FIELD APPLICATION OF Z-SPIKE REJUVENATION TO SALVAGE TIMBER RAILROAD BRIDGES

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Mr. Colin Hepker was point person in making all arrangements on site and off site for the conduct of the research done on the GLIDDEN SUB 69.11 Bridge in Eagle Lake, Texas. He traveled to Colorado State University for preliminary training on shear spiking and to help plan the field work. Mr. Kevin Louis performed the above tasks for the work done on the TOYAH SUB 534.35 Bridge in Stanton, Texas. He arranged and participated in numerous site reconnaissance visits leading to the selection of that bridge. He traveled to CSU for preliminary training on shear spiking and to help plan the field work. Subsequently, he assisted in the setup of instrumentation, organized the site access, site preparation, safety protocols, and communication within the UP network regarding all train traffic passing over the bridge during the periods of load testing. Mr. Geoff Pavlicek also assisted with instrumentation set-up, site preparation and technical support necessary to monitor the types of engines involved and loadings being carried by the various trains. Both these gentlemen conducted the spike installation and maintained site safety.

Dr. Jeno Balogh, of Metropolitan State College of Denver, participated in the site reconnaissance visit to the Glidden SUB 69.11 bridge site. He devised the instrumentation techniques and trained the personnel who did them on site for the subsequent load testing. CSU graduate student Karthik Rechan Rudraprasad directed instrumentation set up on that site, oversaw all load testing, and performed the data acquisition work. CSU graduate student Kevin Hedin assisted with the preparation of needed spikes, instrumentation planning, and data reduction for the Toyah SUB 534.35 bridge site.

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EXECUTIVE SUMMARY

The technique of rejuvenating wood and timber members by shear spiking (vertically inserting fiberglass reinforced polymer rods into deteriorated members) evolved over several years of laboratory research at Colorado State University (CSU). Specimens, including layered or split nominal 2 x 4 members, full-scale wood railroad ties, intentionally damaged individual railroad bridge stringers, a full-scale three span bridge chord, and deteriorated/damaged bridge stringers and chords obtained from the field were successfully enhanced in stiffness by the application. The successful stiffening of such members in the laboratory led to the need to examine the application of the technique in actual bridges in the field.

This report details the outcomes of shear spiking two open-deck, timber trestle railroad bridges and examining the effectiveness under applied loading. The bridges were made available to the researchers by the Union Pacific Railroad and were located in their Southern Region. The first site was located in the vicinity of Houston, Texas, specifically in Eagle Lake – a region of typically hot, humid climate, albeit being cool, damp, and sometimes rainy, conditions at the time of the study. The second site was located in the vicinity of Midland, Texas, specifically Stanton, Texas – in an extremely hot, dry climate at the time of the study. Each bridge was multi-span, but only one span was shear spiked in each case. Also, each was located along an in-service mainline railroad track and, due to rated physical condition, were scheduled to have their stringers replaced, making them available for experimentation but on a relatively fast-fuse basis.

The first bridge site proved to be nonproductive in a technical research sense, due to problems encountered in expediency needed to instrument the bridge for the load test. However, considerable knowledge and experience of coordinating with railroad management and field personnel to conduct the research study on an active railroad line were gained. It also demonstrated that the use of fixed static load required acquisition of a train car for loading and using a curfew time to spike the bridge and do the load test. That led to extensive changes to the instrumentation and load test methods for the second bridge. Also, in part based on the qualitatively observed stiffness improvement, the planned stringer replacement has been deferred for the time being.

For the second bridge, instrumentation was changed so as to be able to monitor bridge deflection response to the passages of actual in-service trains. Recording real-time responses of various trains moving at speeds ranging between 15–67 mph was successfully achieved. For the shear spiked span, the short gain in bridge stiffness was between 9% and 17.5%, depending on the direction of train passage. Overall, the average stiffness gain was 10%. After ten days of in-service train traffic, the overall average stiffness gain was 7–8%, a significant degree of retention. These gains were achieved despite shear spiking only the four exterior stringers of the two chords, and only two of them had previously exhibited the horizontal shear type failure for which the spiking technique is intended. One other exterior stringer was originally in good condition while the other had a significant flexural failure for which shear spiking is not applicable. Actual physical condition of the four interior stringers was not visible, but had been rated as in good condition in the bridge inspection report. Consequently, the gains were for spiking half the members, and only two had evidence of a need. One caveat is that loads were only known approximately, so values could have actually been somewhat higher or lower than just cited.

The observed results suggest the increased evidence of applicability of the shear spike methodology in the field is a tangible result of the project. As the shear spikes are made from commercially available rods and are easily installed and imbedded in the member, they are invaluable as a very low cost repair. In many installations, timber trestle railroad bridges are 50-plus years old but still necessary for daily

operation. It is increasingly difficult to obtain large size timber members needed to repair and upgrade such bridges. Hence, economic repair of bridges is vital as an alternative to replacement of members.

This project shows promise of leading to invaluable, affordable technology for repairing aged timber bridges on short and mainline railroads and on secondary roads. It is particularly critical to maintain safety and economic vitality of the nation's railroad infrastructure in rural areas. These vital links for the movement of agricultural commodities and other freight often depend on aging bridges. The research effort will assist bridge owners by providing a fundamentally new, more structurally effective, substantially lower cost alternative to presently limited and expensive repair methods based on fiber-reinforced composite patches, bandages, and wraps used in the past. Two needs are evident. For the Stanton site, the bridge stringers were subsequently removed and replaced on schedule shortly after the field study (well prior to this report). These members could be made available for physical examination of the spiking bond, seeing actual condition of interior stringers, and used for further laboratory load studies. Field experimentation for much longer periods of time are warranted and for a bridge with more predominant presence of shear type failures. If laboratory and field load tests using the shear spike approach continue to show the successful outcomes seen to date, there would be clear incentive to consider its increasing application and examination as an effective repair for timber railroad bridges.

1. INTRODUCTION

The U.S inventory of railroad bridges includes many long existing timber trestle railroad bridges of the open deck timber trestle type. As many are five decades old or older, they are now encountering much higher trainloads than when they were initially constructed. Increased axle loads due to heavier single cars and double stack cars have significantly raised the needed measured stiffness and load capacity of the bridges. Open deck timber trestle bridges have multi-ply (multi-stringer) chords as their primary superstructure component. Stringers involved have short span distances, are relatively deep, and experience high magnitude moving point (axle) loads. Stringer flexural measured stiffness largely depends on mid-depth horizontal shear stress integrity. Failed members can experience major shear cracking along the mid-depth and further cracking elsewhere if loads continue to pass over. If a rectangular chord member splits along its entire length at midspan, the flexural measured stiffness theoretically drops to one-eighth of its original magnitude. Also, extreme fiber bending stresses quadruple. To repair or rejuvenate such members, it is necessary to restore the shear transfer across cracked or split areas.

Fiber-reinforced plastic (FRP) and other such composites are extremely popular for infrastructure and in situ infrastructure repair, especially concrete members (columns and beams), and to a lesser extent, timber members. Common approaches are fiberglass wrap (bandages) or adding reinforcing plates (patches) to the sides of members. Material costs alone make these approaches relatively expensive and leave an unsightly appearance. These techniques can also require that the members be removed from the bridge for repair to be made, which is the case and is especially difficult for multi-ply chords of timber trestle railroad bridges. The FRP composites also degrade with time due to weather exposure. For timber, "shear spiking" (adapted from "Z-spiking," a construction method used in the aerospace industry), is emerging as a viable alternative to the above techniques for application to timber bridge members. Shear spikes are composite rods inserted from the top or bottom of the member into pre-drilled holes filled with an adhesive, which then cures to bond the spikes to the wood. They tighten the member by providing horizontal force transfer at material separations to restore overall flexural measured stiffness and add horizontal shear resistance, among other benefits. Shear spiking does not require member removal, and the repair is not exposed to weather. In timber railroad bridges, an added advantage is installation can be done into the top of the members; however, methods can be developed for insertion from the bottom.

2. PREVIOUS STUDIES

Researchers at Colorado State University (CSU) have been very active in pursuing this adaptation in the setting of rehabilitation of timber railroad bridge members.

A past MPC research project by Radford et al. [1] explored an innovative alternative to fiberglass wrap and patch repair techniques. A "shear spike" insert approach was tried on small wood members (based on 2" x 4" nominal sizes) and show promising results. Six 48-inch long nominal 2 x 4 beams were cut 22 inches along the neutral axis to simulate partial damage. The stiffness of each 2 x 4 was measured in 3-point bending prior to saw-splitting and then immediately after. The test span for this 3-point bend test was 34 inches, resulting in seven inches of each end of the beam overhanging the support. The resulting stiffness was approximately 50% of that of the original uncut beam for each of the six beams tested. Side-by-side 0.125 inch diameter fiberglass rods were then bonded in as shear spikes, starting just in the uncut 2 x 4, and 1.5 inches from the mid-span loading point. Each pair of fiberglass rods was allowed to cure for more than 24 hours prior to testing in flexure. Sequential pairs of fiberglass spikes were added at 3-inch increments, working toward the outer beam support. After each pair of spikes was added and allowed to cure, stiffness testing was undertaken. A total of six pairs of spikes were added, ending with the last pair just inside the lower beams supports, at 16.5 inches away from the mid span loading point. In general, four pairs of fiberglass rods were required before any stiffness increase was measured, and then the stiffness increased rapidly. Five of the six beams showed the highest stiffness after all six pairs of fiberglass rods were added. Of the six beams, four showed a statistically equal stiffness after the six pairs of fiberglass rods were added as the beams had prior to the introduction of the saw-split. The remaining two beams gained a significant amount of stiffness after spiking, but were only at approximately 80% of the original stiffness. Thus, results of the study show substantial rejuvenation, including a full recovery of the stiffness in four of the six test beams.

A subsequent completed laboratory project by Schilling et al. [2] addressed application of shear spiking to larger timbers (using railroad ties as a medium) with similar encouraging results. The samples were subdivided into two groups, Medium to High Quality Group (MH Group) and Lower Quality Group (L), based on a combination of their physical appearance characteristics and measured initial effective stiffness (apparent EI of a member) (EI = modulus of elasticity times centroidal moment of inertia). Five rows of two spikes each were then installed at each end of the member. Repairs improved the MH group by an average increase (gain) in effective measured stiffness of 51.0% (range of 33%-91%). Repairs improved effective measured stiffness of the L group by an average of gain 66% (range of 48%-99%). The overall average gain in effective measured stiffness was 58%.

In a later research project, Burgers et al. [3] showed modest to high recovery of effective measured stiffness occurred when shear spiking was done to intentionally badly damaged members of timber trestle bridge chord. Burgers [4] et al. [3, 5] performed laboratory testing of a full-scale, three-span, open-deck timber trestle railroad bridge chord that was strengthened through shear spiking. Using a chain saw, researchers intentionally damaged the center span by cutting the four stringers horizontally along the neutral axis. A cut was made over one quarter of the length of the chord at one end (south end based on orientation in the laboratory). As the chain saw length did not reach the mid-width of the chord, two inches of material was left in each stringer. The chord was then ramp loaded to cause that material to crack. That also resulted in unintentional propagation lengthwise extending partially into the middle region (between the load points) of some of the stringers. The chord was reloaded before and sequentially during and after shear spiking of the damaged regions. After all cuts and cracks occurred, the effective measured stiffness dropped to 48% of the undamaged value.

Two spikes per stringer were placed in a lateral row at the south end of the member, resulting in a 12% regain of the prior lost (drop in) effective measured stiffness (cumulative regain of 12%). The south part of the unintentional crack in the region between the load points was then repaired with three or four (the reference document is unclear on the number added) rows of shear spikes per stringer, resulting in an additional 29% regain of effective measured stiffness (cumulative regain of 41%). The north part of the unintentional crack in the region between the load points was then repaired with three rows of shear spikes per stringer, resulting in a 6% drop in effective measured stiffness (cumulative regain of 36%). The drop was attributed to having cut some good quality timber, which dominated over the additional spikes. Three rows of spikes per stringer were added beyond the first pair placed at the south end, resulting in a 17% increment of regained effective measured stiffness (cumulative regain of 53%). An additional four rows of spikes were inserted in the south part of the unintentional crack in the region between the load points, resulting in a 14% drop in effective measured stiffness (cumulative regain of 39%). At that point, the effective measured stiffness was at 132% of the value measured after all the cuts were made and at 72% of that of the original uncut chord.

