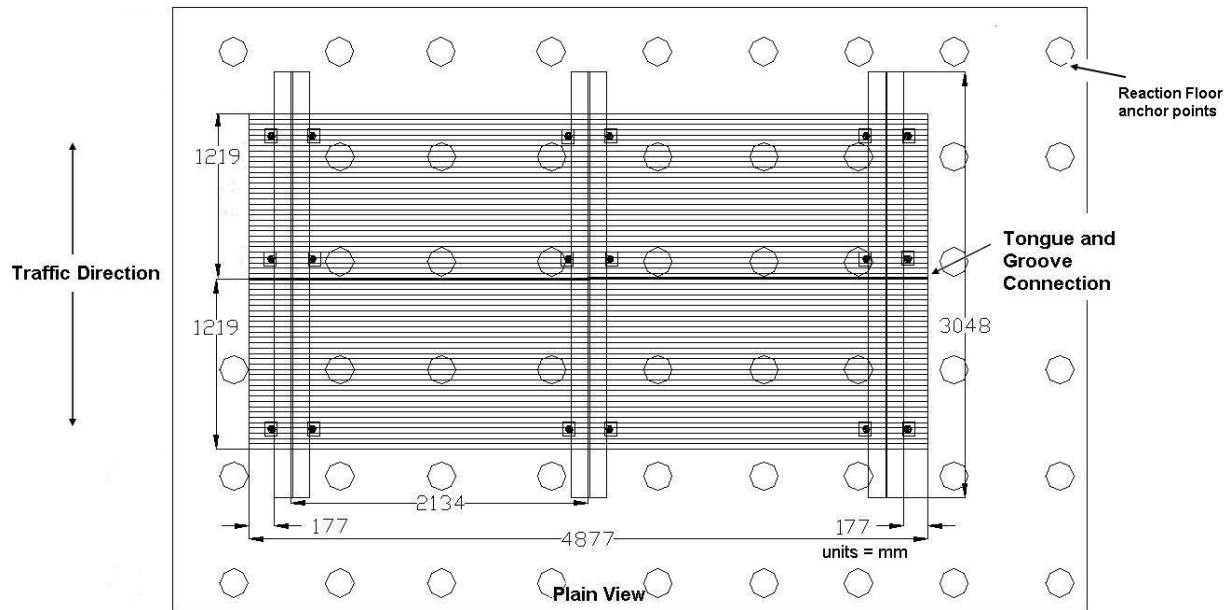


Figure 6b. 171.5 (5-1/8") Thick Panel Bridge Configuration

Instrumentation

Compression washers (Figure 7) and LVDT's were used to monitor the bolt forces and deck deflections respectively. The load was applied using a 490 kN (110 kips) Lebow load cell and an Instron 490 (110 kips) servo-hydraulic actuator mounted below the reaction floor. All of the data were recorded using a National Instruments data acquisition system.

A total of twenty two linear variable differential transformers (LVDTs) were used to measure deflections of the deck at different locations (see Figure 6). All of the LVDT's were mounted above the deck from three wooden support braces. Twelve LVDT's were positioned along the panel's length, 76.2 mm (3") from either side of the tongue and groove joint to monitor relative displacement at the joint. Five LVDT's were placed along the groove panel's length at midspan and another five LVDT's were positioned along the tongue panel's length at midspan.

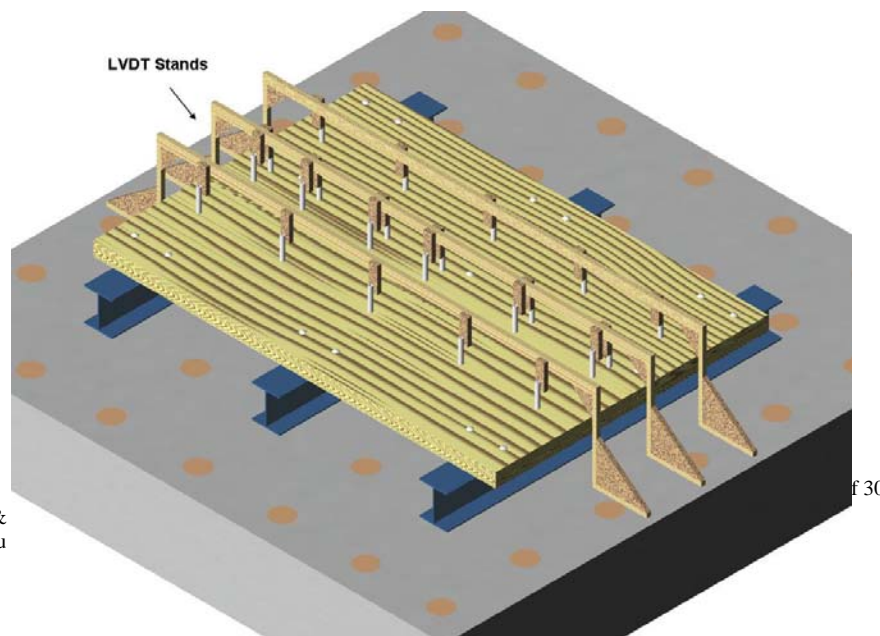


Figure 7. LVDT Placement Plan



Figure 8. Compression Washers Used to Monitor Bolt Forces

Results and Discussion

The deflection results are summarized in Table 4. Deflection directly under the tire patch at the design load for the 170.5 mm (6-3/4”) panels when loaded on both the groove side of the connection (load case 1) and when loaded on the tongue side (load case 2) are within range of the deflection limits recommended by the Timber bridge manual. Deflection limits recommended by AASHTO were exceeded however these limits are quite strict for temporary bridge applications and could probably be relaxed for such applications. Deflections of the 130.5mm (5-1/8”) panels exceeded the Timber Bridge Manual limitations under both load cases. Deflections were only slightly over the limit when loaded on the groove side (+2%) but deflection limits were exceeded by nearly 37% when loaded on the tongue side (load case 2). These excessive deflections may have been due to fabrication imperfections namely excess gap or gap inconsistency between the tongue and groove. However, this cannot be said conclusively given the limited number of tests and specimens. A slight reduction in deck span for the 130.5mm (5-1/8”) panels is recommend for initial uses to ensure that acceptable deflection limits are maintained. This span reduction may not be necessary once more in-service deflection data becomes available.

Table 4. Deflections Under Tire Patch at Design Load

Specimen Number	Load Case	Deflection	Timber Bridge Manual Limit (mm)	AASHTO Limit (mm)
130.2 mm-1-4	1	6.50	6.35	3.05
130.2 mm-2-6	2	8.71	6.35	3.05
171.5mm-1-6	1	7.09	8.89	4.27
171.5mm-4-3	2	7.39	8.89	4.27

Note Load Case 1 = Groove side loading ; Load Case 2 = Tongue side loading

Compression washers were placed on bolts that were expected to see uplift forces. The reasoning for this was to locate and quantify the maximum uplift forces and verify that the selected bolts were adequately designed to resist these tensile forces. At full design load a maximum bolt force of 2058 N (462 lbs) was recorded. Hale (1978) found this bolt/c-clip connector to have an ultimate tensile strength of 18.3 kN (4.1 kips) for a single bolt. Comparing these values the bolt/clip assembly was only stressed to about 11% percent of the ultimate tensile force reported by Hale (1978). Based on this result it appears that uplift bolt forces due to truck live loading can be neglected when this connection configuration is used.

