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Research

Evaluation of Retrofit Procedures for Nail-Laminated and Stringer Bridges

Minnesota Local Road Research Board

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16. Abstract (Limit: 200 words)

Many of the 1,400 timber bridges in Minnesota need to be improved to meet present day standards. When the desired service level can be attained by widening a bridge six feet or less, a retrofit can be done by placing a second, wider, transverse deck onto the existing deck and substructure. Bridge components must be carefully inspected prior to a retrofit project.

Bridge #6641 in Sibley County was retrofitted. First the bituminous surface was removed. A longitudinal beam supported the extended deck. Grout was poured and leveled and then nail-laminated panels were laid transversely. A bituminous surface was laid over the full width of the new deck. The cost of the project was $51,632. (Replacing the bridge was estimated to take 2–3 years and cost $215,000.) The county quantified the strength change and load distribution characteristics of the retrofitted bridge deck.

Static and dynamic load tests were performed both before and after the retrofit. The tests show that adding a second deck effectively decreased the static deflections and improved the transverse load distribution.

Nail-laminated timber bridge #2642, also in Sibley County, was retrofitted in 1992 and was load tested again in 1995. All dynamic deflections were lower than those of the post-retrofit tests in 1992. This improvement can be explained in part by the drying of the moisture that was introduced into the bridge deck during grouting. Subsequent drying would add stability to the bridge deck.

A retrofitted timber bridge is expected to last an additional 20–40 years.

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EVALUATION of RETROFIT PROCEDURES for NAIL-LAMINATED and STRINGER BRIDGES

Final Report

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Gene Isaacson - Sibley County Engineer
Ken Johnson - Wheeler Consolidated, Inc.
Demetrios Beltaos - AEC Engineering, Inc.

Executive Summary

The primary purpose of this project was to evaluate the improvements in stiffness and load distribution for a retrofitted timber stringer bridge. An additional segment of the study evaluated the deflection and load bearing behavior of a nail-laminated bridge after 3 years of service.

The timber stringer bridge (L6641) located on County Road 8 in Sibley County was widened and strengthened because of the high traffic volume on this stretch of highway. Highways 169 and 19 intersect nearby. This section of County Road 8 is used as a short-cut for traffic to connect between these highways.

The stringer bridge was constructed by using three spans: two end spans of 5m - 18 cm (17 ft) and a center span of 7m - 62 cm (25 ft). The road deck is supported by timber stringers which run from the piling cap at the abutment to the piling caps at the midspan pilings. A timber deck was then built over the stringers. The grain direction of the timber deck is perpendicular to the traffic direction and the stringers. During retrofit, an additional longitudinal beam was added to the sides of the bridge and the retrofit deck was added over the top of the existing timber deck. The old bituminous layer was removed from the bridge and a layer of grout was poured and leveled before the retrofit deck was installed. This deck was assembled from prefabricated, nail-laminated panels laid perpendicular to the direction of traffic but parallel to the orientation of the existing deck. After the bituminous surface was applied, the bridge was then load tested. Along with load testing, the substructure of the bridge was inspected and areas of deterioration were treated with preservative.

Results showed that the addition of the second deck decreased static deflections. The maximum midspan deflections were reduced by 32 percent with one lane loaded and 25 percent with both lanes loaded. The reductions in the measured peak dynamic deflections ranged from 26 percent to 55 percent. For the center of the bridge, the traffic speed at which maximum deflections occurred increased from 32 kph to 64 kph. However, at the center of the west-bound lane, the traffic speed at which maximum deflections occurred dropped from 32 kph to 9.7 kph.

Results from the retrofitted nail-laminated bridge (#2642) after 3 years of service showed an unexpected reduction in deflection due to static loading, compared to testing conducted immediately after the retrofit (Leon, Beltaos, Seavey, 1993). The cause for the further reduction

in static bending is probably due to drying of the bridge deck after moisture content increase during the grouting between the original and retrofit deck. The peak dynamic deflections tested in 1995 closely match those of 1992 in showing reductions ranging from 9 percent to 31 percent as compared to the original bridge. Furthermore, the maximum dynamic deflections observed in 1995 occurred at speeds of 9.7 kph and 32 kph, which matches the post-retrofit dynamic test results observed in 1992. These tests indicate that the retrofit procedure used on this bridge continues to perform well in having strengthened the original nail-laminated structure and improved the load distribution.

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CHAPTER 1
INTRODUCTION

This document discusses research that involved the retrofit of a stringer timber bridge with a second transverse timber deck that widened the bridge. It also examines the long term performance of a longitudinal nail-laminated timber bridge that was retrofitted with a second transverse timber deck three years earlier.

It is imperative that fixed transportation infrastructure budgets be used to maximize serviceability, safety and economy. Retrofitting existing timber bridges with wider decks provides a means of accomplishing this goal.

Chapter 2 describes the inspection of the stringer timber bridge so that the existing structure can provide service life realization of the retrofitted transverse deck. Chapter 3 reports on the changes in static and dynamic behavior of the retrofitted stringer timber bridge before and after the retrofit. Chapter 4 examines the load distribution performance of a longitudinally nail-laminated timber bridge which was retrofitted with a transverse deck three years earlier.

CHAPTER 2
INSPECTION OF STRINGER TIMBER BRIDGE

An inspection was done in preparation for retrofitting Bridge No. L6641 with an additional transverse wood deck that increased the width of the bridge. Bridge components were evaluated to insure that the service life of the deck retrofit would be realized. The inspection focused on the pilings of the bridge components as they are most susceptible to bio/mechanical deterioration. This report includes the evaluation of all the pilings and other bridge components. In addition, the report includes a description of the remedial action that was taken to address areas of deterioration.

2.1. INSPECTION

The inspection of the bridge took place in June, by Robert Seavey and Maureen Puettmann. The bridge is located on Country Rd. 8 about 6 miles east of Hwy. 169 in Sibley County. The width of water flow under the bridge at the time of the inspection was about 4.6 m (15 feet). The Rush River flows north under the bridge which is oriented east-west. County Rd. 8 increases slightly in elevation east of the bridge and is level with the bridge to the west. A platform was constructed under the bridge to facilitate the inspection of the bridge and for the static/dynamic testing which was carried out at a later date.

Inspection involved checking for physical damage, moisture content problems, and biodeterioration. Physical damage was assessed visually; moisture contents were evaluated using an electrical moisture meter. Surface biodeteriation was investigated visually and by the use of a probe. The absence/presence of internal decay was determined by sounding with a hammer.

All pilings were inspected below the groundline, at the groundline and above the groundline. Piling caps and other timbers were examined for possible moisture collection and the presence of decay. The pilings were numbered prior to the inspection with a permanent marker. See Figure 2.1 for a plan view of the pilings and number identification layout used for the inspection.