After a period of time, a second chain saw cut was made over one quarter of the length of the chord at the north end of the chord, through the four stringers This time the uncut mid-width material was then cut by use of a two-man wood cutting hand saw, and no crack propagation occurred. The beam was retested before the cut was made, and it had exhibited a 7% regain of effective measured stiffness (cumulative regain of 46%), over time. After the cut was made, a 25% drop in effective measured stiffness was observed (cumulative regain of 21%). Five rows of spikes per stringer were then installed in the north end of the chord, resulting in a 23% regain of effective measured stiffness (cumulative regain of 44%). Interestingly, 92% of the effective measured stiffness lost due to the north end cuts was recovered and the resulting value was at 99% of the effective measured stiffness value measured in the retest before that end was cut. The final effective measured stiffness was at 116% of the value that existed after the north end was cut and at 75% of that of the original uncut chord.

The outcome of shear spiking was a modest increase in effective measured stiffness due to spiking the cuts. For the south end and middle zone cracks, the regain due to spiking them was 39%. The resulting effective measured stiffness was 32% higher than that existing after the cuts. For the north end cuts the regain due to spiking them was 92%. The resulting effective measured stiffness was 16% higher than that existing after those cuts. However, the regains are perplexing, being low for the south end/middle zone repairs and very high for the north end repair. Overall, for all cuts/cracking the effective measured stiffness was 75% of the original value of the chord.

More recently, Gutkowski and Forsling [6] conducted a laboratory study of the effects of modest cycle repeated load on intentionally damaged individual chord members. The base members were newly fabricated members donated by the Transportation Technology Center, Inc., of the Association of American Railroads for a past MPC project. These members were manually inflicted with different types of intentional severe damage. A chain saw was used to make horizontal cuts into the sides of the members over one-third of their length from each end, leaving the middle third of the length intact. Some were cut only at mid-depth and others were also cut at the one quarter point up from the bottom. For each situation, two types of specimens were made, “full cut” and “partially cut.” Full cut specimens had cuts that penetrated through the full eight-inch width of the member. Partially cut specimens had cuts on each side that penetrated three inches into the width. The damaged members were then shear spiked.
Load tests were conducted under ramp loading before and after spiking. After being intentionally damaged, on average, the eight stringers had lost approximately 38% of their initial EI. After the insertion of five rows (two spikes per row) of shear spike reinforcement at each end of the members, the average regain was 66% of what had been lost. The four stringers with full cut(s) had dropped to 34% of their initial EI and regained of 64% of what had been lost. The four stringers with partial cut(s) lost an average

of 12% of the initial EI and regained 67% of what had been lost. Subsequently, low cycle repeated loading and failure loading were conducted. The cyclic loading resulted in showing modest to no effect on the initial regain of effective measured stiffness observed after spiking.

The conclusion was that shear spiking was highly effective in all members, but the partially cut members had very little absolute regain to achieve. This is logical as in the partially cut members, the horizontal shear transfer was still present via the small width of member remaining. Thus, the effect of cutting was to drop the EI value by the small magnitude associated with the thin slits in the member. If under load, the remaining inner material beyond the slits was to fail, the member would be split at those levels, constituting the condition of the full cuts. With a full cut at mid-depth over its full length, the member is split in half with no horizontal shear resistance (ignoring frictional resistance) at mid-depth. The value of I is reduced to one-quarter of the value of the full cross-section. For two full cuts described above, the value of I is reduced to 11/64 of the value of the full cross-section. If the cuts do not penetrate the full width and the remaining material does not fail under loading, the reduction of I is very small. The percentage regained due to spiking is still high but the overall absolute effect of spiking is minor in magnitude.

Based on the success of the above work, laboratory studies were pursued on deteriorated and damaged bridge chord members recovered from actual bridges. Via the Association of American Railroads (AAR), a general email call for available members was made throughout the USA. Among the repliers, Tomasz Gawronski, Director of bridge maintenance (DBM) for the Union Pacific Railroad – Southern Region (UPR – SR), contacted the PIs. In Texas, his field engineers and work crews had been attempting repairs on in-place, damaged stringers of open deck timber trestle bridges by use of mechanical steel connectors. As there are approximately 8,000 timber trestle bridges in the UPR – SR inventory, such repairs are of significant interest to that railroad. However, indications were that the method they were attempting was not proving successful. Hence, Gawronski expressed his interest in cooperating with CSU researchers to explore shear spike technology. Members that had been removed from actual field bridges (for purposes of retrofitting the bridge with new ones) were sent to CSU for laboratory implementation of shear spiking and load testing by Miller et al. [7]. In this study, the members had been in long-term service and exhibited various degrees of physical degradation and damage. One of the three specimens was a four-ply cord member that was substantially damaged. It was incrementally repaired with load tests (ramp load to 15 kips) performed at various intervals. That four-ply chord exhibited a significant increase in effective measured stiffness (267%) as a result of the spiking. At the completion of the spiking, the specimen was loaded to higher load levels (39 kips) in order to demonstrate the effectiveness of the spiking at load levels reflective of actual train loads. The 39-kip level was a load level being considered by the AAR as a potential new design axle load. First the specimen was ramp loaded to 30 kips when the load test was stopped due to a loud cracking noise. The specimen was inspected for damage to the spikes or timber chord members. No damage was seen so testing resumed. The specimen was then ramp loaded to 39 kips. During this test, there was some loss of stiffness observed, as the plot of load versus deflection leveled somewhat under higher loads. After the completion of the 39-kip load test, the specimen was retested for its effective measured stiffness. The effective measured stiffness had decreased to 179%. This was a decrease from the maximum observed effective measured stiffness increase, but a substantial increase over the initial effective measured stiffness of the chord. It was concluded that the spiking procedure is a potential option for repair of railroad chord members that are damaged from service conditions over the years.

Later, an additional four-ply specimen with substantial damage was tested as a part of this research study by Gutkowski et al. [8]. In this study, the specimen was incrementally repaired. Ramp load tests were performed at various times after each increment of spiking. Generally, load tests were performed at three hours, six hours, nine hours, and 24 hours after spiking. This testing protocol and the results demonstrated that the spikes were already somewhat effective after three hours of epoxy curing time as the effective measured stiffness had already begun to increase. The effective flexural measured stiffness continued to increase slowly until the epoxy was fully cured. Once all spikes had been installed, the effective measured stiffness of this specimen had increased to 291% of the original value. At the completion of the spiking, the specimen was ramp load tested to 39 kips three times. The first loading showed a decrease in effective measured stiffness as the load increased. However, the subsequent two ramp load tests showed a consistent effective measured stiffness that was 202% higher than the initial value. These remarkable increases in measured stiffness are likely a result of the highly controlled loading and spiking environment of the laboratory as well as the extensive damage existing in the members.

Prior to the above two chord tests, Miller et al. [7] conducted spiking on two good quality specimens. These other two specimens were a one-ply chord and a three-ply chord, respectively, that were utilized to refine the spiking procedure, including a selection of the epoxy used, the thickener, the size of hole drilled in the members, as well as the preparation of the spikes. The one-ply specimen exhibited a 20% gain in effective measured stiffness due to spiking. The three-ply chord exhibited only a 7% gain in effective measured stiffness relative to its "as received" state. These results further confirm that shear spiking does not significantly improve the measured stiffness of good quality members. Instead, the installed spikes likely act only as a reserve reinforcement (such as stirrups in concrete beams) to inhibit the onset of flexural failure that might occur at lower load levels in members without the shear spikes present compared with load levels that would be reached if the shear spikes are present.

3. RESEARCH OBJECTIVE

The objective of the project described herein was to explore the field work challenges, effectiveness and performance of installing composite shear spikes to salvage in-place damaged and/or deteriorated timber chord members in actual in-service timber trestle railroad bridges.

4. RESEARCH APPROACH/METHODS

The Director of Bridge Management (DBM) for the UPR-SR offered to make an in-service bridge available for the continuation of the research into the aspects of field application. The cooperative work plan consisted of the following:

1. The DBM would identify candidate bridges for shear spiking. Identification of candidate bridges was to be based on structural condition of the chord members (based on inspection reports) and location of the bridge for ease of accessibility. A small number of spans and low train volume were also desired.
2. Inspection records for candidate bridges would be examined by the researchers to select one or more bridges for consideration for spiking and load testing.
3. Joint reconnaissance on site visits would be made to examine the condition of the bridge chord members and applicability of shear spiking to the selected bridges.
4. After selecting the bridge to be used, the spiking locations and sequencing, field preparation, and load testing program and timing of events would be planned.
5. The bridges would be instrumented for electronic displacement data acquisition, e.g., by an in-place data logger with sufficient acquisition rate and total time capability connected to potentiometers.
6. Prior to shear spiking, an attempt will be made to record data for periodic, selected actual trains passing over the bridge. Trainloads would need to be somewhat consistent, such as in the case of coal cars, which more or less have very narrowly varying total weight. The UPR would assist in determining when such trains (not necessarily coal cars) will be passing over the bridge.
7. A controlled test under a fixed known car load positioned on the bridge would then be conducted, before any shear spiking.
8. After the completion of shear spiking, the controlled test loading would then be repeated. If possible, intermediate load tests might be done at selected stages of the shear spiking process. That would be dependent on the extent and sequence of shear spiking involved versus time constraints of testing an in-service bridge. It would also be dependent on the actual bridges selected, the extent and dispersion of deterioration/damage, and the time element involved to cure the adhesive after each spiking increment. A balance between the overall time duration needed and repeated availability of a load car or locomotive would be a main consideration.
9. If possible, displacement data would be collected automatically over time, for actual trains subsequently passing over the bridge.
10. Results of the before and after loadings would be examined to assess the effectiveness of the shear spiking on overall measured stiffness of the bridge.

In conjunction with the above, at an appropriate time after the first bridge was selected, a UPR – SR maintenance manager would come to CSU for education about the shear spike research. Details of the conduct and outcomes of the sequence of laboratory studies summarized above would be covered as well as the techniques for doing the shear spiking. A mock load test of a beam specimen would be shown in a laboratory setting, to convey how to place and utilize the needed instrumentation and acquire the data. Detailed preparations for the timing and conduct of the shear spiking and load tests would be done.

5. CONDUCT OF THE RESEARCH

5.1 Selected Bridges

Using the above research approach, two bridges were eventually utilized in the shear spiking and load test program. As the experience contained in investigating the first bridge was pertinent to the selection and process of investigating the second bridge, each spiking and test program will be separately described in its entirety.

5.2 Bridge No. 1 – GLIDDEN SUB 69.11

Based on inventory and inspection report searches done by the DBM and his staff, the GLIDDEN SUB 69.11 bridge located in Eagle Lake, Texas (in the vicinity of Houston, Texas), was suggested as a good candidate for the project. A complete inspection report (for October 29, 2009) (see Appendix A) was accessed and provided all needed technical details. Photographs from the site of the GLIDDEN SUB 69.11 bridge are included in Appendix B.

5.2.1 Geometry

Based on its inspection report document, the primary features of the bridge were available. The bridge is on a mainline track and rated for 50 mph freight train velocity and 50 mph passenger train velocity. It was built in 1946 and rehabilitated in 1994. The bridge passes over a small creek and is accessible via a short dirt road from a nearby highway. It is a five-span, open-deck, timber trestle bridge with a total nominal length of 75 feet (actual length is 74 feet, 7 inches). All spans are essentially equal at a nominal length of 15 feet. The two chords are composed of four-stringer closely packed stringers. "Closely packed" chords imply the stringers had from little to no side-by-side clearance. The cross ties are timber and rated as being in very good condition. The steel rail is a 136-lb. continuously welded section. The substructure is timber piling and caps at the abutments and pier bents. The underside of the superstructure is reachable for work without the need of ladders. Walkways provide access to the top of the bridge.