The differential deflection between adjacent panels is an important feature of the panel-to-panel joint, since excessive differential displacement can cause detrimental cracking in the wearing surface. For this interconnected bridge decking system the differential displacement between adjacent panels may control the design. The maximum differential movement at the design load of 89.0 kN (20 kips) right under the load patch was 1.37 mm (0.054") for 130.2 mm (5 1/8") panels for load case one (groove side loading) and 3.22 mm (0.127") for load case two (tongue side loading). The maximum differential displacement at the design load for 171.5 mm (6 3/4") panels for load case one was 1.45 mm (0.057") and 1.29 mm (0.051") for load case two. A study of asphalt wearing surface for non-interconnected glulam deck bridges from Virginia Polytechnic Institute and State University called "Investigation of the Structural Behavior of Asphalt/Wood Deck Systems for Girder Bridges" (Howard 1997) recommends a maximum interpanel differential displacement of 1.27 mm (0.05"). In general the results are below or slightly over this limit with the exception the 130.2 mm (5 1/8") panel configuration loaded on the tongue side (load case two). Again this is suspected to be due to a fabrication error but the results are not conclusive give the small sample size. These results are summarized in Tables 5a thru 5d and plots illustrating how the differential displacement varies along the length of the panels are provided in Figures 9a thru 9d.

Table 5a. Summary of Results for Panel No. 130.2mm-1-4 ; Load Case 1 (Groove Side Loading)

Applied Load (kN)	Max. Uplift (Bolt Force) (N)	Panel Deflection under Load (mm)	Max. T and G Average Differential Deflection (mm)
22.3	560	2.44	0.395
44.5	404	3.96	0.739
66.8	1011	5.23	1.03
89.0	1338	6.50	1.37

Table 5b. Summary of Results for Panel No. 130.2mm-2-6 ; Load Case 2 (Tongue Side Loading)

Applied Load (kN)	Max. Uplift (Bolt Force) (N)	Panel Deflection under Load (mm)	Max. T and G Average Differential Deflection (mm)
22.3	837	3.78	1.72
44.5	897	5.83	2.36
66.8	779	7.11	2.81
89.0	1146	8.71	3.22

Table 5c. Summary of Results for Panel No. 171.5mm-1-6 ; Load Case 1 (Groove Side Loading)

Applied Load (kN)	Max. Uplift (Bolt Force) (N)	Panel Deflection under Load (mm)	Max. T and G Average Differential Deflection (mm)
22.3	407	2.64	0.503
44.5	577	4.13	0.577
66.8	1024	5.87	1.49
89.0	1282	7.09	1.45

Table 5d. Summary of Results for Panel No. 171.5mm-3-4 ; Load Case 2 (Tongue Side Loading)

Applied Load (kN)	Max. Uplift (Bolt Force) (N)	Panel Deflection under Load (mm)	Max. T and G Average Differential Deflection (mm)
22.3	21	3.18	0.812
44.5	504	4.75	1.17
66.8	1508	6.09	1.21
89.0	2058	7.39	1.29

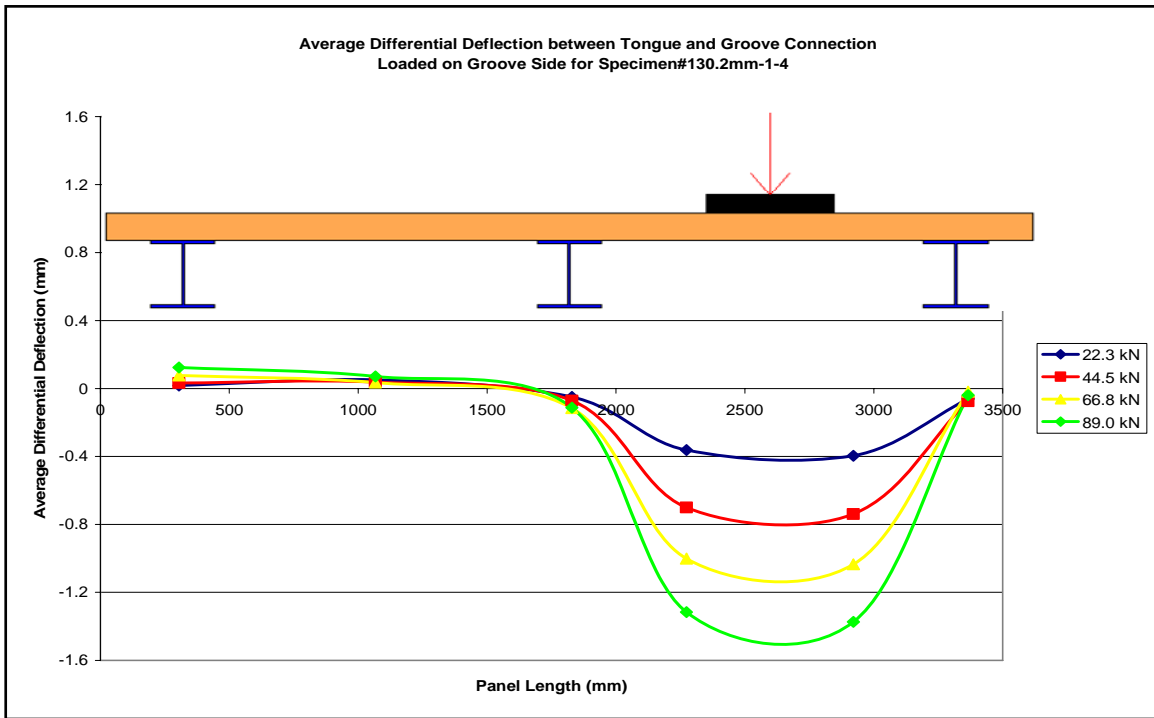


Figure 9a. Average Differential Displacement for Panel no. 130.2mm 1-4 Loaded on Groove Side