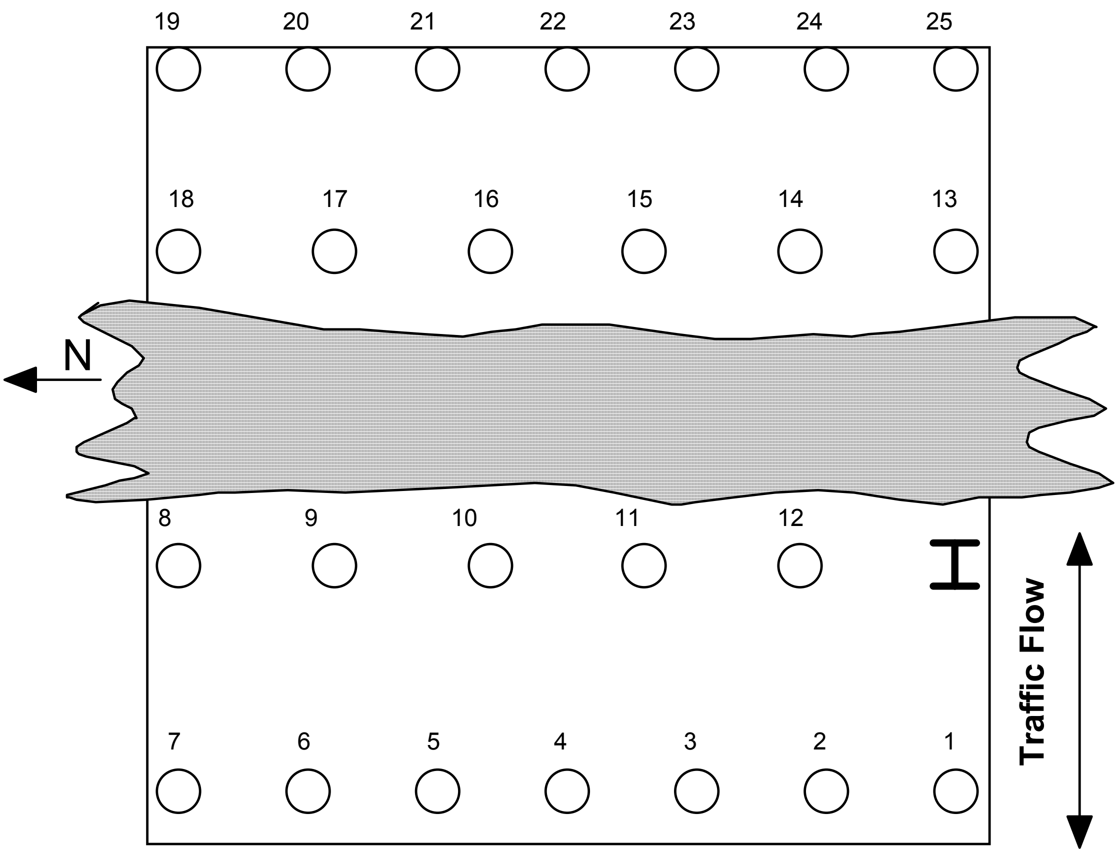

Figure 2.1. Plan View of Bridge Showing Piling Locations

Inspection of Bridge Pilings

P1-P12 are located on the western half of the bridge.

P1-P7 are abutment pilings, fully out of the water.

P7-P12 are the second row of pilings from the west end; some of these pilings were within the stream at the time of inspection.

P10 surface softening/decay 2.5 cm (1 in) inward from surface, beginning 15.2 cm (6 in) above ground; softening was determined using a flat-head screw driver or pick.; breaks that do not produce splinters are most likely decayed. Surface of piling crumbled off when touched or broke away easily.

P6 surface softening/decay 2.5 cm (1 in) inward from surface; begins at groundline and extends below groundline.

P7 slight surface degradation; more localized than P6 and P10.

P1, P2, P3, P4, P5, P8, P9, P11 and **P12** appeared to be sound and in good condition.

P13-P25 are located on the eastern half of bridge.

P13-P18 are the second row of pilings from the east end of the bridge, some of these piles had waterlines rather than groundlines at the time of this inspection.

P19-P25 are the abutment piles, fully out of the water.

P13 piling needs replacement, severe physical damage, plant growth developing.

P16-P18 water level too high to adequately inspect at the groundline and below.

P17 should be inspected when water level decreases. Some surface softening was noticed.

The southeast side bridge diagonal bracing needs replacing. This diagonal has a large crack, probably resulting from the physical damage that caused the break in the adjacent piling (P13). This diagonal is attached to the lower end of P13 and extends to the upper portion of P16.

Using resistant type moisture meters, moisture contents were taken on all pilings that showed signs of decay. All decayed pilings, indicated only the presence of surface decay based on the extent of the inspection; no internal decay was detected.

2.2. REMEDIAL PRESERVATIVE TREATMENT

In October 1995, Robert Seavey and Maureen Peuttmann went to Sibley County to treat the piles which showed signs of decay. The following pilings were treated with CuRap™ 20: P10, P13, P14, and P16. CuRap™ 20 is a patented formulation containing copper napthenate and boraz (sodium tetraborate decahydrate), which is commonly used for remedial treatment of standing utility poles, posts, pilings, railroad ties, or any wooden structure susceptible to attack by wood destroying organisms.

The preservative was applied by spatula up to 0.95 cm in thickness to the piling's exterior surface. Application below the ground line was limited to the water table or level at the time of treatment. The total length of pile treated ranged from 86 cm to 91 cm starting from below the groundline to above groundline. The treated portions of the pilings were wrapped using Pol-Nu Paper which acts as an impervious barrier to protect the preservative from leaching or being washed away and facilitates longterm diffusion of the chemical into the piling.

All pilings except for P16 also received structural bracing, installed by Sibley County personnel.

CHAPTER 3
STATIC AND DYNAMIC CHANGES OF AN EXISTING STRINGER TIMBER BRIDGE DUE TO THE ADDITION OF A TRANSVERSE DECK

3.1. INTRODUCTION

There are more than 1,400 timber bridges in Minnesota, many of them built in the 1950s and 1960s. Many rural timber bridges on secondary roads need to be replaced or improved to meet present day service, strength and safety standards. Because of state and federal financial assistance policies and the current funding levels, county engineers have been motivated to develop retrofit designs and procedures for strengthening and widening existing timber bridges to handle increased traffic and vehicle loads and to improve safety while keeping costs down.

This paper discusses a bridge that was widened to meet state aid standards for the geometrics of the highway segment in order for the county to obtain financial assistance to improve the county road. The total cost of the retrofit to the county to widen the bridge from 24 to 30 feet was $51,632. The cost to widen the bridge to 13.4 m (44 ft), as suggested by the state, was projected to be $115,035. The application for government funds widen to 13.4 m would have meant a delay of 2 -3 years. The life cycle of this second retrofit would be much longer than the existing bridge and, in all likelihood, the asset recovery would never be fully realized. The replacement cost of the bridge would have been $215,000. The state agreed to the low-cost retrofit.

A retrofitted timber bridge is normally expected to serve the community for an additional 20 to 40 years. Therefore, a close inspection is needed to evaluate the integrity of the existing deck and substructure to determine whether it will support a retrofit or require a total replacement of the bridge.

Bridge #6641 is a stringer timber bridge on County Road 8 which runs east/west in Sibley County, Minnesota. County Engineer Gene Isakson proposed a retrofit designed to widen and strengthen the bridge. The design widened the bridge six feet by adding a second, transverse deck.

The Minnesota Department of Transportation (Mn/DOT) asked the county to quantify the strength change and load distribution characteristics of the retrofit. This case study was completed to satisfy that request by reporting the evaluation of the changes in the static and dynamic behavior of the bridge resulting from the retrofit.