The typical layout for timber trestle bridges is shown in Figure 5.1.

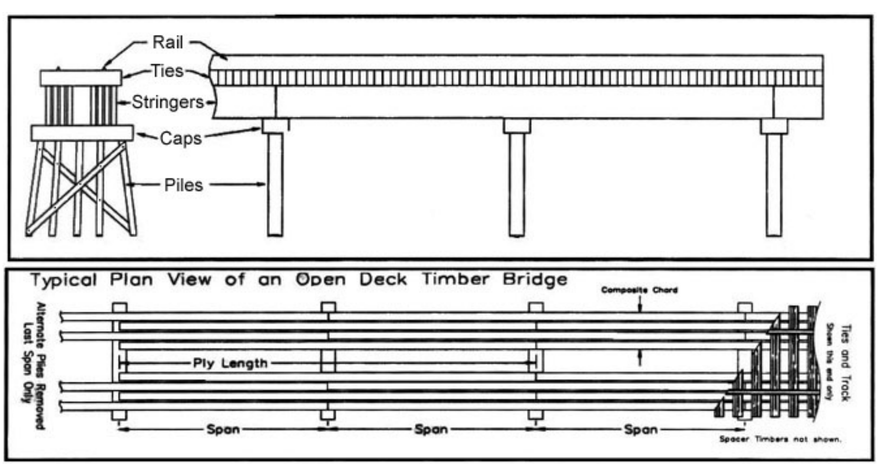

Figure 5.1 Typical Open-Deck Timber Bridge

For this bridge, the nominal component dimensions are included in the inspection report. Main elements are: Piling, 14.0-inch diameter; Caps, 14.0 inches x 28.0 inches x 12.0 feet; Stringers, 7.50 inches x 17.25 inches; Cross ties, 10.0 inches x 8.00 inches x 10.00 feet; and totaled 60 in number for the length of the bridge.

In the standard inspection report used by the UPR, the observed conditions of the bridge members are documented in a coding diagram, indicating the various members in a standard pattern and with each marked according to a condition code. As an example, refer to the inspection report for this bridge (Appendix A). Looking at the coding section of the report is counterpart to standing under the near end of the bridge and looking <u>upward</u> at its underside and identifying the condition of its members. The near end is the end that is closest to the beginning point (zero mileage marker) of the line, that is, the mileage distance is increasing as one goes from the near end to the far end of the bridge. On the inspection sheet the lateral lines of abutments and intermediate piers are numbered (in this case, numbered as lines 1 through 6) on the left side of the sheet and ordered downward. Line 1 corresponds to the abutment at near end of the bridge, and its members are shown in a coding that indicates the condition of the piles and caps in that line. The stringers spanning any two supports are shown between them, also in coded form. Thus, the first span (span 1, but not shown by number) is at the top of the coding diagram, between support lines 1 and 2. The stringers of a span, being visually seen from left to right (as stringer, 1, stringer 2, etc.) while looking upward, are thus coded in that same order on the inspection report. Stringers 1, 2, etc. are thus entered (without use of numbers) from left to right in the inspection coding diagram.   Based on UPR coding, a stringer's inspected condition is coded via a number/letter label. In Appendix A, stringers 1 through 8 in span 1 are coded 4B, O, O, 4A, O, O, O, O, respectively.

Based on the inspection report for the GLIDDEN SUB 69.11, a variety of conditions existed for the stringers. Stringers identified as having a horizontal shear crack at about mid-depth are labeled by the code 4A. In this bridge, 10 stringers had been coded as 4A. Also, all interior stringers were coded with an oval ("O"), signifying its condition was satisfactory. The predominant arrangement of stringers in an open-deck, timber trestle bridges is as depicted in Figure 5.1. The stringers are arranged in a staggered pattern of two-span continuous members, with alternate single-span stringer infilling at the abutments. However, the inspection report does not distinguish these conditions, i.e., does not indicate where the end-to-end butt joints occur. Thus, from the inspection report, one does not know if two consecutive codes along a stringer line are for the same two-span stringer, or not. Thus, spiking pattern was to be determined on site.

5.2.2 Reconnaissance Visit

On December 3-4, 2009, two researchers made a visit to the bridge site, escorted by Gawronski and accompanied by other appropriate inspection and maintenance personnel. It was a snowy day (very rare for Houston) and road conditions slowed the travel time to the site to more than two hours. Figure B.1 shows an overall elevation view of the site and bridge. All three spans were readily accessible laterally and vertically without need of a ladder from the ground below. Normal water flow was present under two of the five spans, but at low depth and relatively still flow. However, that waterway condition eliminated those two spans from consideration.

5.2.3 Actual Condition of Stringers

The physical condition of each stringer was viewed, indicating some stringers to be in worse and others in better physical state than suggested by the inspection codes assigned to them. Not all spans coded 4A, thus suggesting a priority for spiking, proved to be so. In some cases, what was coded as a 4A horizontal crack was actually just the visible outside opening of a seasoning check. One two-span continuous stringer had an odd, but severe, crack (through its width) that emanated near an upper point along span 4 and extending diagonally downward to the outer bottom surface near the end of span 5.

5.2.4 Timing of the Spiking and Load Testing.

The bridge was to be spiked and load tested in late spring of 2010. Subsequent to the field reconnaissance trip, Mr. Colin Hepker, a field engineer of the UPR – SR, traveled to CSU from March 4-5, 2010, for training about the shear spiking technique, adhesive types and curing time vis-a-vis gain in bond, and conduct of placing and using instrumentation and observing a simulated load test in a laboratory setting to demonstrate the data acquisition process. Planning was then done for the field project. As the existing bridge was scheduled for near-term stringer replacement, an accelerated schedule was preferred by the UPR. Coincident with that planned retrofit of the bridge, the nearby track on the Glidden line was also scheduled for a period of maintenance and repairs in about two weeks. Thus, about a five-day window of daily "curfew" time would be available during that time. Curfew was highly opportune, as it is a time period where, on a daily basis, all train traffic is halted on a segment of a train line from about 8 AM to 4 PM, to allow for the scheduled maintenance and repair work. To take advantage of that opportunity, it was decided to schedule the spike work and load test for that period of time, specifically March 15-18, 2010. As a consequence, the lead time before arriving on site was condensed and time available on site was also very tight – limited to three to four days. To partially alleviate this, travel by the research team to the site was to be by air from Denver to Houston and ground travel of approximately 1.5 hrs each way. Thus, the team would arrive about noon on the first day, travel to the site, and install instrumentation. On the second day, a load test would be done before spiking and then spiking would be completed. On the third day, the load test would be repeated after the adhesive had been curing for approximately 12-18 hrs. A return visit would be made one to two weeks later to repeat the load test after a period of further curing and additional train traffic.

5.2.5 Instrumentation and Data Acquisition

As timber trestle bridges are very old, and despite maintenance work, they have experienced an enormous volume of heavy, moving trains and they inevitably loosen over time. This occurs simultaneously with material deterioration due to weathering effects. These effects are evident in gaps that develop between members (e.g., between a stringer end and the pile cap), loss of tightness in connections, and cracks within the members themselves. In addition, piles can have some vertical pumping motion under passing train loads, thus deforming the attached cap. Consequently, a portion of the deflection of the bridge superstructure is a result of the movement between the components of the bridge members to close the gaps and cracks as the members deflect under the applied load of the train. Such free and differential movements were visibly apparent in this bridge and had to be taken into account in devising the instrumentation set-up.

The instrumentation and data acquisition were to be transported as checked airline baggage. To minimize the baggage space and time needed for installation, but still collect pertinent data, the instrumentation set-up was efficiently designed so as to be minimal in extent but able to measure displacement of all stringers of span 5. A key aspect of the scheme was to automatically produce an average measured displacement value for each chord. To accomplish this, a cascade set-up of hangers and short wood slats was conceived (Figure B.2).

From each stringer of a chord, a metal hanger is hooked to an eyebolt embedded into the bottom of the stringer. The eyebolt is placed at mid-span and mid-width of the stringer. This is done in all four stringers of a chord – resulting in four hangers. The bottom ends of each pair of adjacent hangers (one below an exterior stringer and one below an interior stringer) is hooked to one of two eyebolts embedded in the opposite ends of a short wood slat – resulting in two suspended wood slats. Thus, each hanger then is hinged at both its ends and able to freely settle into position, as are the wood slats.

At the mid-point on the bottom of each adjacent pair of "suspended" wood slats, another hanger is hooked (hinged) to an eyebolt and then hooked (hinged) at its bottom end to another eyebolt embedded in a lower, "sub-suspended" wood slat. Thus, two sub-suspended wood slats result. From each of these two suspended wood slats, midpoint hangers are attached to eyebolts and hinged at their bottom ends to eyebolts embedded into a single lower level, "sub-sub-suspended" wood slat.

Finally, the free end of a string potentiometer is attached to a hook embedded on the underside of the single sub-sub-suspended wood slat. The string extends upward from a string potentiometer base installed below, which is to be fixed to a wood cross-tie firmly placed on the ground below. This single potentiometer reading constitutes the directly measured average of the mid-width vertical displacements of the four stringers at the top of the cascade – at the point along the stringer length where the first hanger is located.

A cascade was done at mid-span of each chord and as close to the ends of the chords (adjacent to the pier cap and abutment cap) as physically possible (Figure 5.3). By subtracting the average of the two end potentiometer readings from the mid-span potentiometer reading, the net average mid-span displacement of the four stringers in the chord at mid-span is obtained – that is, the value with all measured end support movements eliminated. By conducting a load test before and after spiking, the effect on this net displacement can then be observed as an indicator of measured stiffness improvement achieved in each chord. (NOTE: all hanger material was common stock purchased at a hardware store and selected so as to provide close to horizontally level wood sticks at each suspension level as expectable. However, despite this intent, any lack of levelness has no effect on the accuracy of the measured change in vertical displacement that occurs under loads applied to the bridge).

Unfortunately, the actual field conditions necessitated several important changes from the ideal instrumentation set-up envisioned above. On arrival at the site (in mid-afternoon on the first day, March 15, 2010), it became evident that a large gravel incline at the abutment end had not been removed as planned. Elsewhere, the vegetation had grown significantly and added to the ground surface constraints. This situation prevented stable placement of cross-ties on the ground for mounting string potentiometer bases. An expedient, on-site decision was made to alternatively install a nominal 2 x 12 dimension lumber board below each chord centerline and spanning flat-wise between the abutment and intermediate pile bent. A wood shelf would be mounted to the piles at each end of the span, and the ends of the 2 x 12 board then nailed to them (Figure B.4) The string potentiometer bases would then be mounted to this 2 x 12 board (Figure B.5). This delayed the work as the additional materials had to be obtained that evening and instrumentation was put in place the next day. It also introduced concerns of the stability of the 2 x 12 during the load testing process. In addition, due to various beam-to-cap attachments present on the sides of the caps and piles, it was not possible to position cascades all the way at the ends of the chords. Of necessity, they had to be placed approximately 8–9 inches away from the ends, introducing some degree of error in the desired vertical displacement measurements, due to the very slight flexural curvature that occurs (under loading) in the stringers at these short distances way from the ends.

To further complicate the unexpected field situation, the weather on the second day (March 16, 2010) was a steady rain the entire day. Thus, instrumentation set-up was significantly prolonged (lasting all day) and delayed the spike installation until the third day (March 17, 2010).