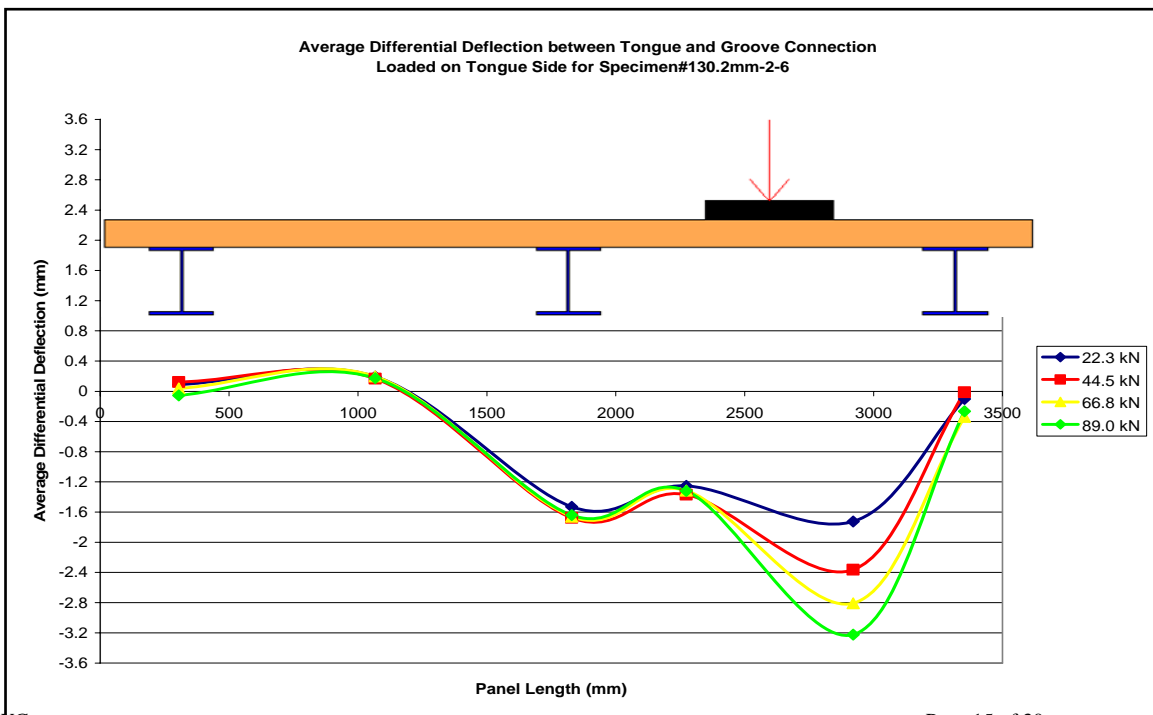


Figure 9b. Average Differential Displacement for Panel no. 130.2mm 2-6 Loaded on Tongue Side

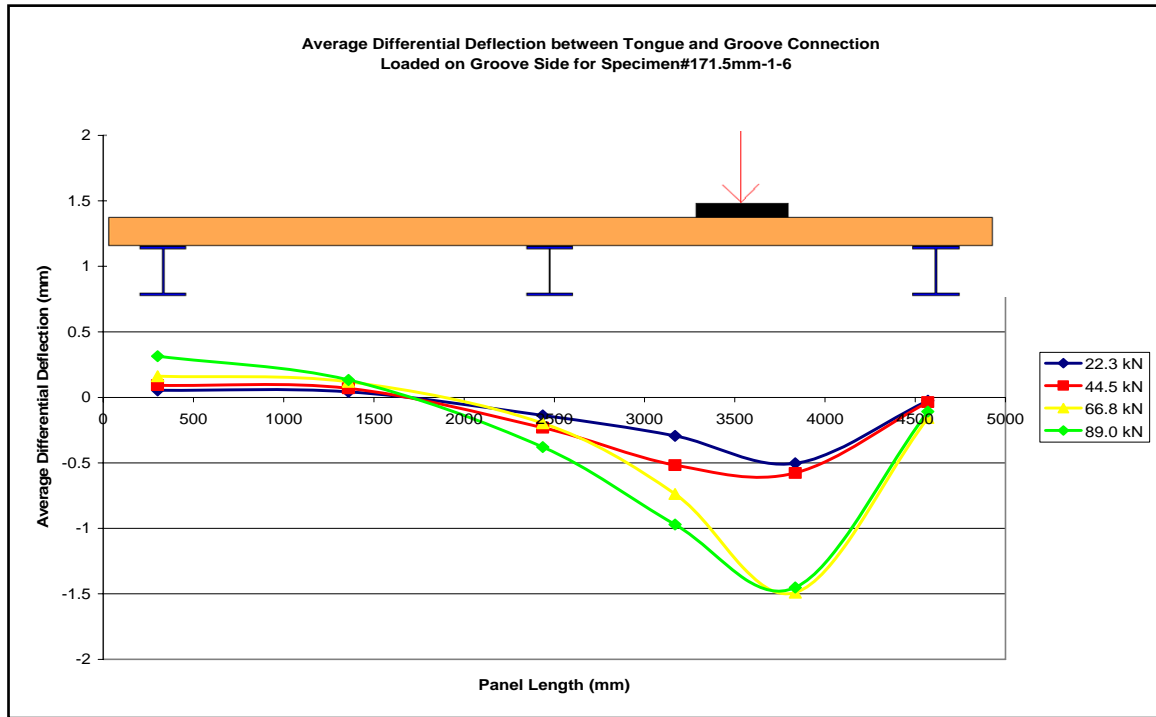


Figure 9c. Average Differential Displacement for Panel No. 171.5mm 1-6 Loaded on Groove Side

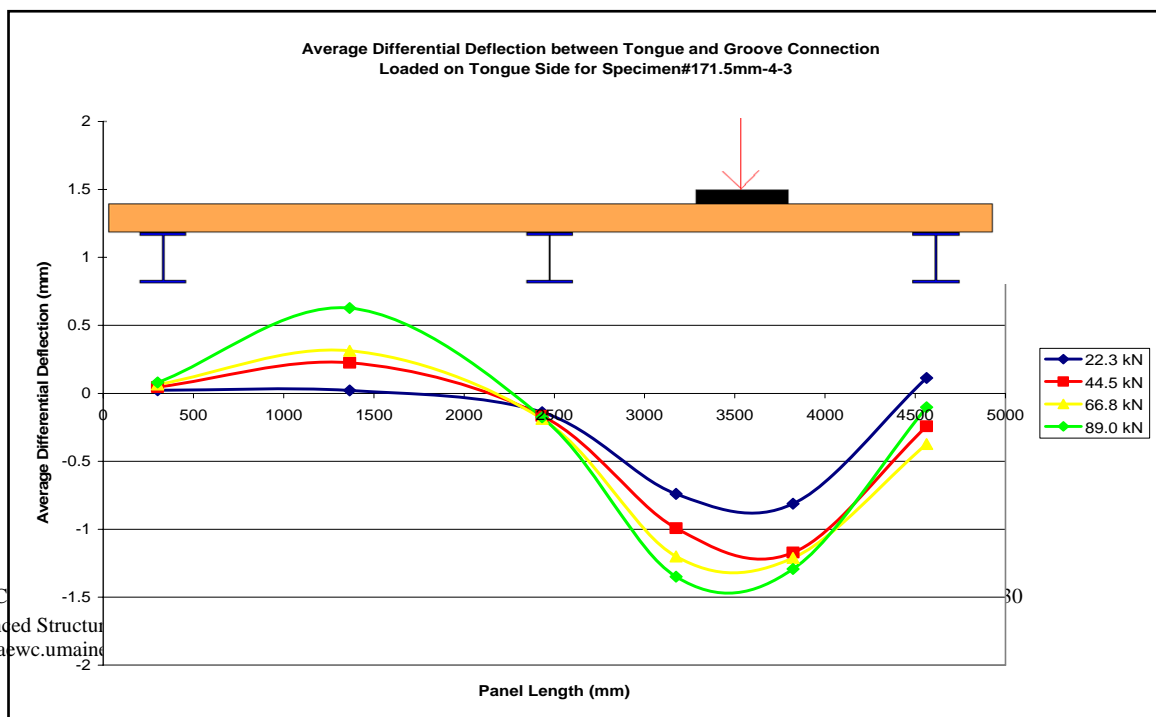


Figure 9d. Average Differential Displacement for Panel No. 171.5mm 4-3 Loaded on Tongue Side

Ultimate Load Test to Failure

130.2 mm Panels Load Case 1 (Groove Side Loading)

There was a linear load versus deflection relationship until the wood laminations cracked at 310 kN (69.7 kips), as shown in Figure 10. The deck continued to hold a substantial amount of load after the initial failure, until ultimately failing locally underneath the load patch. The ultimate load capacity of the deck was 394 kN (88.5 kips) with a maximum deflection of 27.5 mm (1.08"). The progression of failures shown in the load-displacement response created a ductile response, which could warn people of an impending failure, allowing them time to get to safety. For this deck configuration and loading condition, the factor of safety was calculated to be 4.43 times AASHTO HS25-44 truck loading.