3.2. METHODS AND MATERIALS

The first step in the retrofit process was to remove the old railing and bituminous overlay. Figure 3.1. is a cross-section of the retrofitted bridge showing the transverse pier cap beams which were extended 91cm (3 ft) at each end using bolted scabs. A longitudinal beam composed of three 10.2 cm x 35.6 cm (4 in x 14 in) stringers supported at each end of the scabbed cantilevers support the extended retrofit deck. Grout was poured and leveled to screed boards that were secured to the outer edge of the existing transverse deck. Prefabricated, nail-laminated panels were then laid transversely and secured, and a new railing system was installed. Finally, a bituminous wearing surface was laid over the full width of the new deck.

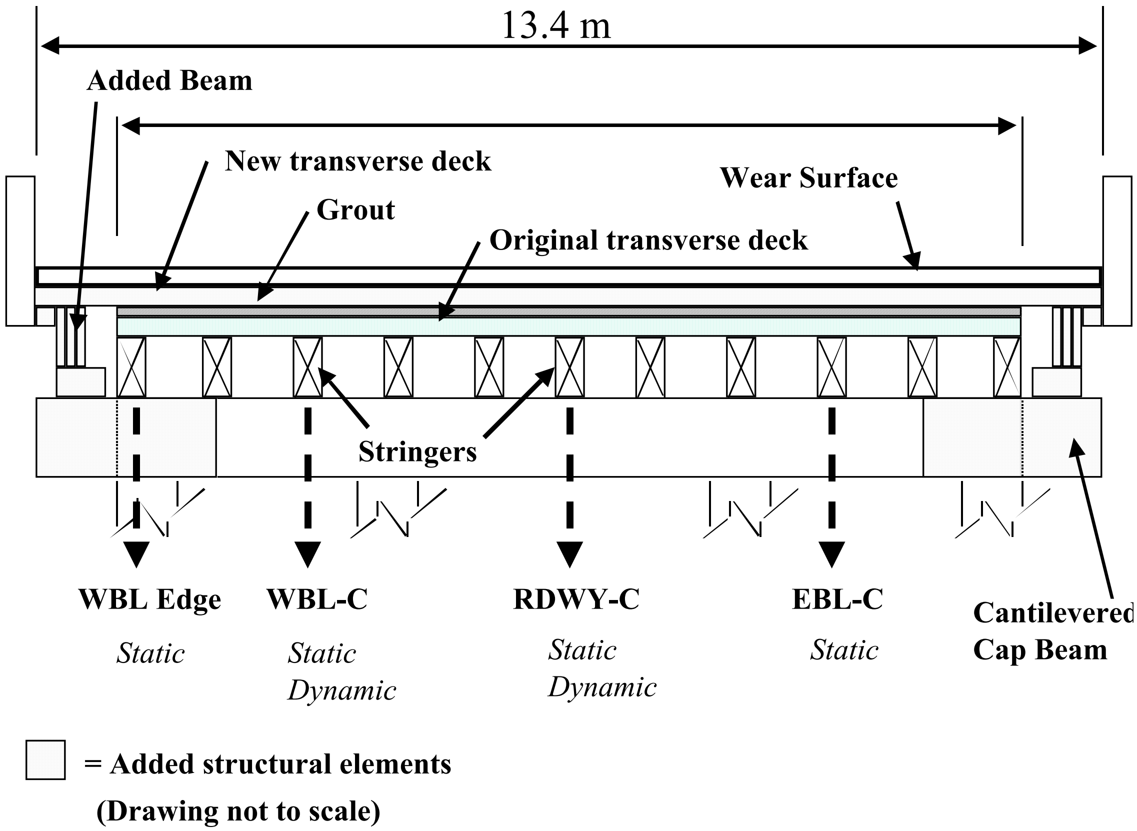

Figure 3.1 - Cross-section of Stringer Bridge

The bridge has three spans, a center span of 7 m - 62 cm (25 ft) and end spans of 5 m-18 cm (17 ft) each. Therefore, the total span of the bridge is 17 m - 98 cm (59 ft). A platform was built beneath the center span between the pilings to support the load testing instruments. Static and dynamic load tests were performed, both before and after the retrofit, at the mid-point of the center span.

Deflections were measured using linear variable differential transformers (LVDTs). As shown in Figure 3.1, these were placed under the mid-point of four center span stringers. Each LVDT was calibrated before and after use to establish a direct correlation between voltage and displacement (i.e. 2.54 cm = 10.000 volts). A digital voltmeter, which enabled deflection readings to 0.0025 cm (0.001 in.), was used to monitor the voltages during static load testing. For dynamic load testing, a dual pen, analog X-Y plotter was used to record deflections and vibrations. Since the plotter had only two pens, the deflections of only two of the four stringers (as shown in Figure 3.1) were recorded during dynamic load testing.

Finger clamps fixed the LVDTs to the top of threaded rods, 1.9 cm (0.75 in.) in diameter that extended from the platform up to the bottoms of the appropriate stringers. The ends of the LVDT cores were screwed into brackets secured to the underside of the four stringers. The following designations are used to identify the respective LVDTs. The LVDT at the stringer under the outer edge of the west-bound lane is referred to as WBL Edge. WBL-C is the LVDT attached to the stringer nearest the center of the west-bound lane. The LVDTs located at the roadway centerline and near the center of the east-bound lane are referred to as RDWY-C and EBL-C, respectively.

Sibley County provided two loaded trucks (#54 and #9103) for the static and dynamic load testing. The truck weights were adjusted to represent an American Association of State Highway and Transportation Officials (AASHTO) HS-20 design load, which is equivalent to the design load originally used. The rear tandem of truck #54 weighed 14,736 kg (32,420 lbs) with a total truck weight of 21,482 kg (47, 260 lbs). The rear tandem of truck #9103 weighed 14,882 kg (32, 740 lbs) with a total weight of 21,764 kg (47,880 lbs). The distance from the front axle to the forward rear tandem of each truck is 361 cm (11 ft 10 in.); the distance between the tandems is 135 cm (4 ft 5 in.). The front wheels are 201 cm (6 ft 9 in.) apart. The distance between the center of the dual wheels of each tandem is 183 cm (6 ft).

Four static load tests were done both before and after the retrofit. Static test #1 was performed with truck #54, in the west-bound lane, with its outboard wheels at the north edge of the bridge. Static test #1 measured deflections of the four, center span stringers with the front axle at the following nine locations: at each abutment, at the center of each span, at each pier, and at the quarter points of the center span. Static test #2 measured deflections at the same locations as test #1 but with the center of the rear tandem over the measurement points. Static test #3 measured deflections with truck #54 in the east-bound lane, with the rear tandem at the center of each span and at the quarter points of the center span. Finally, static test #4 took readings with both trucks on the bridge, with the center of their rear tandems at the same locations as those in static test #3.

The dynamic load tests measured deflections/frequencies as both trucks were driven abreast in dual runs over the bridge at speeds of 9.7 kph (6 mph), 32 kph (20 mph), 64 kph (40 mph), and 96 kph (60 mph). The LVDTs placed on the timber stringers at WBL-C and RDWY-C were connected to an analog plotter that recorded a graph of the bridge's structural response at both points to each successive velocity pass of the trucks.