5.2.6 Spiking Materials

The spikes were a ¾-inch diameter pultruded glass fiber reinforced polymer rod obtained from the vendor Liberty Pultrusions. They were cut from standard 10-foot long rods into 16-inch long segments, the length was such that upon installation they would have approximately one inch of cover from the tip to the bottom surface of the stringer to prevent the adhesive from flowing out. The lead end was also ground to form a pencil point, which serves to reduce the tendency of scraping adhesive off the circumference of the tubular hole as the spike is driven into the member. The spikes are manufactured with a glossy finish. In order to improve the bond between the spikes and the epoxy, the spikes were lightly sanded to provide a rougher surface for the bonding of the epoxy. The epoxy used was a two-part epoxy manufactured by West Systems. The 105 epoxy (Figure B.6) was used with the 205 fast hardener (Figure B.7). The fast hardener was selected rather than the slow hardener (as had been used in the prior lab research setting) due to the cool damp conditions at the time of spiking and the desire to cure the epoxy more quickly so that a more fully developed bond would exist at the end of the curfew when the bridge would be subjected to significant train traffic. The epoxy was thickened with milled fiberglass (West System 403 Microfibers) so that the viscosity was increased to a consistency that is similar to honey. This reduced the tendency of the epoxy to flow out of cracks in the specimen.

5.2.7 Locations of Spiking

Considering the short lead time before the travel to the bridge site, the limited on-site time accessibility, and actual condition of the members, it was decided in advance that only stringers in span 5 (the far end approach span) would be utilized for spiking. Thus, a fixed supply of 66 spikes was deemed sufficient in number and fit in one small suitcase. All spikes were prepared at the CSU location and taken to the site as checked baggage. Spike locations were decided on site, so as to use them all but place them strategically. As a first priority, spikes were installed at locations where horizontal cracks were evident on the outside vertical surfaces of exterior stringers. Some were placed in most interior stringers that exhibited little or no obvious damage, as the side surfaces could not be seen and thus may or may not have had horizontal cracks. (Note: Inspection personnel indicated that when horizontally cracked external stringers are removed, it is not unusual to find the interior stringers had such cracks as well.) Two spikes were placed side by side at each chosen location. Within that location, the two spikes were staggered as much as possible (one to two inches) in the longitudinal direction of the stringer. No spikes were placed in the stringer with the odd diagonal crack observed in the site reconnaissance visit. The main reason to not spike that member was that the method is intended for horizontal cracks due to shear at mid-depth. In any event, at the lower end of the crack, spikes would not reach the crack, and as one moved farther along the spikes would only embed for short distances below the crack. It was behaving like a flexural crack when trains were seen passing over the bridge at that time. Thus, it was severely opening and closing a wide vertical gap, suggesting a bond could not be achieved if spikes were placed across it.

5.2.8 Installation and Curing of Spikes

Figures B.7 – B.15 show the spike installation process. Spiking was done during the morning of the third day, having been delayed by previous day's rain. Before the spiking was done, a load test was performed to obtain the base displacement readings. The rain also delayed the crews doing maintenance work on the nearby track during curfew periods. These crew members were needed to do the spike installation. In contrast to a planned slow, day-long, time-controlled pattern of alternating drilling holes and installing spikes, the crew worked on an expedited basis. Thus, significant deviations from the procedures used in the past laboratory studies occurred. All holes were drilled in gang crew operation and then all spikes were installed in like manner. Spikes were placed from the top of the stringer, in the space between adjacent cross ties. Each hole was stopped about 1–2 inches short of the bottom of the member to prevent the epoxy from flowing out. In contrast to the laboratory work, where a rubber blow hammer was used, the UPR crews only had steel hammers and a steel rod available in their tool kits. The holes were made in a single drilling pass in each hole, unlike the sequential process used in the laboratory research. (In the lab, due to constraints of equipment, the drilling process was done in three steps. First, a ½-inch diameter hole was drilled, followed by a ¾-inch diameter hole. Finally, a 25/32-inch diameter hole was drilled.) A 25/32-inch diameter drill bit was not common stock in the UPR drill sets so a 13/16-inch diameter drill bit was used, instead. The drilling and spike installation was completed in about a one- to two-hour time period.

After the spiking was completed, at about noon, the spikes cured until 4:00 PM. At 4:00 PM, the curfew period of that day ended and all backed up trains were allowed to pass over the bridge in the following hours. Consequently, cure time was about four hours before that occurrence and cure time continued during any additional later normal train traffic. The actual number of trains and their length and loads are unknown, but the typical traffic on this main line was up to 25 trains per day. Thus, numerous trains likely passed over the bridge throughout the later curing period.

5.2.9 Load Testing

5.2.9.1 Method of Loading

Due to the nearby ongoing maintenance and repair work on the Glidden train line and resulting periods of curfew, it was possible to conduct a controlled static load test. Thus, the intent was to utilize such an approach to examine the effectiveness of the shear spiking technique. The nearby work made it possible to obtain the use of a single train car mounted with a fixed crane, and of weight similar to a locomotive. Thus, the planned approach was to use the crane car to conduct a sequence of static load tests over time. At the time of the spiking, the crane car would be put in place in two set positions on the bridge span of interest (span 5) after instrumentation was in place but before spiking. The measured net mid-span displacement of each chord would serve as a base indication of measured stiffness of the chords. Subsequent to spiking and period of curing of the resin, the load testing would be repeated. The percent reduction in measured net mid-span displacement of each chord would constitute an indicator of the degree of effectiveness of the spiking process. Conceptually, the instrumentation and static load test (using the same crane car) could then be replicated some later time (perhaps a week or two) after the rejuvenated spans had been subjected to a significant number of in-service moving trains during that time interim. In the case of the Glidden track, that would be as many as 25 trains a day, or 175 trains per week, with up to 100 cars per train (up to 17,500 train cars) – moving at up to 50 mph. Thus, the effect of that volume of fast moving train, loading on the sustainability of the shear spiking adhesive could be assessed. By placing the instrumentation, repeating the same static load test, and measuring the net mid-span displacements of the chords, the resulting measured values could be compared with the prior results to get a percent change, increase or decrease, or possibly no percent change in measured stiffness. Note: As the same train car would be used in the same static position for all load tests, there is no need to know the actual axle loads of the car, as an absolute measured stiffness was not being sought.

5.2.9.2 Conduct of the Load Tests

The crane car was made available on the site for the morning (March 17, 2010) of spiking and utilized before the spiking was done. It was positioned in two locations. One position was with its front pair of axles centered on mid-span of bridge span 5 (Figure B.16). The other position was with its rear pair of axles centered on mid-span of bridge span 5 (Figure B.17). In the latter load position, the two front axles were within span 3, and considered to have a minor effect on the displacement of span 5. The axle loads were unknown but not needed. Subsequent to the spiking, the instrumentation and first load test were repeated, using the same crane car. This was done during a subsequent trip to the bridge site from March 31-April 1, 2010.

5.2.9.3 Results of the Load Tests

Measured data were processed by converting measured voltages into displacements in inches, based on calibration factors established in the laboratory before taking the string potentiometers to the bridge site. Then, by spreadsheet, the measured chord end displacements were subtracted from the measured chord mid-span displacement to yield the net mid-span displacement. This was done for both chords and for the before spiking and after spiking data. Unfortunately, many concerns arose from the data, causing a basis to consider the results unusable – and likely erroneous.

For net mid-span displacement, one chord deflected downward 0.34 inch before spiking and measured 0.38 inch upward (!) after spiking. The other chord displaced a measured 0.30 inch downward before spiking and 0.34 inch downward after spiking (all values rounded). This is illogical. So the direct data were examined.

Direct displacement values measured by the potentiometers at the ends of the chords and at mid-span before and after spiking were not much different – and some were clearly erroneous. Considering the front axle at mid-span loading position, the data (before spiking value vs. after spiking value) were as follows. There were decreases in displacement at the ends of one chord 0.42 inch vs. 0.38 inch, and 0.34 inch vs. 0.29 inch, all downward. But at mid-span, the values were 0.72 inch downward and 0.04 inch upward (!), respectively. For the other chord, there were slight increases in the displacement at the ends of the chord, 0.306 inch vs. 0.315 inch (equal if rounded) and 0.260 inch vs. 0.275 inch. For the rear axle at mid-span, the measured values were somewhat the same. At the ends of the chords (in the same order of information) the values were 0.40 inch vs. 0.39 inch, 0.38 inch vs. 0.34 inch, 0.29 inch vs. 0.31 inch and 0.30 inch vs. 0.295 inch (0.30 inch, rounded). At mid-span, the values were 0.73 inch vs. 0.03 inch upward (!) and 0.60 inch vs. 0.63 inch. As the front and rear axle loads were different but likely very similar, the consistency seems okay. However, on structural logic, these results are deemed undependable and in part, or entirely, erroneous. Of further concern, it was noticed from the recorded information that in their no load positions (before the crane car was moved on to the bridge), there were different extensions of the strings (distance they were advanced from the base on the 2 x 12 board to the hookup on the underside of the wood slat) in the reinstall for the second test, compared with the initial install for the first test – despite the 2 x 12 board presumably being in roughly the same position. The differences ranged between 0.10 inch and 3.8 inches. The former is possible, the latter is puzzling. All potentiometers had been numbered so as to reinstall them in the same positions, and all hanger pieces were marked for repeat of their locations as well.

5.3 Observations

The questionable, likely erroneous, displacement data indicate that physical instrumentation set-up issues must have occurred. Most likely the expedient step of mounting the potentiometers to the 2 x 12 board rather than to the stable cross-ties on the ground is the primary source of error, possibly moving as the static train load came on to the bridge. It also might be that one or more of the potentiometers malfunctioned, but surely not all. The potentiometer that recorded the upward displacement at mid-span of a chord was particularly suspect. However, its calibration was subsequently confirmed as unchanged.

In the odd chance that some of the data were correct, the effect of shear spiking would appear to have been only slight. If so, then the interruptive rapid passage of trains after the end of curfew on the day of spiking could have compromised the bond of the adhesive, which had only cured approximately four hours. In the absence of a curfew, a preferred 12-18 hours of curing would have occurred (under normal timing of periodic trains) if the spikes had been installed at the end of the preceding day. In addition, the vertical deflection of the one stringer with the odd crack was visibly opening widely when observed under passing trains in the reconnaissance visit and displacing dramatically more than the adjacent stringers in that chord. In that instance, having not been spiked, it would likely deflect much more than the other stringers in the static load test as well. If so, that effect is hidden in the automatic averaging achieved by the cascade placement of potentiometers, in effect skewing that average result compared with the displacements of the individual spiked stringers compared with that defective one.

Despite the above head scratching, the worst case would be one of the bonds having been interrupted and failing completing, so as to effectively have stringers with a set of holes in them. If so, the percent loss of cross-section in a single stringer would be 2(13/16)/7.5(100) = 12.2%, and that would only occur at isolated locations, not over the entire span. The proportion of span that would have that reduction would be no more than about (16 spikes; eight per end) times (13/16)/(15x12) (100) = 7.2%. So as rough logic, only 7.2% (0.122) = 0.8% increase in deflection would occur. The presence of the holes would have a very minor effect on the overall deflection of the stringer. Thus, the large increases in downward displacement cited above are indeed erroneous. The upward net displacements defy all logic.

Several days later, an assessment of the outcome and needed next steps was conducted by the CSU researchers and Gawronski via phone conference. All agreed that the results were questionable and thus undependable. At least initially, field observations by UPR field personnel suggested that only minor effectiveness on the overall measured stiffness of the bridge was evident. It was deduced that the most likely cause was loss of bond during the passage of the backed up trains due to curfew on the day of spiking. With about four hours cure time, the rapid, heavy traffic on this mainline track could have interrupted or destroyed he bonding that had developed to that point. Gawronski expressed optimism about the shear spiking technique and a desire to do further studies.

As the stringers of the bridge were scheduled for near-term removal, a mutual decision was made that the best course of action would be to remove the stringers as scheduled (for the entire bridge) and ship those that had been spiked to CSU for examination. Some could be retested in the laboratory and then additional spikes added to see if the new spikes were more effective than apparent for those installed in the field. With the field situation for spiking and the heavy train loads being dramatically different than the laboratory research conditions, this would be very beneficial. Subsequently, the members could be carefully cut at some spike locations to expose the field spikes and the laboratory spikes to examine the bond conditions. However, after several months had passed, Gawronski informed the team that a decision had been made to keep the stringers in place in the bridge, in part due to the qualitative observation that the spiked span appeared to have improved performance. Thus, he preferred that a second bridge be located for a second trial of the shear spiking technique.