The initial failure was tension perpendicular to grain in the tongue panel at the top of the tongue. The grooved panel then began to carry most of the load and eventually failed in localized shear approximately the depth of the panel (d) or 130.2 mm (5 1/8") away from the edge of the loading plate. Finally the bottom edge of the groove "unzipped" - failing in a combination of shear and tension. The failure progression is illustrated in Figure 11.

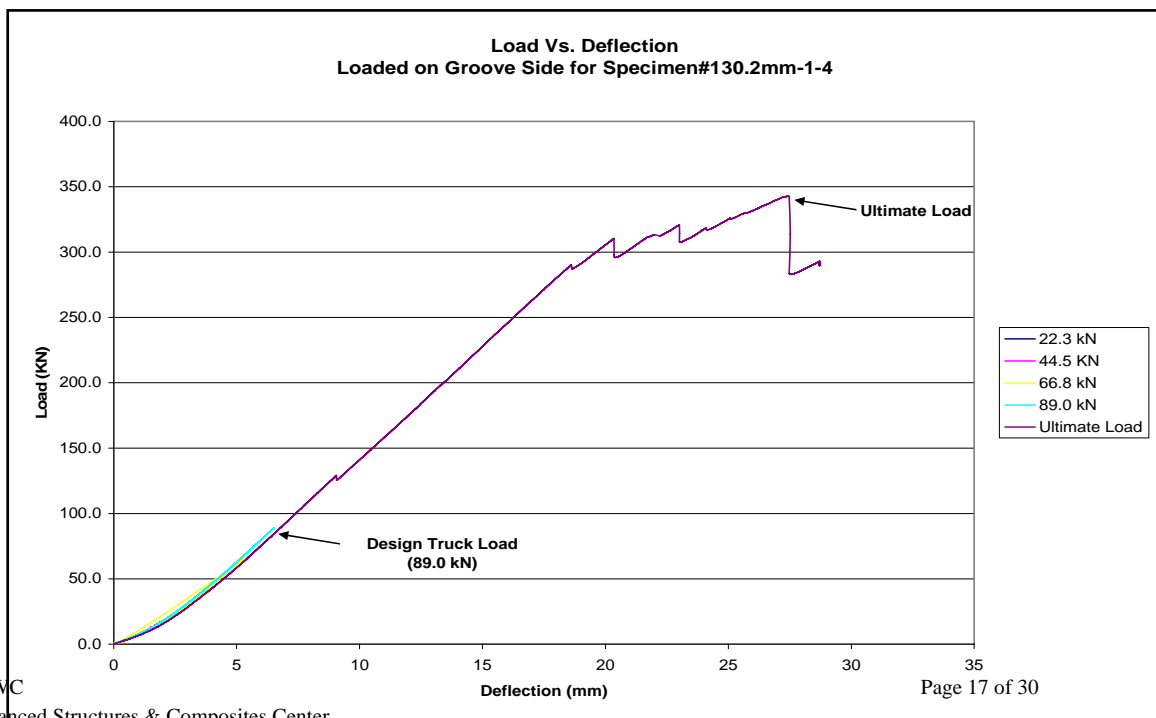


Figure 10. Load-Deflection Results (130.2mm Groove Side Loading)



Tension
perp. failure
at interface

Figure 11a. Intial Failure - Tension Perp at Tongue/Panel Interface



Edge of
Loading
Plate

Horizontal
shear cracks

Figure 11b. Secondary Failure – Localized Shear Failure Near Load Patch



Figure 11c. Ultimate Failure – “Unzipping” of Lower Side of Groove

130.2 mm Panels Load Case 2 (Tongue Side Loading)

Behavior of the deck system when loaded on tongue side was quite similar to the results described previously for groove side loading. For this load case the ultimate load reached was 330 kN (74.2 kips) with a maximum deflection of 27.0 mm (1.06”). The factor of safety for this bridge arrangement and load case was 3.71 times the design load for HS25 truck. The initial failure occurred in the tongue as a shear failure at a finger joint which propagated down through the thickness of the panel. 4. The secondary failure occurred as the same type of unzipping of the bottom edge of the groove similar to Figure 11c.

171.5 mm Panels Load Case 1 (Groove Side Loading)

For the ultimate load test of 171.5 mm (6 ¾”) panels, loaded on the groove side there was a linear relationship between the load and deflection in the deck until the laminations cracked at 318 kN (71.5 kips). The load continued to increase until fracture occurred in the deck at an ultimate load of 376 kN (84.5 kips) with a maximum deflection of 32.3 mm (1.27”) (see Figure 12).