3.3. RESULTS AND DISCUSSION

Figure 3.2 shows a representative selection of results from the static load tests before and after the retrofit. These results are plotted as single data-points and a curve of an elastic analysis of the bridge using a single stringer with a load distribution factor of S/4.0 (AASHTO Code Table 3.23.1, Appendix E page E1). The elastic analysis curve show the midspan deflection with respect to static test #1 (the front axle). The elastic analysis is for a single line of tires and is further adjusted by the load distribution factor. As stringer spacing is reduced, more stringers carry the tire loads, resulting in the reduced load distribution factor. The elastic analysis was performed to determine whether the midspan deflections observed in the static load tests were reasonable. The elastic analysis establishes an upper boundary for the static load test deflections. A stringer spacing of 58 cm (23 in.) was used for the elastic analysis because it yielded the best fitting curve. Regarding the results of the static load tests shown in Figure 3.2, the measured deflections seem reasonable.

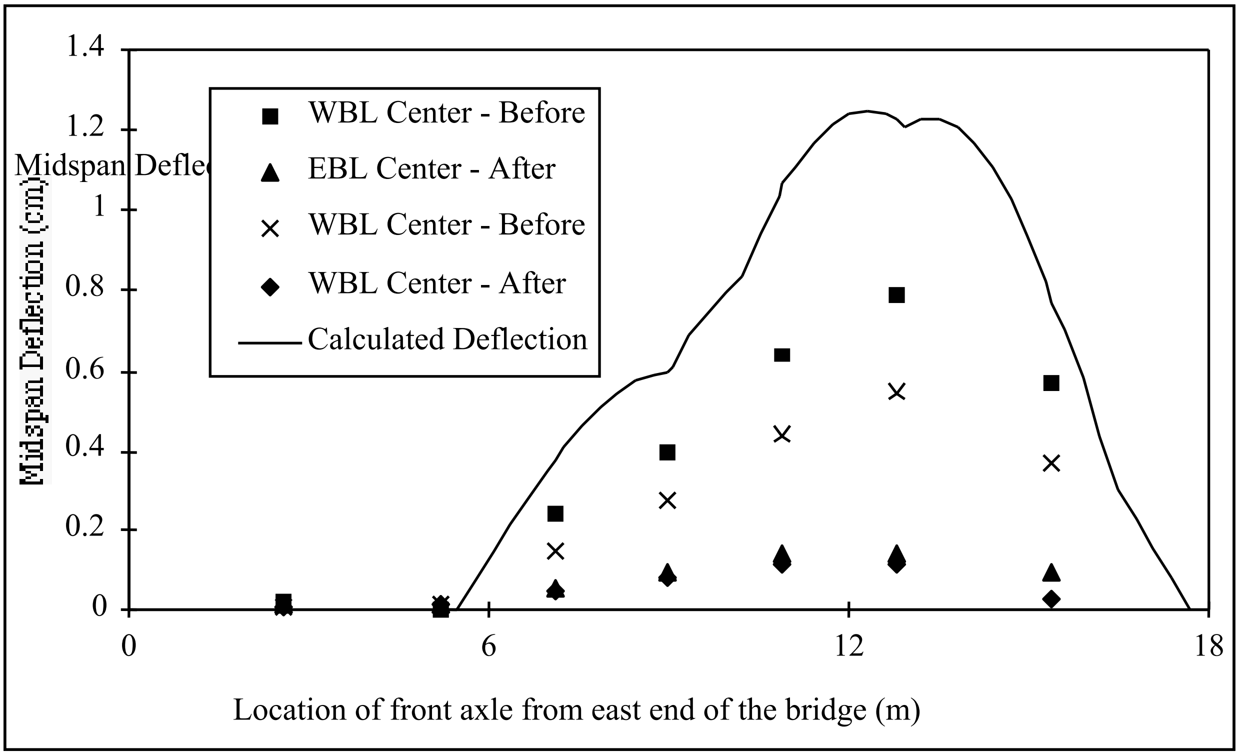

Figure 3.2. - Midspan Deflection of the Center Span: Before and After Retrofit

Maximum static deflection values are an important part of the tests because they determine structural performance. With one lane loaded, the maximum deflection before retrofitting was 0.80 cm (0.315 in.). With both lanes loaded, the maximum deflection was 0.945 cm (0.372 in.). The elastic analysis established a maximum deflection of 1.23 cm (0.492 in.).

With only one lane loaded, the maximum static deflection drops from 0.795 cm (0.313 in.) in the original bridge, to 0.541 cm (0.213 in.) after the retrofit, a 32 percent decrease. With both lanes loaded, the maximum static deflection is reduced from 0.945 cm (0.372 in.) in the original to 0.702 cm (0.276 in.) after the retrofit, a reduction of 25 percent.

Figure 3.3 shows the peak maximum deflections before and after the retrofit due to the rear tandems of two trucks traversing the bridge side by side at four different speeds. The magnitudes of the maximum dynamic deflections were similar to the maximum static deflections. There was significant vibration after the trucks left the bridge at 96 kph (60 mph) and 64 kph (40 mph) before the retrofit. As seen in Figure 3.3, the maximum dynamic deflections before retrofitting were 1.102 cm (0.434 in) and 0.782 cm (0.308 in) at RDWY-C and WBL-C respectively, and they occurred at traffic speeds of 32 kph (20 mph). After the retrofit, there is less variation in the defection between transverse location and truck speed.