5.4 Modifications Needed

Subsequent to the load testing of the above Bridge No. 1, several significant desired changes in approach were identified. The main desired needs were seen as:

1. A bridge site with a shorter ground travel distance from the airport than the two hours involved for Eagle Lake
2. An instrumentation set-up that is quicker to install and eliminates the need to measure from the ground
3. Ability to individually measure displacements of all stringers in a chord
4. Data acquisition capability that works for in-service moving trains
5. A means to determine the loads of passing trains

6. A means to determine the position of a locomotive or train car on the bridge at times coincident with the instant of collecting electronic data

5.5 Bridge No. 2 - TOYAH SUB 534.35

An attempt was made to locate a bridge site closer to the Houston airport and on a line other than a main line and with less intensive train traffic than the Glidden line. On August 19-20, 2010, two of the PIs made a visit to the Houston area to examine the Eagle Lake bridge site and reassess the site conditions, data acquisition and instrumentation methods, and load testing methods used for that bridge, for the purposes of later making adjustments for the next bridge test. Some consideration was given to retesting the spiked span, but the logistics challenges encountered in the first load test, lack of any curfew periods and costs, obviated that option. Reconnaissance was then done on a large number of open-deck timber bridges on various short lines and other less traffic lines. Kevin Louis of the UPR escorted the PIs to numerous sites within the closer environs of Houston that exhibited a variety of number of spans, existing member conditions, accessibility (from roadways and from the ground below), daily train traffic etc. While several were seen as having one or more desirable characteristics, none was found that satisfactorily provided all aspects. However, Louis gained extensive insight into the conditions that the researchers were seeking.

Following that visit, the PIs reevaluated all aspects of the load test of Bridge No. 1 and postulated adjustments and alternative approaches in order to more readily work within the challenging environment of conducting shear spiking and field load testing of a bridge on an in-service line. In a conference call with Gawronski and Louis, positive changes to the spiking and load testing methodologies involved were recommended (described below) and accepted. In lieu of limiting the bridge location to the Houston area specifically, it was decided that the entire inventory of the Southern Region would be considered and that Louis would locate a bridge via a combination of examining bridge inspection reports, communication with local bridge maintenance managers, and viewing sites within his daily travel within the region.
In April 2011, a highly promising candidate bridge was located; specifically the TOYAH SUB 534.35 bridge located in Stanton, Texas (in the vicinity of Midland, Texas), was suggested as a good candidate for the project. Although located along a main line track, its daily traffic was about 10-12 trains, about half that of the Glidden line. It was also located about 20 miles from the Midland airport, dramatically reducing ground travel time to the bridge site, as compared with the GLIDDEN SUB 69.11 bridge site. The bridge was scheduled for replacement of all its chord members in the near term, July or August. Thus, a tentative plan to investigate the effectiveness of shear spiking was targeted for June 2011, including load testing under actual train loads. Photographs from the site of the TOYAH SUB 534.35 bridge are included in Appendix C.

5.5.1 Geometry

A complete inspection report (for February 16, 2011), (see Appendix A) was accessed and gives all needed technical details. The bridge is on a main line track used only for freight trains and rated for 70 mph train velocity. It was built in 1962 and had not had any significant rehabilitation to the chords since that time. However, the cross ties had been replaced recently. The bridge is straight (no curve) and passes over a small waterway and is directly adjacent to U.S. Highway 20 Business bypass, thus accessible via the land immediately adjacent to the site. It is a three-span, open deck, timber trestle bridge with a total nominal length of 42 feet (actual length is 42 feet, 2 inches) (Figure C.1). All spans are essentially equal at a nominal length of 14 feet. The two chords comprise four-stringer closely packed stringers. The cross ties are timber and rated as being in very good condition, given they were essentially new. The steel rail is a 136-lb continuously welded section. The substructure is timber piling and caps at the abutments and pier bents. The underside of the superstructure is reachable for work without the need of ladders. Walkways provide access to the top of the bridge.

Nominal component dimensions are included in the inspection report. The main elements are: Piling – 13.0-inch diameter, Caps 13.5 inches x 13.5 inches x 14.0 feet, Stringers 8.00 inches x 15.25 inches, Cross ties 8.00 inches x 10.00 inches x 10.00 feet and totaled 45 in number for the length of the bridge.

Based on the inspection report, a variety of conditions existed for the stringers. In this bridge, six stringers had been coded as 4A. Also, all interior stringers were coded with an oval ("O"), signifying its condition was satisfactory. However, in the inspection report it was indicated that the third span exhibited a state in which none of the stringer spans had the 4A code. Thus, no spiking was anticipated for that span.

5.5.2 Reconnaissance Visit

On May 17-18, 2011, the CSU research team, escorted by Mr. Louis, made a reconnaissance visit to the bridge site. The parties arrived at the Midland Airport and traveled to the site by rental car, arriving about midafternoon. It was a dry day and the temperature was in the 90-95 (F) degree range, amidst a long period of drought. No water flow existed under the bridge. All three spans were readily accessible laterally and vertically without need of a ladder from the ground below. However, each end span had significant inclines of rock material at the abutment, coming very close to the bottom of the chord members, making access to those end points extremely tight. Because of that situation and another tight schedule being in place, it was preliminarily decided, that at most, two spans would be involved in the investigation. Because visible damage to exterior chord members was most evident in spans 1 and 2, the actual condition of the members in those spans was documented. Photographs were taken, accompanying sketches drawn, and pertinent notes recorded.

The instrumentation configuration and method of load testing of the bridge to be conducted in subsequent visits to the site are described below. On this reconnaissance visit, the CSU team transported its data acquisition and instrumentation equipment as checked baggage. Materials and tools needed to install them were purchased locally and left on site for subsequent needs. A trial load test was conducted using an expedient simulation of the planned instrumentation method, by which measurements were to be made of the mid-span vertical displacement of each stringer of each chord at mid-span, measured relative to the displaced ends of the stringer. Displacements of the stringers in one chord of span 2 were recorded for several actual trains that happened to pass over the bridge. The intent was to confirm that the instrumentation performed correctly and could successfully record a stream of data as the train cars moved over the bridge. The outcome was that the data acquisition rate used was sufficient to capture the variation of the response and the peaks of displacement, but adjustments would be needed to reduce the electronic noise to a sufficiently low level.

5.5.3 Actual Condition of the Stringers

As noted above, only spans 1 and 2 of the bridge were of interest to the research team as potentially to be spiked. As indicated in the inspection report (Appendix A), stringers 1-8 (again, left to right in the inspection report) in span 1 (top span in the coding diagram) were coded as 2A, 2V, 5A, 4A, 4A, O, 2B, and in span 2 (second span down in the coding diagram), they were coded as 4A, 5V, 2V, 4A, 4A, O, O, and 5A, respectively. The observed condition of these stringers in the field was as follows.

Stringer Line 1

In span 1, (Figure C.2 – Figure C.8) the member had experienced a failure, albeit remaining in place tied to the adjacent stringers. There was a severe crack extending from the bottom surface of the member adjacent to the abutment. At that end, an L-shaped steel member was vertically attached to the stringer and then connected to the pile cap of the abutment; the crack extended diagonally upward from that location in a wavy pattern and terminating at mid-depth about two feet from the other end of the stringer, essentially above the outer face of the pile cap at that intermediate pier. A large spike knot existed in the bottom of the member at about one-quarter span from the abutment. A large edge knot existed at the bottom edge of the member at about mid-span. It is odd that a flexure-like crack would initiate from the bottom surface near the end of a member, where low bending moments exist. More likely the restraint of the L-shape bolting caused the wood in between to split due to thermal expansion effects stretching the L-shape. Or possibly, the knots were the location of the initiation of the crack.

In span 2, (Figure C.9) there were two horizontal cracks in the lower part of the member, emanating from an L-shaped steel member connected to the stringer and cap of the first intermediate pier, i.e., at near end of the stringer. The lower crack was about one inch above the bottom of the member, very wide at the abutment end, and narrowing as it extended about two feet along the member where it appeared to terminate. The upper crack was about two inches higher, hairline in width and extended a few feet more into the span. As both cracks were between the lower two bolts of the L-shaped connection, they very likely were caused by thermal expansion of the L-shape. However (Figure C.10), there was also a horizontal hairline crack at mid-depth in the middle portion of the span, which was not noticed in the field. A closer look at the photo of the end of the member shows a very fine hairline crack at mid-depth as well. It appears the crack extended from that point all along the member, thus possibly constituting a shear crack. However it was not possible to determine if it entirely split the member. Alternatively, the crack could also have been the outside opening of a seasoning check but it is unlikely that it would be straight and extend the full length of the member.

Stringer Line 4:

A long diagonal crack (about one-quarter inch wide) extended from just above mid-depth of the stringer in span 1 to just below mid-depth of span 2 of the stringer in span 2. It was relatively flat and began about 9.5 feet from the far end of span 1 to about 5.5 feet from the near end of span 2 (Figures C.11 and C.12). Thus, at 15 feet in length, it was equal to the length of an entire stringer. Figures C.13 and C.14 appear to indicate a vertical gap extending through the full depth of the member above the pier. At first look this suggests either butted ends of two single span stringers exist or that a two-span continuous stringer had fractured into two spans. However, by zooming in on the photos, it appears that most of the apparent gap is the result of the buildup of a line of mud over dripped creosote that casts a shadow on the member. It also reveals that there is continuous wood fiber in some locations along the depth at that location. Some frayed wood is evident locally in an area directly above the horizontal crack. Also, the apparent vertical crack is near the edge of the cap width, not at its center. If, in fact, there are two separate stringers with butted ends, not a crack, the opening would be at or very near the center of the cap width. Also, at the pier, the horizontal (slightly diagonal) crack emanates from the same vertical location into each stringer. Overall, these observations appear to indicate a continuous two-span member to be the case and that some longitudinal tensile fracture occurred in addition to the horizontal shear crack.

Stringer Line 5:

(Figure C.15 and Figure C.16) Similar to stringer 4, stringer 5 had a horizontal crack (about 1/8 inch wide) extending from above the intermediate pier about six feet into the far end of span 1 and about 12 feet into the near end of span 2 (Figures C.17 and C.18). However, this crack was essentially flat and about two-thirds of the depth above the bottom of the stringer. Also similar to stringer 4, (Figures C.19 and C.20) there appeared to be a vertical crack through the depth of the stringer above the intermediate pier and a horizontal crack emanated in each direction at the same level above the bottom of the stringer. However, zooming in for a closer look shows this to be two stringer ends butted to each other. A second, somewhat horizontal crack was evident in span 1 (Figures C.21–C.23). It emanated at the near end (at the abutment end) of the member, about mid-depth. The crack was horizontal for a few feet then curved downward near the bottom of the stringer and extended horizontally for several more feet, ending about 12 feet from the abutment end.

Stringer Line 8:

Despite a coding of 4A, stringer 8 in span 1 had no visible damage (Figure C.24). There was no evident damage in span 2, either (no photo).

Stringers Lines 2, 3, 6, and 7:

These are the interior stringers of the chords, thus, their side faces were not visible. With one exception, there was no evident damage in the bottom surfaces of any interior stringers. Stringer 3 had been visibly compressed (but not frayed) in bearing above the pier between span 1 and span 4 causing that stringer to deform into a position below the adjacent other stringers in that chord (Figures C.25 and C.26). It is not known if there was a two-span continuous stringer crossing the cap at that point or if two stringer ends were butted to each other.