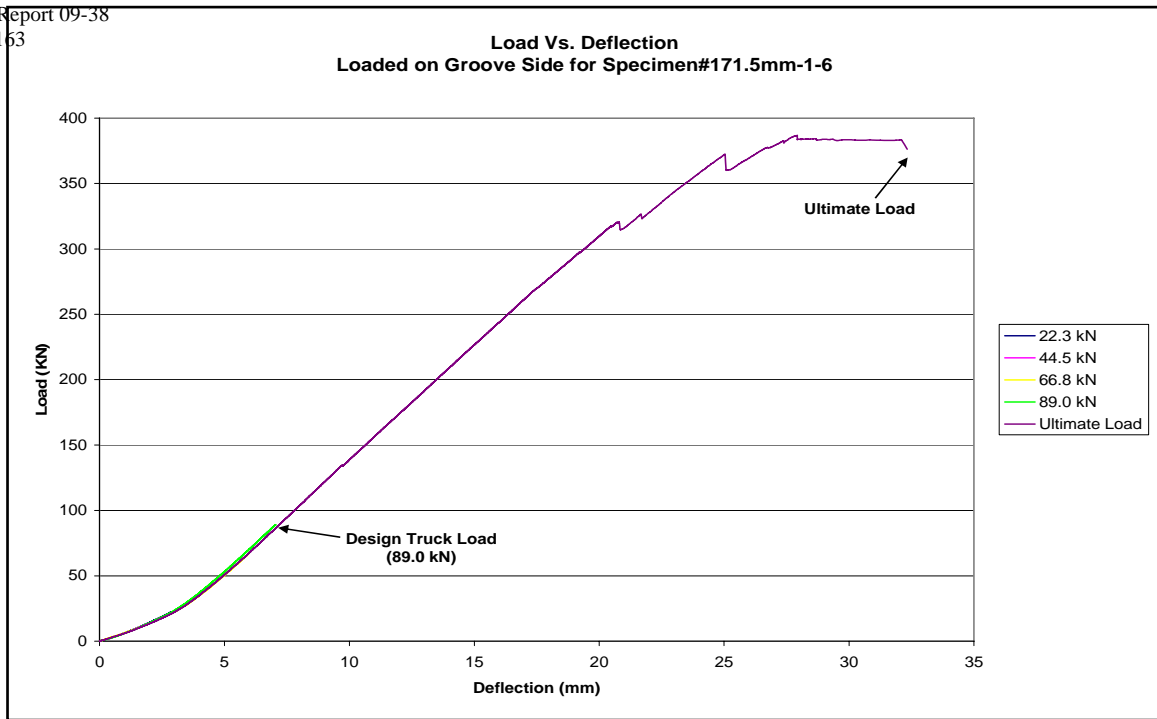


Figure 12. Load Deflection Results for 171.5 mm Loaded on Groove Side

For this load case the factor of safety was 4.22 times the design loading. The failure initially occurred at the tongue and groove interface. First, the groove was pressing on the top of the tongue resulting in a combination of tension perpendicular to grain and horizontal shear failure on the top side of the tongue along the entire length of the loaded span. Then, the top face of the groove failed at a finger joint in shear and the shear failure progressed along the length of loaded span. Finally, the loaded panel (groove panel) continued to fail in shear about eight laminations from the panel-to-panel joint (see Figure 13), just to the side of the loading plate. The same type of failure occurred on the other panel (tongue panel) about 6 laminations from the joint.



Failed in shear,
6 laminations
from the joint

Figure 13a. Shear Out Failure of Tongue Panel



Horizontal shear
failure along the
length of loaded
span

Figure 13b. Shear Out Failure of Groove Panel

171.5 mm Panels Load Case 2 (Tongue Side Loading)

For this bridge setup, the ultimate load was the lowest of all of four tests. However, the factor of safety for this test was 3.32 times the design loading of 89 kN. Examining the failure mode for this ultimate load test revealed that the tongue for the loaded panels was pressing on the bottom side of the groove. This resulted in a tension perpendicular to grain and a shear failure of the bottom side of the groove along the entire length of the loaded span. There was also a slight tension perpendicular to grain failure noted on bottom side of the tongue near the load. The panel continued to fail in shear, propagating in towards the center of the groove panel. The groove panel sheared along its length about 152.4 mm (6") or 6 laminations in from the edge of the panel-to-panel joint. Lastly, the tongue panel also sheared along its length about 6 laminations from the edge of the joint in much the same manner as the panels described in the previous section.

Fatigue Testing

Two full-scale mock bridge decks (one 130.2mm and one 171.5mm thick) were tested to evaluate the fatigue durability over a life span of 50 years. The static test results indicated that loading the tongue side of the joint caused more damage to the panels than loading the groove side. Given this result tongue loading was selected for the fatigue testing. Each bridge deck was subjected to 2 million cycles of load over a 14 day period to simulate the wearing of a permanent bridge deck over its life expectancy of 50 years. Finally, an ultimate load test was conducted after the fatigue cycle to determine the residual strength of the glulam deck panels. The fatigue testing program was based on ASTM D6275-98 Standard Practice for Laboratory Testing of Bridge Decks (ASTM 1995).

For typical in-service bridge decks, the dead load is about 10 percent of the total design live load (ASTM D6275). The ASTM standard called for each specimen to be loaded with a sine-wave function from the dead load of the deck (approximately 8.90 kN) (2 kips) to HS25 truck design live load, plus the dead load (97.9 kN) (22 kips) for a total of 2 million cycles. During the fatigue testing, the cycling was stopped at increments of between 150,000 to 250,000 cycles and a static test was performed to track the stiffness degradation of the decks over the testing period.

The test results are summarized in Figures 14 thru 17. The results for both thicknesses show no sign of any significant stiffness loss or hysteretic damage accumulation, indicating that the durability of this bridge configuration will remain constant over a 50 year life span.