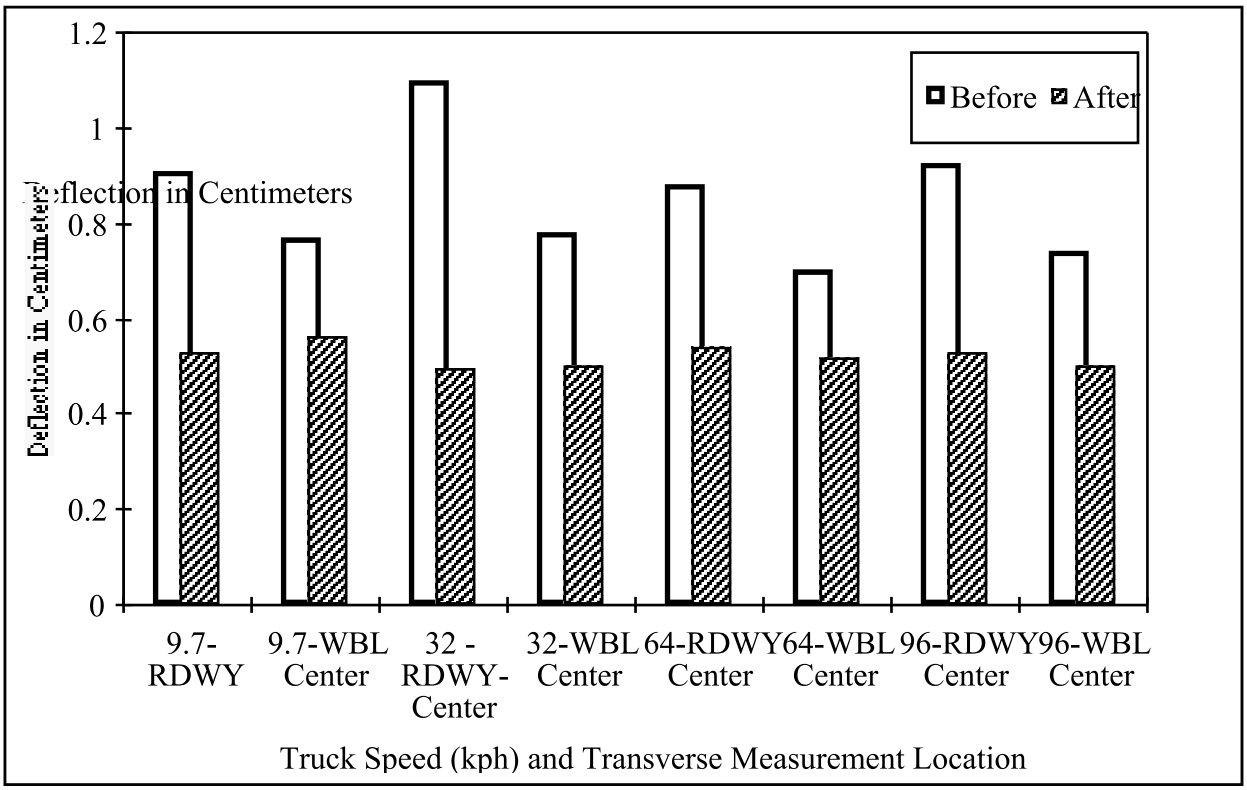

Figure 3.3. - Maximum Dynamic Deflections of the Center Span of the Timber Bridge Before and After Retrofit

The period of free vibration after the trucks had passed was approximately 0.3 seconds. A dynamic analysis was completed to determine an estimate of the theoretical natural period using a simply supported single stringer. The theoretical natural period was calculated to be 0.115 seconds, which does not correlate well with the actual natural period. It is possible that the theoretical model is not sophisticated enough to accurately reflect realistic boundary conditions, external and internal damping, and other contributing factors.

Some of the results from the static load tests after the retrofit are shown in Figure 3.2 An elastic analysis was also completed for the retrofitted bridge. In this case, a load distribution factor was backsolved to gain a better fit between the elastic analysis and static load test results. The corresponding load distribution factor was S/6.5, using a stringer spacing of 58 cm (23 in.). Note that by adding the 8.9 cm (3.5 in.) thick retrofit deck to the existing 11.7 cm (4.625 in.) thick deck, for a total deck thickness of approximately 20 cm (8 in.), the load distribution factor according to AASHTO standards would be S/4.25, compared to S/4 for the original 11.7 cm (4.625 in.) thick deck alone.

Figure 3.4 shows the before-and-after deflection profiles across the center of the bridge with respect to the location of the: 1) front axle of one truck and 2) rear tandem of two trucks. Comparing results before and after, the retrofit the transverse load distribution has been improved by the retrofit. The convergence of the front-axle data going from WBL-C to the EBL-C, also indicates better load distribution in the retrofitted bridge. With the truck traveling in the east-bound lane, the deflection of the stringer at WBL Edge increased after the retrofit, which indicates a better distribution of the transverse load as a result of the retrofit. The reduction in RDWY-C deflection with two trucks also indicates a more uniform load distribution between stringers in the center span of the retrofitted bridge.

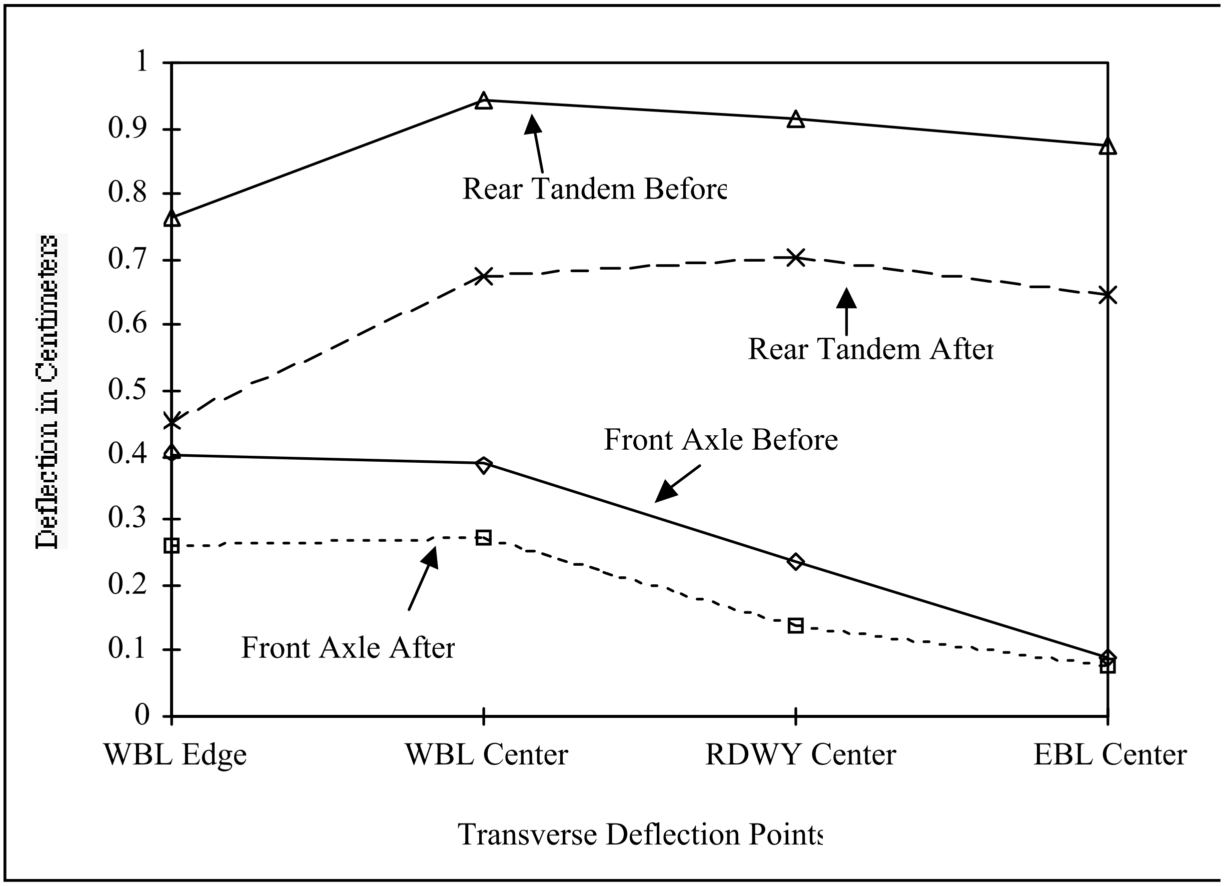

Figure 3.4. Transverse Deflections with Front Axle of One Truck and Rear Tandem of Two Trucks at Loading Points

In addition to the retrofit deck, the two outboard beams (added in widening the bridge) probably helped to improve the transverse load distribution. They would have carried some of the load, especially with just one lane loaded. For example, this would have reduced the load carried by the WBL Edge stringer. The deflection in these new stringers was not measured and, therefore, their effect cannot be quantified.

The tests show that the structural retrofit brought reductions in maximum dynamic deflections that range from 26 to 55 percent, for a mean of 37 percent. The tests further showed that, even at the maximum tested traffic velocity, there was no detectable vibration after the trucks left the bridge. After retrofit, the traffic velocities at which the maximum deflection occurred were: 64 kph (40 mph) at RDWY-C and 9.7 kph (6 mph) at WBL-C. Prior to retrofit, that figure was a uniform 32 kph (20 mph) at both locations.