5.5.4 Timing of the Spiking and Load Testing

On May 18, 2011, immediately following the reconnaissance of the bridge, Louis traveled with the research team to CSU for two days of training about the shear spiking technique, adhesive types and curing time vis-a-vis gain in bond, and to observe a simulated load test in a laboratory setting and to assist in planning the field study. Plans to simplify the instrumentation and improve and speed up the data acquisition process were shared. Aspects of how to ascertain train timings and loads were brainstormed. Louis indicated he thought it might be possible to obtain train loads in real time via radio link to the dispatcher in Omaha, Nebraska. Planning was then done for conducting the field project. A determination was made that up to two spans would be spiked and spikes would again be taken by air travel baggage, some of which he would take on his return flight to Houston. Louis indicated two personnel would be utilized on site and they would travel to the site by company truck, bringing all power equipment, tools, and materials needed for the preliminary repairs, as well as the adhesive materials for spiking as they could be acquired in the Houston area. A pair of 25/32-inch diameter drill bits would be purchased and shipped in Houston as well. Once on site, the UPR personnel would perform the preliminary repairs and install the spikes. The UPR personnel would also use computer monitoring of the Toyah line and communication with the dispatcher to monitor oncoming trains and control safety as trains neared and passed over the bridge. Louis agreed to further investigate how actual train loads might be determined.

As the existing bridge was scheduled for near-term stringer replacement, an accelerated schedule for the field research study was preferred by the UPR. It was estimated that about two to three days of time could be made available for the field activity, as a matter of practicality and cost as much as work needs on site. The dates of June 7-10, 2011, were mutually set for the field study. The researchers would travel by air bringing the data acquisition equipment, instrumentation, and remaining needed spikes as checked baggage.

5.5.5 Instrumentation and Data Acquisition

The test set-up was significantly modified in several ways from the method used in Bridge No.1. Figure C.27 shows the improved test setup under the bridge. The concept of measuring displacements relative to the ground and the subsequent expedient of installing a support board between the end pile bents were abandoned. Instead, the test setup for this bridge consisted of a single string potentiometer attached at mid-span to the underside of each of the stringers in the bridge (Figures C.28 and C.29). The need to measure the end displacements of the stringer (due to substructure deformation and gaps etc.) was also avoided as follows. For each stringer, the mid-span string potentiometer was mounted to a nominal 2 x 6 board (oriented edge-wise, i.e., edge faces at top and bottom) that was suspended from the ends of the stringer. At each end of the board, a pair of metal straps (one per side face of the 2 x 6 board) was connected to the board by a single through bolt and nut. The other end of each strap was likewise through bolted to a small wood block that was screwed to the underside of the stringer. This approach resulted in the single string potentiometer recording the mid-span stinger displacement relative to the displaced ends of the stringer. In other words, if the ends of the stringer move vertically, then that movement is excluded from the mid-span measurement. Note that the metal straps were hung very close to the pile caps, eliminating the offset issue present in using end string potentiometers in the instrumentation of Bridge No. 1. The metal straps were also slightly inclined from the vertical. This served to allow the use of an uncut 2 x 6 board and also leave a clearance from the ends of the board and the pile bents. The nuts of the single bolts at top and bottom of the strap were tightened to maintain the horizontal position of the ends as much as possible.

The string potentiometers function by outputting a voltage that is proportional to the extension of the string. They were fixed to the top of the 2 x 6 boards attached to each stringer in the center span of the bridge. The string was then hooked to an eye screw on the underside of the stringers at mid-span. The string potentiometers were attached to the computer via a National Instruments USB-6009 DAQ (NI-6009) ("data acquisition puck"), which provided the 5V excitation voltage for the string potentiometers (Figure C.30). The NI-6009 can collect eight analog channels simultaneously at a rate greater than 5,000 samples/second. All eight string potentiometers were then able to be connected to a single NI-6009, which in turn was connected to a laptop PC via a single USB cable. The laptop power supply generates the power to the NI-6009 that is responsible for the 5V string potentiometer excitation voltage. A separate power source of somewhat higher voltage was considered, but the convenience and stability of the 5V signal from the NI-6009 was deemed of greater merit than alternative external power supply options considered.

The NI-6009 replaced the CR-1000 data logger that had been used in previous testing due to the need to capture deflection data on eight channels, simultaneously, for trains travelling up to 60 mph. Previous testing on the GLIDDEN SUB 69.11 bridge had been performed with a static load. To estimate the required data acquisition rate, it was assumed that, based on the single-span length, and a train speed of 60 mph, any point on the train would cross the single span in less than 0.15 seconds. This would suggest that a data acquisition rate of 100 Hz would be only just sufficient as it would result in roughly a one-foot capture resolution, and could easily compromise the capture of mid-span maximum deflection. Since 100 Hz is the maximum sampling rate for multiple channels when using the CR-1000, the unit was replaced with the NI-6009. The NI-6009 was set to capture the data at a rate of 1,000 Hz, which is well below its maximum capture rate. At 1,000 Hz, the distance travelled between data points at 60 mph drops to approximately one inch, which was determined to be adequate to capture the maximum mid-span deflection.

The data from the string potentiometers were logged using the LabView Signal Express software. Signal Express was set to record data from all eight channels (one per string potentiometer) at a rate of 1000 Hz for a 30-second interval. Assuming that a passing train was traveling at 60 mph, this rate of data acquisition would correspond to one data point for approximately one inch of train travel. The 30-second interval was selected as the researchers were unsure at the time of planning what train cars would be of interest: the locomotive, empty cars, loaded cars etc. The 30 seconds of data provided flexibility in what data were used. Signal Express was manually triggered as the train approached the bridge. Data were logged as voltage, which was then able to be converted to the position of the string potentiometer in inches. The string potentiometers had a full extension of 10 inches and were attached to the bottom of the stringers near an extension of five inches. Thus, the pattern (over time) of mid-span vertical position of each of the stringers relative to their ends as the trains crossed was recorded.

5.5.6 Preliminary Repairs

Before any spikes were installed in the bridge, two preliminary repairs were performed by the UPR employees on site. One was felt necessary for bridge safety considerations. During the reconnaissance visit it was noticed that there was a significant gap between stringer 8 and the cap at the intermediate pier (Figure C.31 and Figure C.32). That gap opened and closed noticeably (perhaps an inch or more) as the trains crossed the bridge. An existing shim between the cap and the stringers had badly deteriorated and also deformed under pressure. That stringer also had a rectangular notch over the shimmed cap (Figure C.32). It is not known whether a notch was originally made during construction of the bridge, or if the bottom of the stringer had been crushed over time (or both). Nonetheless, a horizontal crack propagated outward from one corner of that notch. These conditions resulted in the originally two-span continuous stringer behaving as one single span (of double the length of one span) stringer whenever vertical motion was occurring at the cap location. Thus, Louis wisely decided to replace the shim. That task was completed at the outset of the next visit to the site. Figures C.33–C.35 show the new shim. Also, per the request of the researchers, there was one other repair made and was done following the installation of the shim. Due to the large inclined crack in stringer 4, that member experienced significant pumping action (opening and closing of the crack) as trains crossed the bridge. The researchers were concerned that this pumping would make it difficult for the epoxy to bond to spikes that were inserted in the area of the crack. Therefore, two check bolts were placed vertically through the stringer about one foot on either side of the pier between span 1 and span 2. They were tightened in order to close the gap and prevent the pumping while the epoxy cured. The check bolts were to remain in place after the spikes had been installed.

5.5.7 Spiking Materials

The shear spikes used were of the same diameter and material as used for Bridge No. 1, but of shorter length, approximately 14 inches, due to the smaller stringer depth in Bridge No. 2. The same epoxy and filler material were also used with the exception of the hardener. For this bridge, both the 205 fast hardener (used in the first bridge) and the 206 slow hardener (Figure C.36) were made available on site. In this case, the spiking was performed during time periods between trains crossing the bridge and there was no foreknowledge of how much time would elapse between the insertion of the spikes and the passage of the next train. Therefore, the fast hardener was again chosen to provide a faster cure and more assurance of a bond between the spikes and wood when the trains passed. However, the spiking was to be performed in very hot weather conditions and it was unknown if the fast hardener would cure too fast. Therefore, the slow hardener was available if it was observed that the fast hardener was not providing a sufficient pot life.

Table 5.1 provides a comparison of the two hardeners available for spiking. It should be noted that the cure time listed in the table are based on room temperature conditions. For the day of spiking, afternoon temperatures exceeded 100°F. A general rule of thumb is that the cure rate doubles for every 10°F above room temperature. Thus, the conditions had a large impact on the curing of the epoxy.

Table 5.5 Hardener Cure Times

Hardener	Cure Time (at room temp)		
	Pot Life (100g cupful)	Working Time (thin film)	Cure to Solid (thin film)
205 Fast Hardener	9-12 min	60-70 min	6-8 hrs
206 Slow Hardener	20-25 min	90-110 min	10-15 hr

Prior to the installation of the spikes, any significant or noticeable cracks visible on the outer surfaces of the stringers were filled with spray foam (urethane spray foam). Figures C.37, C.38, and C.39 show examples of this filling. This was done in order to reduce the amount of epoxy that may have flowed out of the stringers when the spikes were installed. Despite this, some epoxy did flow out of a few cracked locations on the sides or bottoms of stringers (Figure C.40). In one case, the drilled hole evidently penetrated the bottom of the member as droplets of epoxy were seen emanating from the hole (Figure C.41). As an expedient, cardboard or pieces of wood were scabbed on to stop the flow (Figure C.42).

5.5.8 Locations of Spiking

Span 2 (the center span) of the bridge was the focus of the research study. On that span were 11 spaces between the cross ties that could be utilized for spiking; however, two of these gaps contained beams supporting the walkway and thus were not used for spiking. Therefore, there were nine rows of spikes installed in the center span. A row of spikes consisted of two spikes placed across the width in each of the outer stringers in each chord (Figures C.43 – C.45). The two spikes were staggered slightly along the longitudinal direction of the member. Enough spikes were brought to the site to spike all four of the stringers in the chord; however, that was not done due to hesitation raised by the UPR employees on site. As the actual condition of the interior stringers was not known, it was felt holes might be drilled into members of good condition. The researchers concurred that spiking such members would be unnecessary. Also, in drilling the outer members, instances of sensing voids in the material were occurring, which caused the driller to hesitate about continuing to the interior members. However, such voids would be filled by the epoxy being placed as an intended feature of the spiking technique. Such voids would be evident by a need for additional epoxy, but that was not occurring. Nonetheless, it was decided that the spiking would cease with just the outer stringers spiked. It was surmised that the spiking of just the outer stringers would provide enough data to observe if the spiking was effective. A total of 71 of 72 locations were spiked. The first location was where a check bolt had been installed. Consideration was given to removing the check bolt and spiking that location. However, it was decided to leave it in place.

Initially, it was planned to spike span 1 of the two-span continuous stringer 4 (which had the significant shear crack). The intent of this would be to observe how much effect that spiking on the end span would have on the measured stiffness of the center span. It was decided to not do this so as to instead observe the difference between spiking a span and not spiking a span of the same two-span stringer.

5.5.9 Installation and Curing of Spikes

Spiking was done during the morning of June 8, 2011. There was no curfew for spiking as had been the case for the GLIDDEN SUB 69.11 site, so spiking was performed between the passages of in-service trains. The spiking was done by the two UPR employees who were on site using standard UPR safety practices. In the absence of a rubber mallet, a sledge hammer and wood post were used to pound the spikes into the epoxy-filled holes (Figures C.46 and C.47). The researchers were responsible for preparation of the epoxy and passing it to the UPR employees on the bridge. Table 5.2 provides a detailed chronological accounting of the timing of the spiking and the passage of various trains during the period of spike installation. Before spiking commenced, two trains passed over the bridge and then installation commenced. The tabulated information indicates when holes were drilled, cups of epoxy were mixed, and spikes and epoxy were placed. Times of passing of trains, their measured speed, and occasional notes about their nature are also noted in the table. The spiking began at 8:10 AM and was concluded at 1:49 PM, i.e., 5 hours and 39 minutes. During the spiking, six trains had approached and crossed the bridge, necessitating a ceasing of work and following all standard safety procedures. Two additional work stoppages occurred for other interruptions. Subtracting those periods of time (a total of 93 minutes), the cumulative time spent installing 71 spikes was 4 hours and 6 minutes, i.e., 3.5 minutes per spike. Table 5.2 also has recorded notes about subsequent trains passing later that day and on other days, when load testing was being conducted.