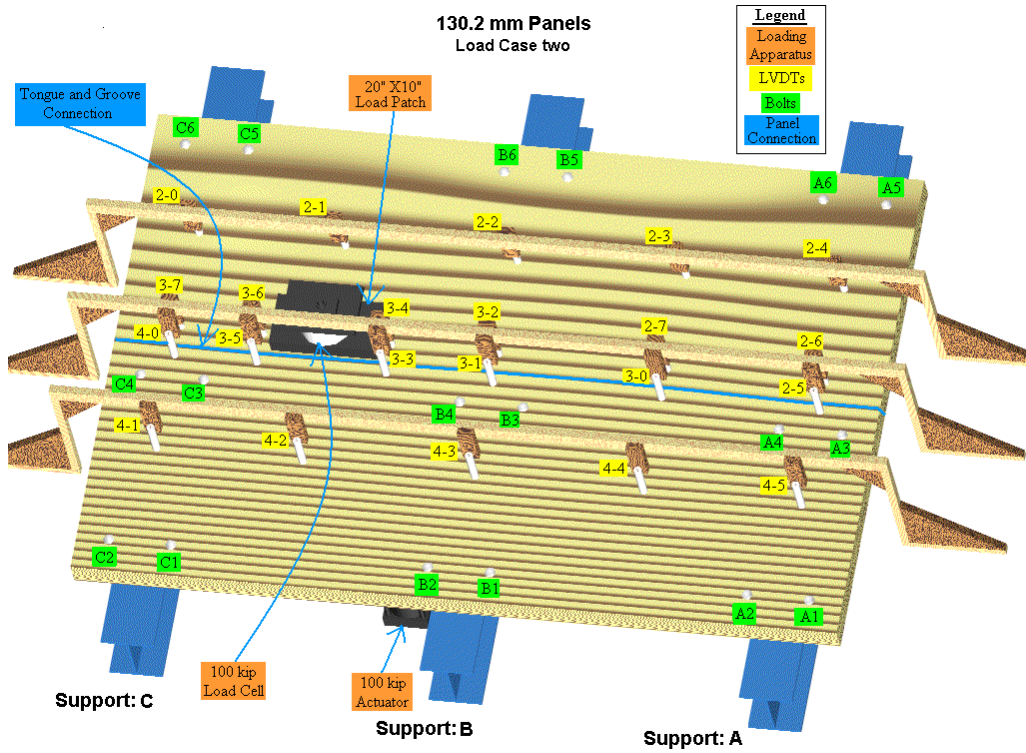


Figure 14. LVDT Numbering Scheme

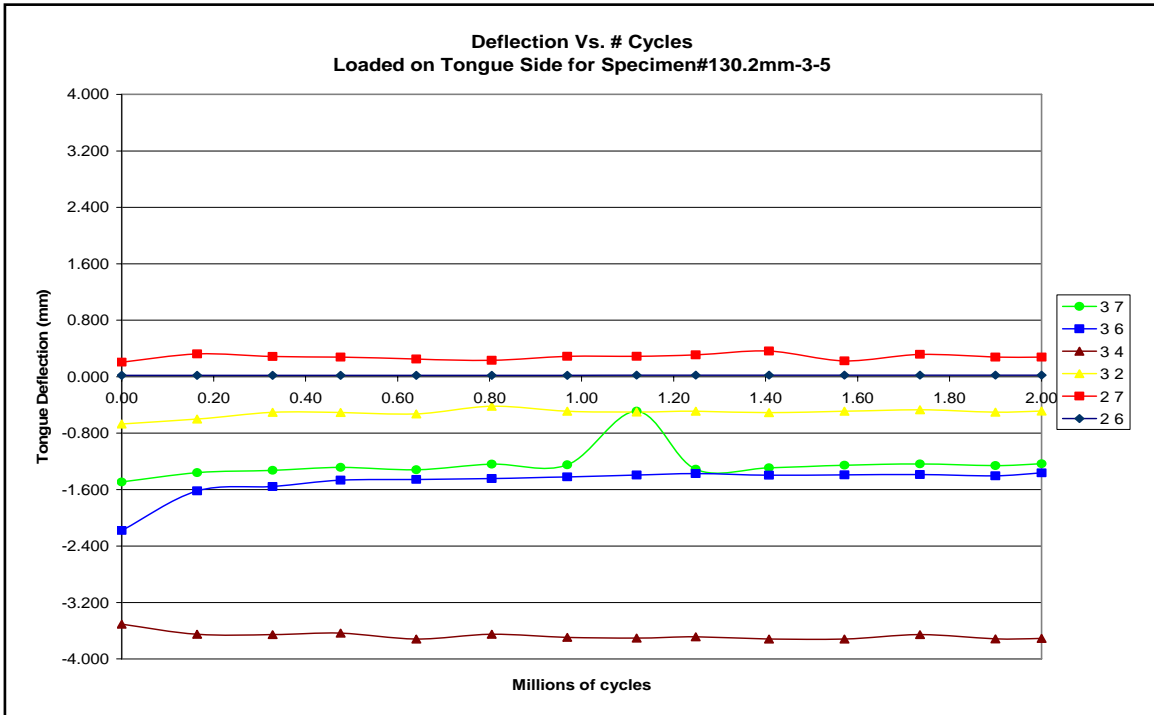


Figure 15. Deflections at Tongue Versus Number of Load Cycles (130.2mm Panels)

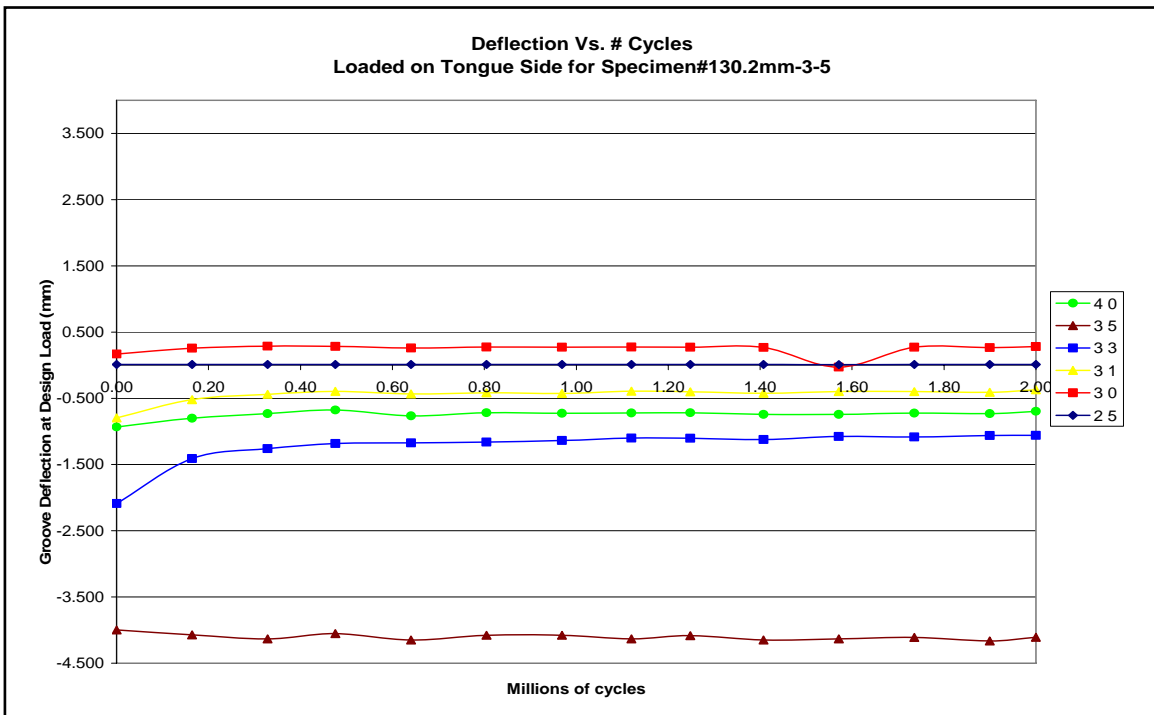


Figure 16. Deflections at Groove Versus Number of Load Cycles (130.2mm Panels)

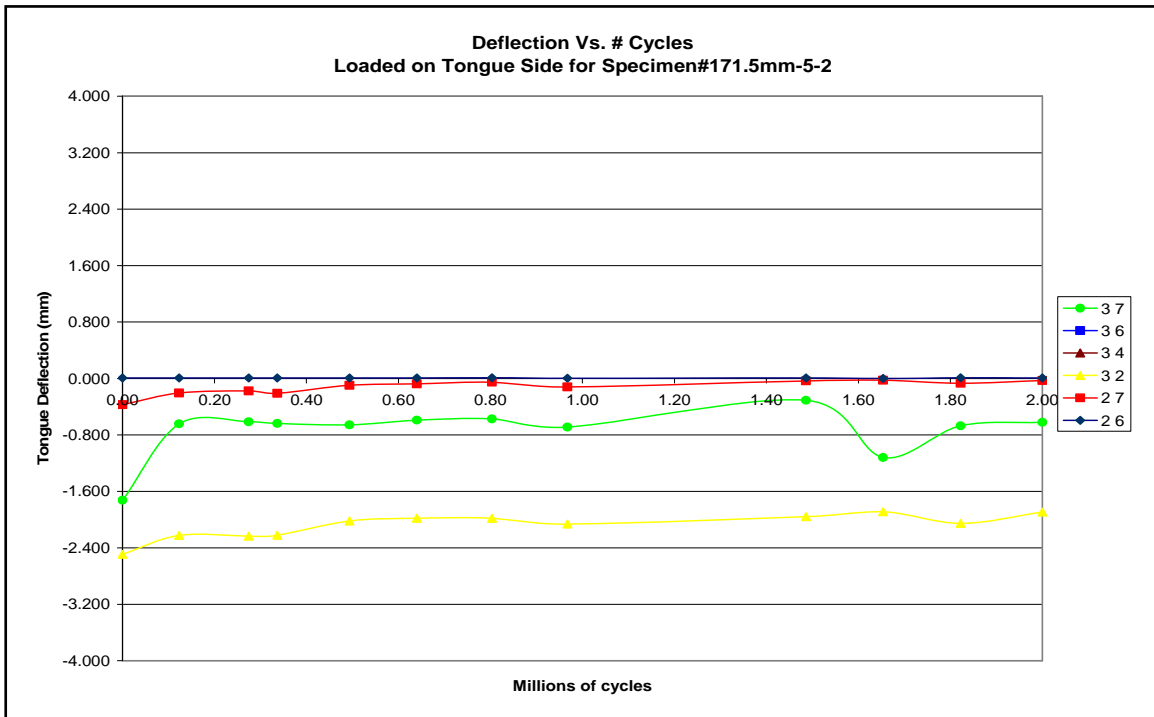


Figure 16. Deflections at Tongue Versus Number of Load Cycles (171.5mm Panels)