A dynamic analysis was conducted to estimate the natural period for the retrofitted structure. This was calculated to be 0.119 seconds, closely matching a calculated value of 0.115 seconds for the original bridge.

3.4. CONCLUSIONS

1. For bridge #6641, adding a second deck to a stringer timber bridge effectively decreased the static deflections. Maximum midspan deflections were reduced 32 percent with one lane loaded and 25 percent with both lanes loaded.
2. The retrofit deck, and new stringers, appear to have contributed to improving the transverse load distribution. Working backwards from the results, the AASHTO load distribution factor was calculated to be approximately S/6.5.
3. Reductions in the measured, peak dynamic deflections ranged from 26 percent to 55 percent.
4. For the center of the bridge, the traffic speed at which maximum deflections occurred increased from 32 kph (20 mph) to 64 kph (40 mph). But at the center of the west-bound lane the traffic speed at which maximum deflections occurred dropped from 32 kph (20 mph) to 9.7 kph (6 mph).

CHAPTER 4

LOAD DISTRIBUTION: LONG-TERM PERFORMANCE OF A RETROFITTED LONGITUDINAL NAIL-LAMINATED TIMBER BRIDGE

4.1. INTRODUCTION

There are more than 1,400 timber bridges in Minnesota, many of them are nail-laminated and have been in service for 30 to 40 years**.** Typically, financial assistance for improving a rural road system is contingent on meeting increasing standards for width and capacity of its bridges. According to the federal rating system for bridges (PONTIS), many of these timber bridges have serious deficiencies and yet are too sound to warrant replacement, in these cases a retrofit of the existing bridge is the only option. Cities and counties must pay for the retrofits themselves so they must be cost effective.

When the desired service level of the modified bridge can be attained by widening of six feet or less, the retrofit can be done by placing a second, wider deck onto the existing deck and substructure, and with the added advantage of minimal traffic disruption. (Typically, when the bridge widening exceeds six feet, the substructure must be enhanced to support the additional dead load of construction materials, all at a greatly increased total cost in time and money.)

In Sibley County, Minnesota, county engineer Gene Isakson proposed to the state just such a retrofit design for Bridge #2624, a nail-laminated, timber bridge on Co. Rd. 8 near the city of Winthrop. The procedure would widen the bridge four feet by adding a four inch-thick, transverse wood deck to the existing deck. The state, concerned about load transfer between the two decks, requested an evaluation of the static and dynamic behavior of the retrofitted bridge. At the same, the performance of the grout layer between the decks was evaluated by the Civil Engineering and the Wood and Paper Science departments at the University of Minnesota.

Leon, Beltaos and Seavey (1993) found that the retrofitted bridge reduced static load test deflections by 26 to 29 percent for one lane, loaded. With two lanes loaded, deflections were reduced by 16 percent. Peak dynamic deflections were reduced with the retrofitted deck by 15 to 22 percent. Maximum deflections after the retrofit occurred at 9.7 km/h compared to 64.4 km/h for the original bridge.

Two grout mixtures were evaluated as fillers between the decks. One was Conbextra S grout and the other was a Portland cement grout. Three laboratory deck specimens, two with grout and one without, were tested for fatigue, for response to monotonic loading/unloading, and transverse load distribution characteristics. Results of this testing indicate that the stiffness of both grouted specimens was significantly greater than that of the ungrouted specimen. The retrofit laboratory deck with grout was not affected by the cycling in the fatigue tests. At a maximum load of 18,600 kg, the load-deflection curves for all three specimens remained in the elastic range. The maximum applied test load of 18,600 kg was more than double the design load of 7,260 kg for one tire.

Nail-laminated timber bridge #2642 was retrofitted in 1992 and was load-tested both before and after retrofit. The retrofitting of another Sibley County bridge, in 1995, offered the opportunity to retest Bridge #2624 for static and dynamic behavior, to compare with the test results in 1992. This paper reports the results of those comparisons.

4.2. EXPERIMENTAL MATERIALS AND PROCEDURE

The bridge is a three span, nail-laminated, timber bridge. The original deck is 7.92 m wide and 25.4 cm deep. The clear roadway width was 7.32 m. The center span is 4.87 m center-to-center of the piers, and the two outer spans are 4.72 m center to center of the piers. Longitudinal timbers are 25.4 cm by 7.62 cm by 4.87 m long for the center span. At the middle of each span is a 15.2 cm by 30.5 cm transverse stiffener beam attached to the underside of the nail-laminated deck.

Figure 4.1 shows the cross-section of the retrofitted bridge. Retrofit panels were 1.22 m wide, 9.14 m long and 10.2 cm thick, and contained pre-drilled grout holes spaced .91 m on center in both directions. After the retrofit panels were attached to the original bridge, a grout consisting of Type I portland cement, silica sand, water, and an admixture called Acryl 60, was pumped into the pre-drilled grout holes. The mix proportions by volume for the cement, silica sand, liquid mix were

8:8:5, with the liquid part made up of a 4:1 combination of water to Acryl 60. Acryl 60 is used to increase the bond strength of cement mixes and, thus, generally increases the compressive strength. The retrofitted clear roadway is 8.53 m.