When the spiking began, the temperatures were around 80°F, and it was decided that the fast hardener would be used. However, by 1:00 PM, the temperature had risen enough that the epoxy with the fast hardener did not have sufficient pot life. Therefore, the switch was made to the slow hardener. The 72 spike locations consisted of nine rows of spikes with two spikes per stringer in each of the four outer stringers. However, one of these was the check bolt placed in stringer 4, which was left in place. Of the spikes, 55 were installed with the fast hardener and the final 16 spikes were installed with the slow hardener. At room temperature (68F), the epoxy with fast hardener is expected to take approximately six to eight hours to develop an initial cure, while, in contrast with the slow hardener the cure time would be approximately 20 hours. The decision to start with the fast hardener was based on the train traffic expected on the bridge. Since an extended curfew was not available during the spiking of this bridge (as had been the case with the GLIDDEN SUB 69.11), a fast cure was desired so that there would be significant bond shortly after spiking. Again, as a rule of thumb, these cure reactions double in rate with each 10°F increment in temperature. Thus, at the elevated temperatures observed during the TOYAH SUB 534.35 spike installation, it was predicted that the pot life of the epoxy would be just sufficient to install a group of spikes (about 10–15 minutes) and that initial cure would take between one–two hours. While such increased rates of cure generally reduce the maximum cured epoxy performance, the trade-off was deemed acceptable to ensure an advanced cure state between rail traffic. The change to the slow hardener was made when the ambient temperature increased to a point where the pot life with the fast hardener was reduced to a point that a group of spikes could not be inserted before the epoxy cure initiated. With ambient temperatures reaching greater than 100°F, the slow cure hardener still resulted in a high degree of cure within less than two hours and thus, in general, between rail traffic. At the completion of the field spiking, the epoxy had an average cure time of approximately three hours and, through visual inspection, demonstrated a significant degree of cure.

Table 5.6 Spiking Activities Record

Time	Stoppage	Train ID	Holes Drilled	Cups Mixed	Spikes Inserted	Spiking Notes	Train Speed	Train Notes
8-Jun-11								
738	TRAIN	MWCDA 06					30	long, sequential variety
747	TRAIN	ITIDI 06					55	modest # of cars
810			H1					
816			H8					
822				C1		dedicated track		
824			H9-H24		SI-S4			
826				C2	S5-S8			
830				C3	S9-S12	end dedicated track		
834				C4	S13-S16	16 filled, 8 left to fill		
909	TRAIN	MTUFW 07					30	lots of empty box
924			H25-H40	C5	S17-S20			
931				C6	S21-S24	24 filled, 8 left to fill		
INTERRUPTION								
939				C7	S25-S28			
944				C8	S29-S32	32 filled, 8 left to fill		
1009	TRAIN	ZLCMN2 07					59	long, single stack
1024				C9	S33-S36	leak in stringer 6 midspan		
1028				C10	S37-S40	40 holes filled, none empty		
1035	TRAIN	ZLAMQ2 07					67	
1041				C11	S41-S44			
1043			H41-H48					
1050				C12	S45-S48	48 filled, none empty		
1057	STOP							
1100			H49-H56					
1105				C13	S49-S52			
1108				C14	S53-S56			
1122	TRAIN	IMNLB 07					37	lots of box, then mostly 2 stack, some 1
1130			H57-H60					
1135	STOP							
LONG INTERRUPTION								
1201			H61-H64					
1300	TRAIN	LBD78 08					52	lots of oil, gas?
1315			H65-H72					
1317				C15	S57-S58	fast cure, fill 2 holes		
1328				C16	S59-S62			
1332				C17	S63-S66			
1334	TRAIN	STFXB 07					49	engine 4526, 2 eng., empty flats
1339				C18		lost cup in cure due to train		
1348				C19	S67-S70			
1349				C20	S71-S72			
	DONE							
1818	TRAIN	IDILB 8					42	
1843	TRAIN	ZAILC 08					52	
1923	TRAIN	ILXMN1 06					52	
9-Jun-11								
1130	TRAIN	ILBDI 08					30	
1142	TRAIN	MTUFW 08					11 to 15	gaining from 0
1141	TRAIN	IMNLB 08					18-20	gaining from 0
1315	TRAIN	ZLAMN 8					62	
1742	TRAIN	ZMQLC 09					52	
1805	TRAIN	MFWWC 09					49	
10-Jun-11								
915	TRAIN	MODFW 09					42?	
16-Jun-11								
1802	TRAIN	ZMQLC 16					20-22, 25-27?	89 cars, 65-70 tons per car 2 stack
1935	TRAIN	IBMN 16					50	
17-Jun-11								
849	TRAIN	ILXMN1 17					45	
920	TRAIN	MWCDA ??					15-16	gaining from zero
931	TRAIN	IDILB 16					24-25	gaining from zero

Note: Spike S1 was actually a check bolt put in the previous evening.

5.5.10 Load Testing

5.5.10.1 Method of Loading

The logistics of conducting a static load test on an in-service railroad bridge are extremely challenging. The opportunity to do such for Bridge No.1 was based on a curfew in place for scheduled track repairs and subsequent replacement of bridge stringers. More important, the daily curfew in place for a period of time allowed for an eight-hour window of time each day for the purpose of spiking and load testing. The static test option was used as it afforded a controlled application of load and was consistent with the load testing done in a laboratory setting, which was a ramp load up to 39 kips, which was a test load consistent with what the Association of American Railroads was considering as a possible new code provision. In essence the test procedure on Bridge No. 1 was a “shake down” test. By conducting a load test before spiking and a week or so later, the effect of daily train traffic over a period of time on the retention of adhesive bond, and thus spike effectiveness in rejuvenating horizontal shear resistance could be observed over the short term. Indeed, hypothetically, such a test could be conducted periodically starting soon after spiking to observe cure trend and then endurance. However, arranging the exact same load for both cases necessitates bringing in the same train car each time, which is impractical.

To load test the TOYAH SUB 534.35 bridge, significant changes in the approach to loading were incorporated. It was recognized that the most realistic approach was to utilize actual loads in service trains. However, this presented its own challenges. One challenge was to either know at what precise times that trains would arrive or to use triggers to remotely start and stop data acquisition only when trains were crossing the bridge. A second challenge was to know the actual loads of a given train that would pass over the bridge. A third challenge was the ability to effectively collect displacement data in quantity, quality, and fast enough acquisition rate to capture the real-time displacements response as trains moved across the bridge at velocities up to 70 mph.

Due to the UPR’s indicated intent to replace the stringers of this bridge in July or August 2011, the lead time before spiking and load testing was very short. Also, all on-site tasks had to be done amidst active daily train traffic. These necessary limitations, plus expenses, precluded devising and setting up trigger devices to detect and record displacement data for trains passing over the bridge. Thus, the data acquisition system described in Section 5.5.5 was used with a manual trigger, enabling the researchers to collect data for each train that crossed the bridge while the researchers were on site.

The data acquisition provided data that enabled researchers to determine the deflection of the stringers under the passing train loadings. Various sequences of train cars were involved. Following one or more locomotives was usually a stream of trains cars of the same type, e.g., a series of double stacked cars (Figure C.48), or a series of empty cars (Figure C.49), or a series of oil or gas tankers (no photo), etc. But sometimes an irregular mix of different type cars (e.g. some single stack cars, some double stack cars, some empty cars in no orderly pattern).The UPR computer-based train tracking programs provided a system that allowed the researchers to see various details of the trains as they approached, including the total load of the train, and the number of loaded cars and unloaded cars. Based on the train ID number, the UPR employees were also able to look up the approximate weights of the locomotive and train cars (measured at an unknown location). In general, the locomotive weights varied between 395 and 420 kips (one small locomotive weighing 276 kips also crossed the bridge).

5.5.10.2 Conduct of the Load Testing

There were many variables in the loading that needed to be considered; therefore, it was desirable to collect as much data as possible. The variables included the speed of the trains, the direction from which the trains approached, as well as the type of locomotives and differing nature of cars in the train. Numerous load tests were performed while the researchers were on site from June 8, 2011, to June 10, 2011. Prior to any spiking, data were collected from two trains crossing the bridge to serve as a baseline for the measured stiffness of the bridge. During the course of the spiking work, data were collected for each train that crossed the bridge. This was intended to provide an indication of the increasing measured stiffness as more spikes were placed in the stringers. After spiking was completed, additional data were taken at various times over the next two days as the epoxy cured. Whenever a train approached, a voice signal was given so that the researcher under the bridge with the data acquisition system could trigger the software. Data were then collected for 30 seconds.

The researchers returned to the bridge site for more load testing on June 16 and June 17, 2011. The intent of these load tests was to once again collect as much data as possible and also to observe the measured stiffness after a week of train loads crossing the bridge. All prior lab tests had been highly controlled, and the researchers hoped to gain insight into the durability of the spiking repair under the extreme loading conditions. Table 5.3 provides a detailed summary of the load tests performed. The table includes information such as date and time as well as the status of the spiking, the train ID with the lead locomotive model number, and the speed and direction of the train as it crossed the bridge. By mishap, for train 22, the suffix numbers for the train ID and locomotive model were not recorded and its train ID is suspect as well.

5.5.10.3 Data Collection

The speed of each train was measured using a handheld Bushnell speed gun that displayed speeds in mph, to a single decimal point beyond the whole number. The angle of incident relative to the train distorts the reading from the head-on velocity. Thus, the readings were taken from very near to the bridge site to the farthest of the approaching cars so to be at as flat an angle as physically possible and safely allowed, probably about 5-10 degrees (Figure C.50). The angle of the gun was laterally shifted as the train passed, so as to observe the slightly fluctuating readings. Readings were taken in that way for about 30 seconds so as to capture the maximum reading during the time of displacement measurements. Typically, the values did not fluctuate more than two–three mph during those 30 seconds and during the lateral shifting. One exception occurred. Trains 13 and 14 approached the bridge from a dead stop about one mile away. They had been pulled to a siding to allow train 12 to pass through from the opposite direction. Train 13 moving was gaining from zero as it reached a velocity of 11–15 mph as it passed over the bridge. Train 14 was gaining from zero as it reached a velocity of 18–20 mph as it passed over the bridge.

The stringer displacement data consisted of a 30-second graphical (and digital) record of the changing position of the members as the train crossed the bridge. Figure 5.2 provides the position data for all eight stringers of the center span of the bridge from a single load test. The start of the plot shows the unloaded position of the bridge. Then as the locomotive of the train comes onto span 1 of the bridge, the center span experiences some uplift; then as the first set of axles come onto the center span, the first valley of the data occurs. The minimum value in the valley corresponds to the condition that the first set of axles is centered on the center span (Figure C.51). At this point there are no other loads on any span of the bridge. Then as the train continues to cross the bridge (Figures C.52 and C.53), each subsequent valley in the data corresponds to each later truck of axles being centered on the center span.