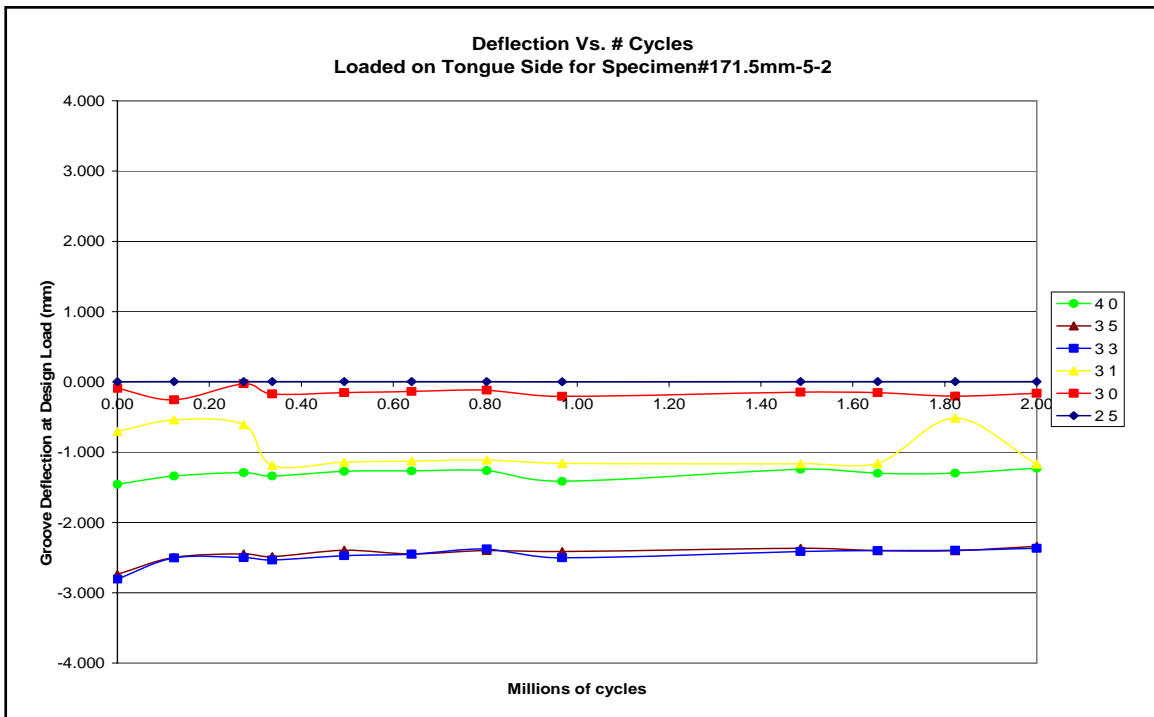


Figure 17. Deflections at Groove Versus Number of Load Cycles (171.5mm Panels)

After the full-scale fatigue tests were conducted, an ultimate load test was performed to determine the residual strength of each panel set.

Figure 18 compares the ultimate load for both the static test and the residual strength test conducted on the 130.2 mm (5 1/8") panels loaded on the tongue side (load condition two). As shown in the figure, the panels had a residual strength of 330 kN (74 kips) with a maximum deflection of 30.2 mm (1.19"). Comparing the residual strength to the ultimate static load shows that the panels failed at approximately the same load. Another important conclusion drawn from this test is that this bridge configuration has approximately the same stiffness after fatiguing it for 2.0 million cycles as it did before being exposed to fatigue cycling. From this test, it was determined that this bridge setup had a residual strength factor of safety of 3.71 times the design loading. The failure modes of the were nearly identical to those seen in the static tests with an initial failure occurring in the tongue panel and then a secondary failure of the groove panel.

Figure 19 shows the residual strength of 171.5 mm (6 3/4") panels loaded on the tongue side. Again, this figure shows the same tendencies observed in the ultimate static test. The residual strength of 286 kN (64 kips) and panel stiffness are practically identical to the ultimate static load test performed on the same bridge configuration and loading case. This bridge setup had a residual strength factor of safety of 3.21 times the design live load and exhibit failure modes similar to those observed in static testing.

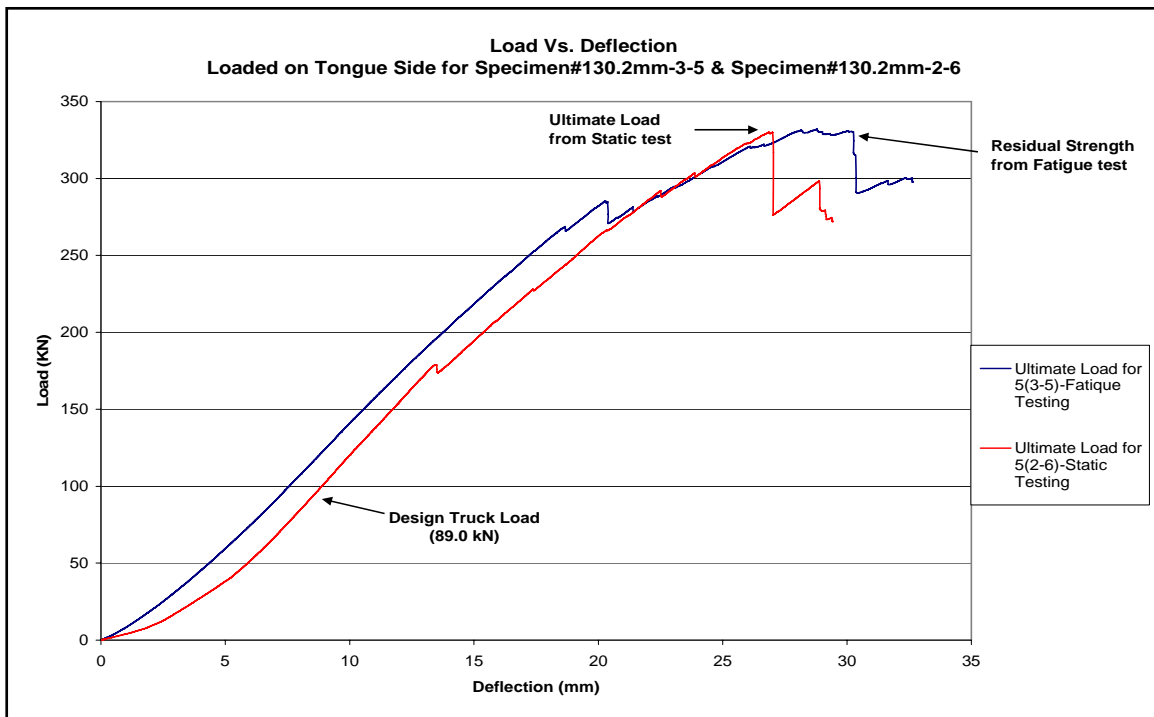


Figure 18. Comparison of Post Fatigue and Non-Fatigue Response for 130.2mm Panels