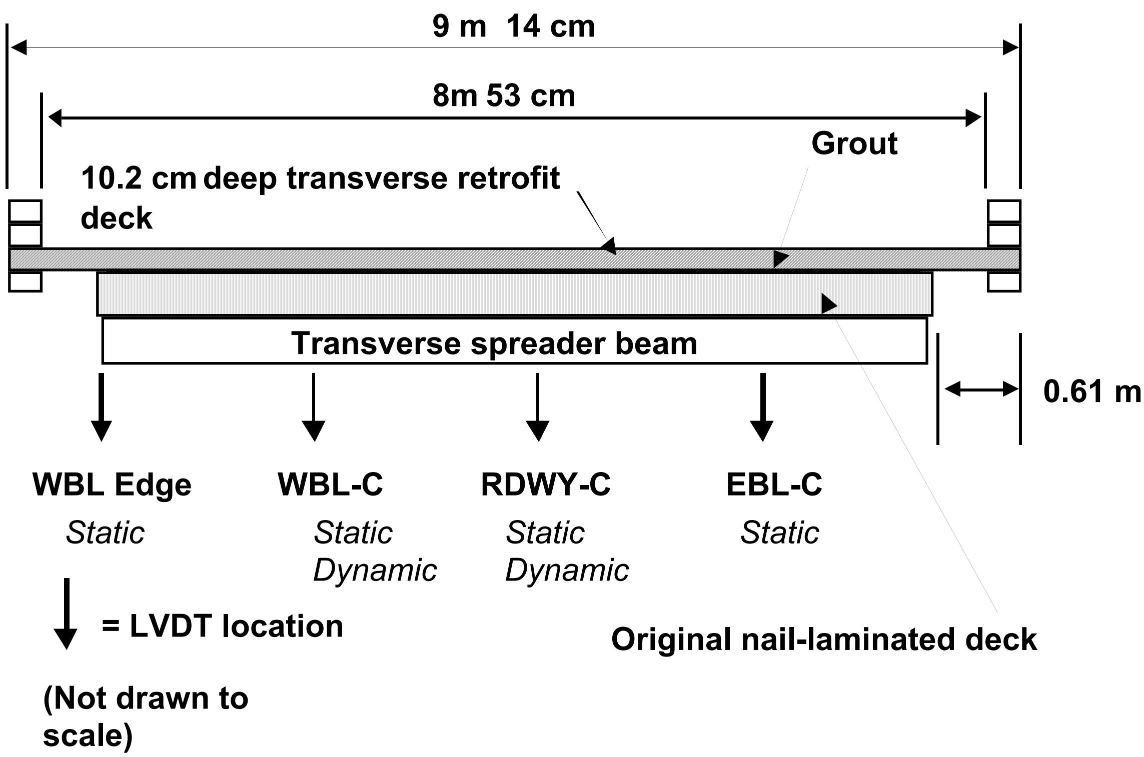

Figure 4.1 - Cross-section of Nail-laminated Bridge

A platform was built beneath the center span between the pilings to support the load testing instruments. Static and dynamic load tests had been performed, both before and after the retrofit, at the mid-point of the center span. The static and dynamic load test regimen in 1995 was identical to that of the 1992 tests.

Deflections were measured using linear variable differential transformers (LVDTs). As shown in Figure 4.1, these were attached to the underside of the center span transverse spreader beam. Each LVDT was calibrated before and after use to establish a direct correlation between voltage and displacement (i.e. 2.54 cm = 10.000 volts). A digital voltmeter, which read deflection to 0.0025 cm to monitor the voltages during static load testing was used. For dynamic load testing, a dual pen, analog X-Y plotter to record deflections and vibrations was used. Since the plotter had only two pens, the deflections of only two of the four LVDTs (as shown in Figure 4.1) were recorded during dynamic load testing.

Finger clamps fixed the LVDTs to the top of threaded rods, 1.9 cm in diameter, that extended from the platform up to the bottom of the appropriate position under the stiffener beam. The ends of the LVDT cores were screwed into brackets secured to the underside of the beam. The following designations are used to identify the respective LVDTs. The LVDT under the outer edge of the west-bound lane is referred to as WBL Edge. WBL-C is the LVDT attached under the center of the west-bound lane. The LVDTs located at the roadway centerline and near the center of the east-bound lane are referred to as RDWY-C and EBL-C, respectively.

Sibley County provided two loaded trucks (#54 and #9103) for the static and dynamic load testing. The truck weights were adjusted to represent an American Association of State Highway and Transportation Officials (AASHTO) HS-20 design load, which is equivalent to the design load originally used. The rear tandem of truck #54 weighed 14,736 kg (32,420 lbs) with a total truck weight of 21,482 kg (47, 260 lbs). The rear tandem of truck #9103 weighed 14,882 kg (32, 740 lbs) with a total weight of 21,764 kg (47,880 lbs). The distance from the front axle to the forward rear tandem of each truck was 361 cm (11 ft 10 in.) and the longitudinal distance between the tandems was 135 cm (4 ft 5 in.). The spread between the front wheels was 201 cm (6 ft 9 in.). The spread between the center of the dual wheels of each tandem is 183 cm (6 ft).

Four static load tests were conducted. Static test #1 was performed with truck #54 in the west-bound lane and its outboard wheels at the north edge of the bridge. This test measured deflections at four points across the stiffener beam, with the front axle at the following nine locations: at each abutment, at the mid-point of each span, at each pier, and at the quarter points of the center span. Static test #2 measured deflections at the same locations as test #1 but with the center of the rear tandem over the measurement points. Static test #3 measured deflections with truck #54 in the east-bound lane, with the rear tandem at the mid-point of each span and at the quarter points of the center span. Finally, static test #4 took readings with both trucks on the bridge, with the center of their rear tandems at the same locations as those in static test #3.

The dynamic load tests measured deflections/frequencies as both trucks were driven abreast in dual runs over the bridge at speeds of 9.7 kph (6 mph), 32 kph (20 mph), 64 kph (40 mph), and 96 kph (60 mph). The LVDTs placed on the stiffener beam at WBL-C and RDWY-C were connected to an analog plotter that recorded a graph of the bridge's structural response at both points to each successive velocity pass of the trucks.

4.3 RESULTS AND DISCUSSION

A representative result of the four, static load tests is shown in Figure 4.2. This figure gives midspan defection as a function of front axle location, Test #1 described earlier. The maximum midspan deflection for the retrofitted bridge (as tested in 1995) was 0.37 cm. In 1992 the maximum deflection for the retrofitted bridge was 0.48 cm as seen in Figure 4.2. Before the retrofit, the maximum deflection was 0.69 cm with one lane loaded. Maximum deflection based on front axle location was reduced 46% from the original bridge to measurements taken in 1995. The shift in the maximum deflection to the right may have been from a slightly different transverse location of the truck during the test from earlier tests.

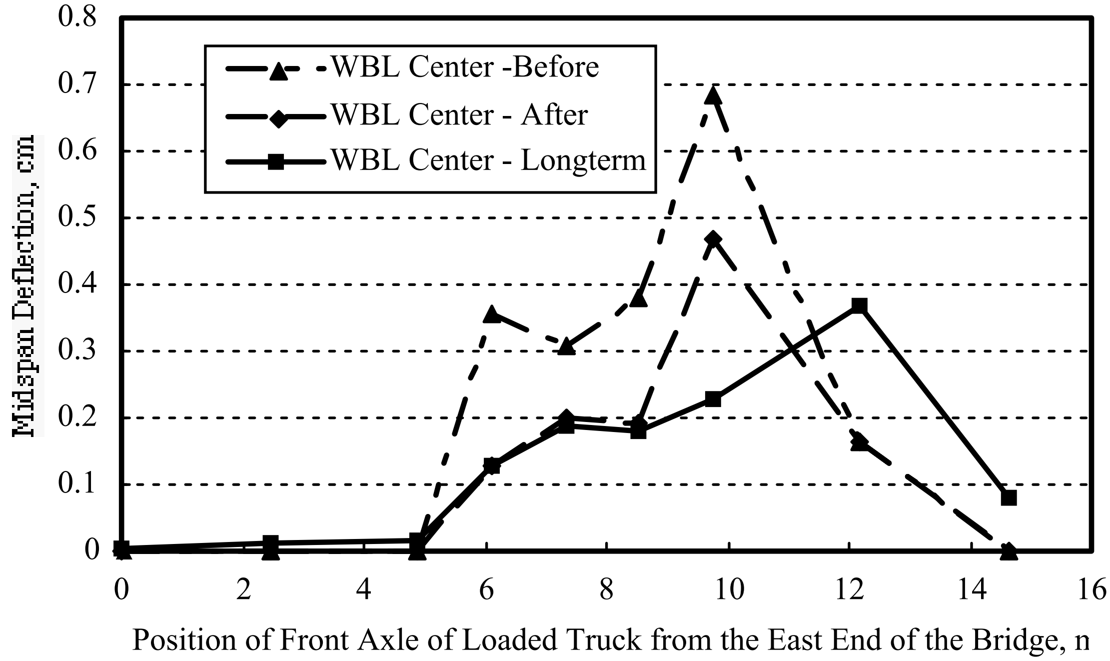

Figure 4.2 - Results of Static Load Testing of Nail-laminated Bridge

With both lanes loaded (Test #4), the maximum static deflection was 0.43 cm (0.17 in) in 1995. In 1992 the maximum deflection for the retrofitted bridge was 0.61 cm (0.24 in) and before the retrofit, with two lanes loaded the maximum deflection, was 0.76 cm (0.30 in). Based on the results from Test #4 a 43% reduction in maximum deflection was observed between the original bridge measurements and 1995 measurements.

According to these measurements, the retrofit continues to effectively reduce static deflections compared to those observed in the original bridge. The decrease in observed post-retrofit deflections from 1992 to 1995 is probably due to a change in moisture content of the retrofitted deck from near the fiber saturation point in 1992, to an equilibrium moisture content of 16% - 18% in

1995. Applying grout to the bridge deck would have increased the moisture content of the wood and thereby decreased the stiffness of the deck.

Figure 4.3 shows the observed deflection values from Test #4 for transverse load distribution as measured in 1992, both before and after the retrofit, and again in 1995. It shows that the immediate improvement seen in load distribution after the retrofit, has been sustained. Again one will observe a reduction in deflection from after the retrofit in 1992 to 1995. A possible factor is the lower moisture content of the transverse deck during the 1995 test, as mentioned earlier.

The observed maximum dynamic deflections show that the 1995 findings are similar to the results from 1992. Also, in both test years, the dynamic deflections tended to decrease as the speed of the test truck increased. Maximum deflections of 0.432 cm in 1995 occurred at both 9.7 kph (6 mph) and 32 kph (20 mph). In 1992 the maximum deflection was 0.633 cm and it occurred at 9.7 kph (6 mph). The original bridge had a maximum deflection of 0.741 cm at 64 km/h. The shifting of the maximum deflections to lower speeds is beneficial as the average speed of vehicles over this bridge is near the speed limit of 89 km/h. The maximum dynamic deflection has been reduced 41% from the original bridge in 1992 to the results measured in 1995.

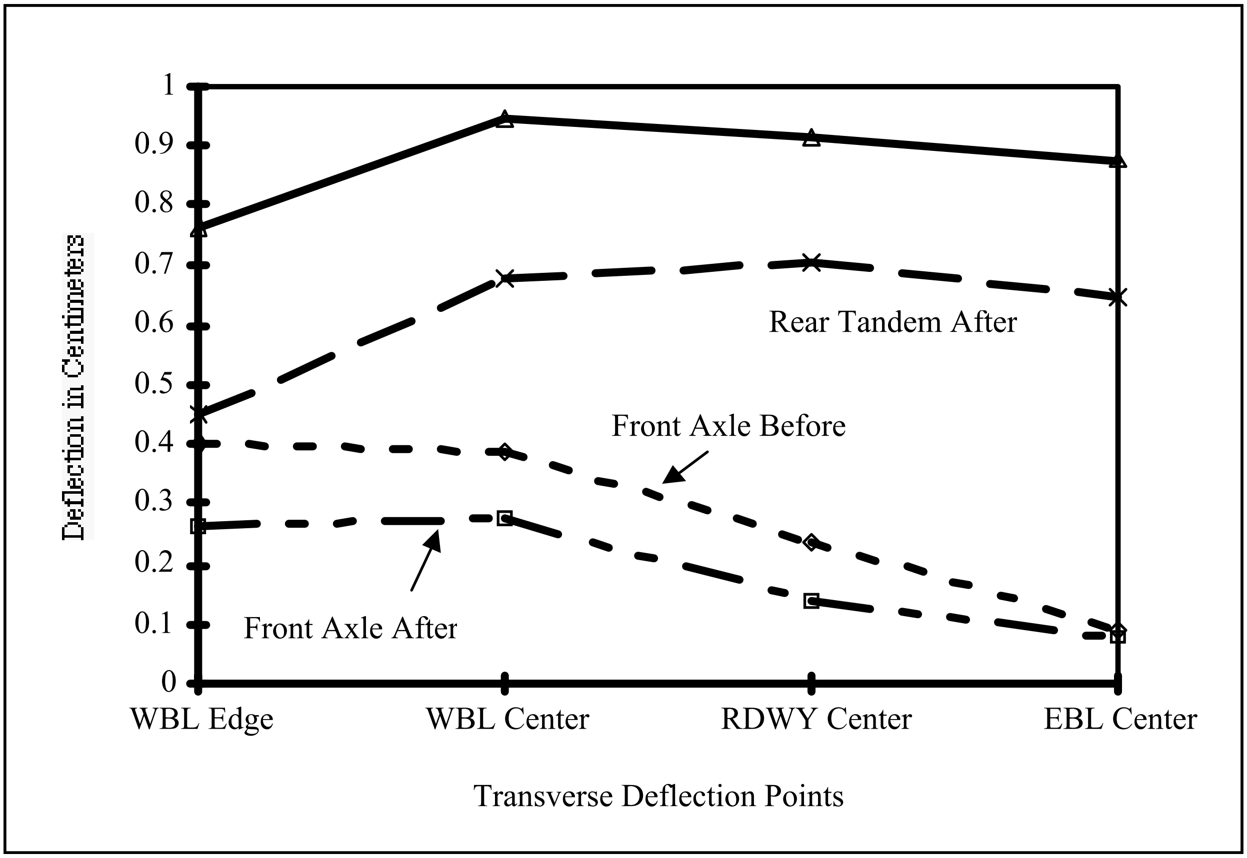

Figure 4.3 - Results Transverse Load Distribution of Nail-laminated Bridge

All maximum dynamic deflections recorded in 1995 were lower than those of the post-retrofit tests in 1992. Again, this improvement can be explained in part by the decrease in moisture content of the transverse deck over time.

As with the load tests of 1992, we noted minimal free vibration in 1995 after the test trucks left the bridge. Because of the low amplitude of the free vibration, it was not possible to determine the natural period of the bridge from the results of the dynamic test. We surmised, however, that the natural period of the bridge is similar to the 1992 results since the maximum deflections occurred at the same truck speed.

4.4 CONCLUSIONS

1. In the case of nail-laminated, timber bridge #2624 in Sibley County, Minnesota, static deflections observed in 1995 are similar to, or less than , those observed after the bridge was retrofitted in 1992. All post-retrofit testing for static deflections, compared to that of the original bridge, continues to demonstrate the favorable outcome of the retrofit.
2. Tests performed in 1995 also confirm that there has been no deterioration of the improvement in transverse load distribution characteristics that were observed in 1992 as a result of the retrofit.
3. With regard to peak dynamic deflections, test results in 1995 closely match those of 1992 in showing reductions ranging from 9% to 31% as compared to the original bridge. We note also that the maximum dynamic deflections observed in 1995 occurred at speeds of 9.7 kph (6 mph) and 32 kph (20 mph), which matches the post-retrofit dynamic test results observed in 1992.
4. Our tests indicate that the retrofit procedure used on this bridge continues to perform well in having strengthened the original, nail-laminated structure. Furthermore, since the retrofit panels distribute the load transversely without mechanical devices or prestressing strands, we believe this bridge will remain serviceable well into the future.

5.0 REFERENCES

Leon, R.T., D. O. Beltaos and R.T. Seavey. 1993. *Retrofit of Wood Bridges*. Minnesota Department of Transportation Report. MN/RC - 94/16.