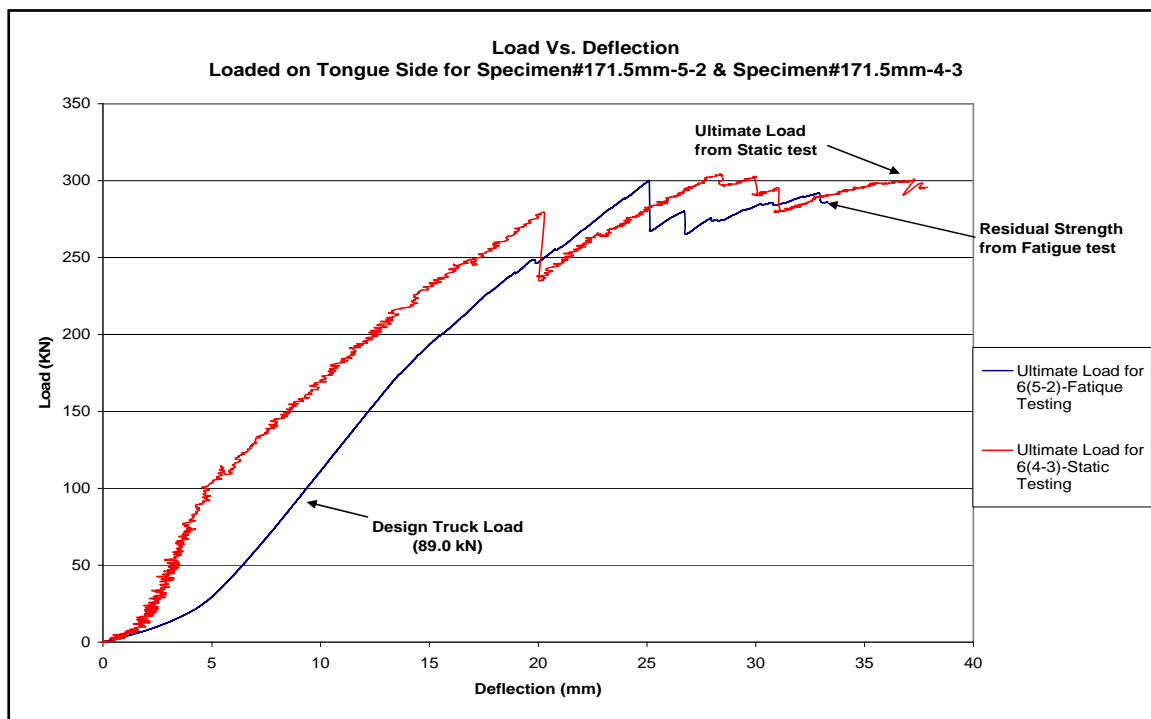


Figure 11 Comparison of Post Fatigue and Non-Fatigue Response for 130.2mm Panels

FEA Modeling

An FEA model was developed using ANSYS Parametric Design Language (APDL) that is available within the ANSYS software package. The FEA model allows the user to input parameters for the glulam deck-to-steel girder hybrid bridge, then the model solves and outputs critical results in a tabular format into text files. The FEA model consisted of solid elements, contact elements and spring elements to capture the overall behavior of the bridge. A higher-order twenty-noded quadratic solid element was used to model the glulam panels. A higher-order solid elements and multi-point constraint contact algorithm was used to model the I-beams. Twenty-node solid elements, surface-to-surface contact elements and a multi-point constraint contact algorithm were used to model the tongue and groove (panel-to-panel connection) interaction. Spring and surface-to-surface contact elements were used to model the panel-to-girder connection. The model also incorporated the loading apparatus and boundary conditions that were used in the static and fatigue experimental testing program. Finally, the FEA model used a non-linear solver.

Unfortunately, this model does not produce reliable results at all load levels and run time is too long for it to be used as a practical design tool. It was originally expected that a reasonably accurate model could be produced with limited complexity. This was found not to be the case. This model development effort showed that a high level of detail and complexity is required to accurately model the tongue and

groove interface and that the behavior at this interface has a very large impact on the overall response of the panels.

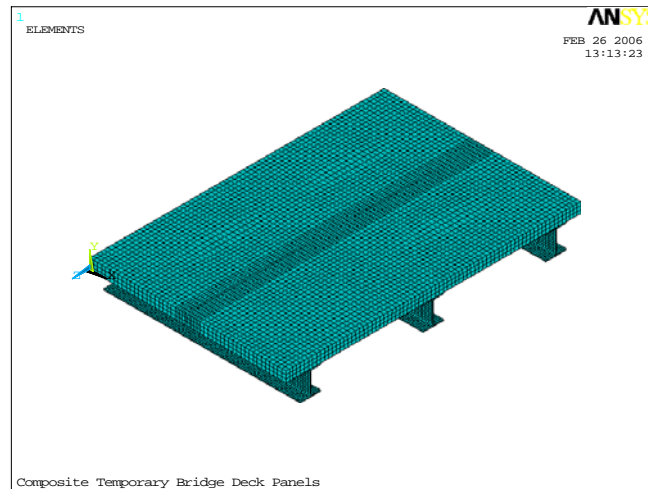


Figure 20. ANSYS FEA Model Element Mesh

The model was developed to a point where the results indicated that in order to accurately model the bridge deck response with commercial FEA code, sub modeling of the T&G joint would be necessary. This was considered beyond the scope of this project and since the original goal was to develop a simple, user friendly design model, development of this model was discontinued. If further modeling efforts are pursued, more simplistic empirical models or closed form analytical solutions may prove to be more suitable for this application.

Summary and Conclusions

A simplistic and innovative interlocking glulam bridge deck panel was developed for use in temporary bridge deck applications. The key feature of this panel system is the interlocking tongue and groove joint. The joint provides both displacement compatibility and structural load transfer between panels. The unique structural tongue-and-groove design successfully addresses two common issues with existing transverse glulam bridge decks:

1. It reduces differential movement between panels which is a very common problem with non-interconnected glulam deck systems in use today.
2. It allows the deck system to be installed completely from the top, eliminating the need for workers to install bolts from underneath the bridge deck.

Other design advantages of the panels include: prefabrication for rapid on-site installation, reusability, and because they are made from wood, the panels are relatively lightweight and have a relatively high strength-to-weight ratio when compared to steel and concrete alternatives.

The static test results show that:

- Differential panel deflection is within acceptable limits for HS-25 Loading
- Uplift force on bolts are minimal and can be neglected in design
- The Ultimate load range from 3.2 to 4.5 times the AASHTO HS-25 Loading. These Safety Factors are comparable to essential structure, such as hospitals, schools and high volume traffic bridges.
- Design sizing of the panels will likely be controlled by either differential deflection limits between panels or overall panel deflection limits.

The fatigue results show:

- No significant stiffness loss or hysteretic damage accumulation after 2 million design load cycles (50 yr life span)
- Residual strength and stiffness of fatigue panels is approximately the same as unfatigued strength and stiffness (no strength or stiffness degradation).
- Failure modes of fatigue panels are the same as those of unfatigued panels indicating that there are no fatigue sensitive design details.

Recommendations for future work are follows:

- Investigate the environmental durability of the decking system
- Conduct a demonstration project
- Investigate a wearing surface and a railing system for the decks
- Investigate the possibility for this decking system to be a permanent deck for bridges
- Develop a simplistic user friendly design tool which may be in the form of a model or simply design tables.
- Work with industry to develop a automated manufacturing process system for the tongue and groove.

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