Table 5.7 Load Test Locomotives

Train	Date & Time	Elaspsed Time (hr)	Rows of Holes	Rows of Spikes	Train ID	Locomotive Model	Speed (mph)	Direction
1	June 8, 7:48	0	-	-	MWCDA-06	SD9043A	30	Westbound
2	June 8, 7:57	0	-	-	ITIDI-06	SD70ACE	55	Westbound
3	June 8, 9:19	1.32	3	2	MTUFW-07	C45ACCTE	30	Westbound
4	June 8, 10:19	2.32	5	4	ZLCMN2-07	SD70M	59	Westbound
5	June 8, 10:36	2.6	5	5	ZLAMQ2-07	C45ACCT	67	Westbound
6	June 8, 11:23	3.38	7	7	IMNLB-07	C45ACCT	37	Westbound
7	June 8, 13:00	5	8	7	LBD78-08	GP15-1	52	Eastbound
8	June 8, 13:34	5.57	9	8.25	STXFB-07	SD70M	49	Westbound
9	June 8, 18:18	10.3	9	9	IDILB-08	C45ACCTE	42	Eastbound
10	June 8, 18:44	10.73	9	9	ZAILC-08	ES40DC	52	Eastbound
11	June 8, 19:25	11.42	9	9	ILXMN1-06	SD70M	52	Westbound
12	June 9, 11:31	27.52	9	9	ILBDI-08	C45ACCT	30	Eastbound
13	June 9, 11:43	27.72	9	9	MTUFW-08	SD70ACE	15	Westbound
14	June 9, 11:52	27.87	9	9	IMNLB-08	C45ACCT	20	Westbound
15	June 9, 13:15	29.25	9	9	ZLAMN-08	SD70ACE	62	Westbound
16	June 9, 17:54	33.9	9	9	ZMQLC-09	C45ACCT	52	Eastbound
17	June 9, 18:06	34.1	9	9	MFWWC-09	C45ACCT	49	Eastbound
18	June 10, 9:13	49.22	9	9	MODFW-09	SD70M	42	Westbound
19	June 16, 18:02	202.03	9	9	ZMQLC-16	C45ACCTE	22	Westbound
20	June 16, 19:35	203.58	9	9	ILBMN-16	C45ACCTE	50	Eastbound
21	June 17, 8:49	216.82	9	9	ILXMN1-17	SD70M	25	Eastbound
22	June 17, 9:20	217.33	9	9	MWCDA-??	SD70ACE??	16	Eastbound
23	June 17, 9:31	217.52	9	9	IDILB-16	C45ACCTE	25	Westbound

Figure 5.3 provides the deflection data from Stringer 1 in the bridge for Train #1 listed in Table 5.3. In this figure, there are six valleys at the beginning of the data that show deflection of the bridge under the load of the three locomotives that were on the train (three locomotives equal six sets of axles). The locomotives were followed by four unloaded cars and then seven loaded cars. The variation in deflection that occurs before the train arrives is an indicator of the electronic noise and perhaps other sources of inaccuracy (e.g., possible slight vibration of the bridge as the train approaches as well as vibration of the string potentiometer hanger set-up). Once the front truck of the locomotive moves on to the first span, the stringer deflects upward approximately 0.06 inch in span 2. That would be expected. Then, when the front truck moves on to span 2, that span begins to deflect downward, to a maximum deflection of 0.32 inches. Subsequent peaks in deflection constitute the effects of subsequent trucks of train axles moving on to, along, and then off the bridge.

Figure 5.4 shows another example of the data collected from a train. In this case, the data are for Stringer 1 from Train #17 in Table 5.3. This train was traveling significantly faster and had many more heavily loaded cars.

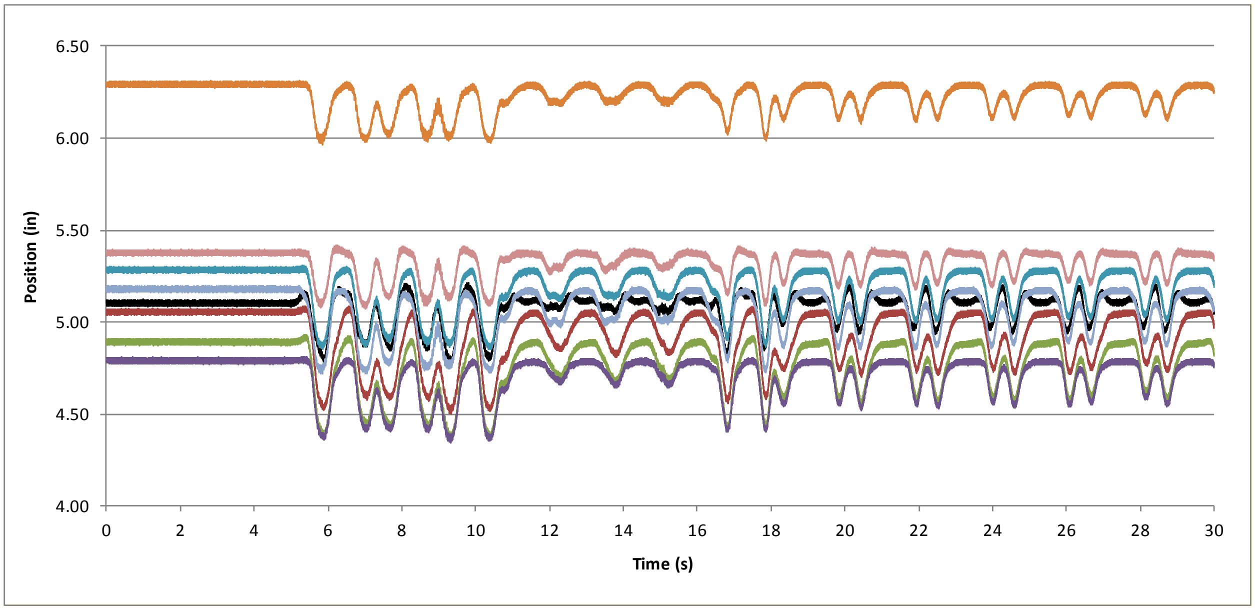

Figure 5.2 Data Record providing position of all at stringers during load test.

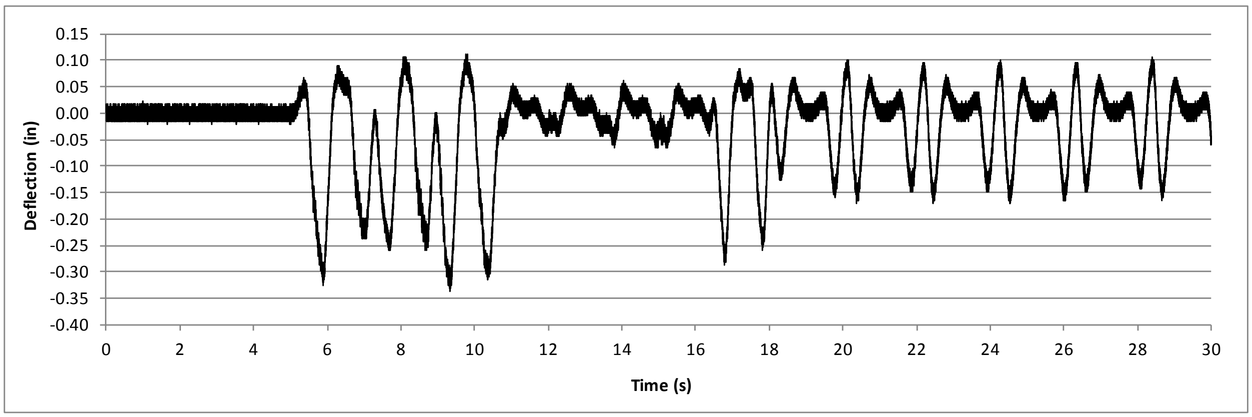

Figure 5.3 Position record for Stringer 1 under load of Train #1.

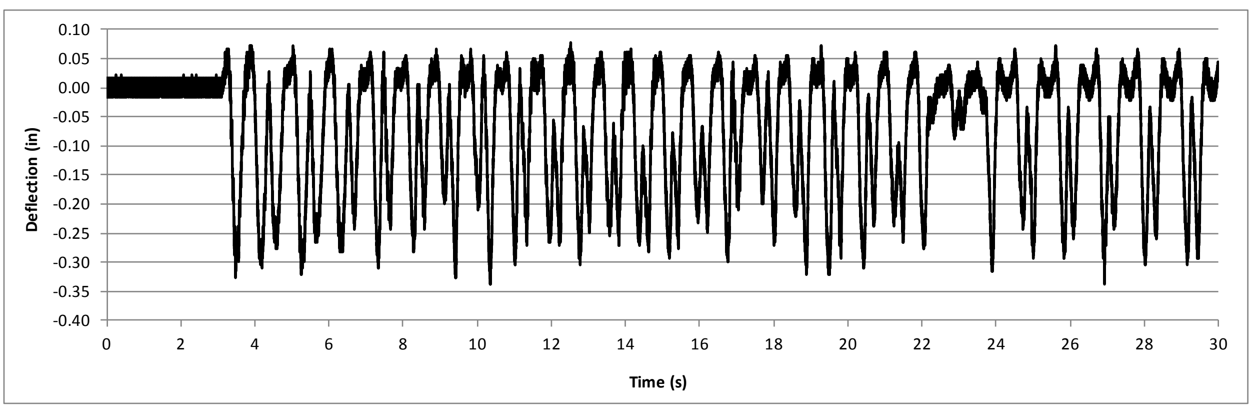

Figure 5.4 Position record for Stringer 1 under load of Train #17.

5.5.10.4 Results of Load Testing

The intent of the research study was to monitor the changes in flexural measured stiffness that resulted from the spiking process. Thus, it was necessary to determine the vertical displacement of the stringers under the load of the train. It was determined that the measured stiffness would be measured under the loading of the front truck (set of axles) of the lead (first) locomotive (some trains had additional locomotives being pulled at the front and/or back ends of the stream of cars) in the train. This was deemed to be the most convenient loading for measuring the stiffness, as it was at this time that there were no other loads acting on any point along the bridge that could affect the deflection. The deflection of the bridge stringers under this load was determined from the data collected. However, before the deflection could be determined, the issue of noise in the data needed to be addressed. Figure 5.5 shows the plot of deflection versus time for the stringer exemplified in Figure 5.3 but with it zoomed in to show the deflection under the first truck of the locomotive. Clearly, from the plot there is significant noise in the data.

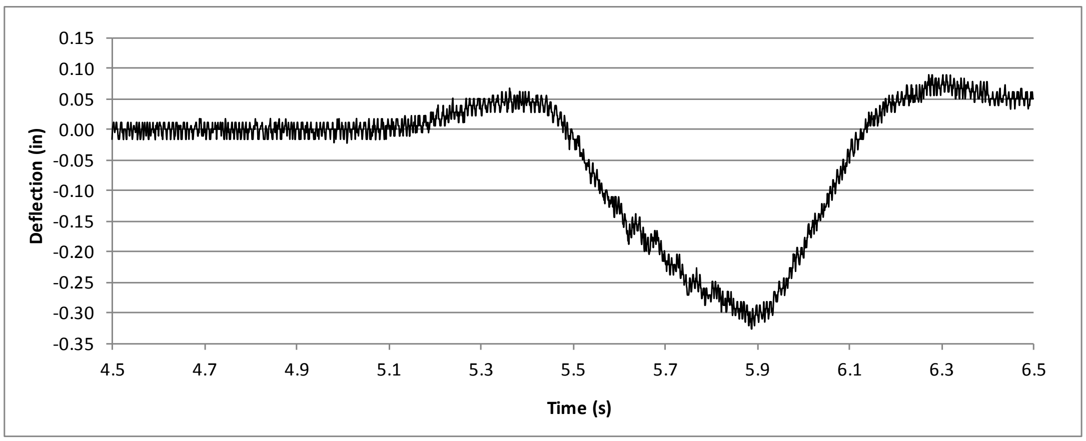

Figure 5.5 Position record of single stringer under the load of first truck of first locomotive for Train #1.

There are two primary sources of noise that could exist in the system. First, there is the electrical noise that may exist in the string potentiometer and data acquisition puck. There is also potential noise from the vibration of the test set-up. This would likely include the longitudinal and transverse vibration of the string potentiometer hangers. A Mathcad program was used to remove the effect of the noise from the data by using a Fourier Transform. Noise with frequencies between 10-499 Hz was removed from the data. It was determined that this range was sufficient as increasing it (above 499 Hz) did not remove any additional noise, and decreasing the range (below 10 Hz) began to remove some of the desired position measurement.

Figure 5.6 shows both the raw position data of the stringer exemplified above plotted atop the position data with the noise removed. Compared with the plot in Figure 5.5, the initial position of the potentiometer string is much more stable. The zero deflection position is taken from the "Noise Removed" data as the average of the data for all time before the train locomotive load is on the bridge. The deflection is calculated from the minimum value in the first valley of the deflection vs. time plotted data with the noise removed. The maximum deflection is determined from the minimum value in the first valley on the deflection vs. time plot.

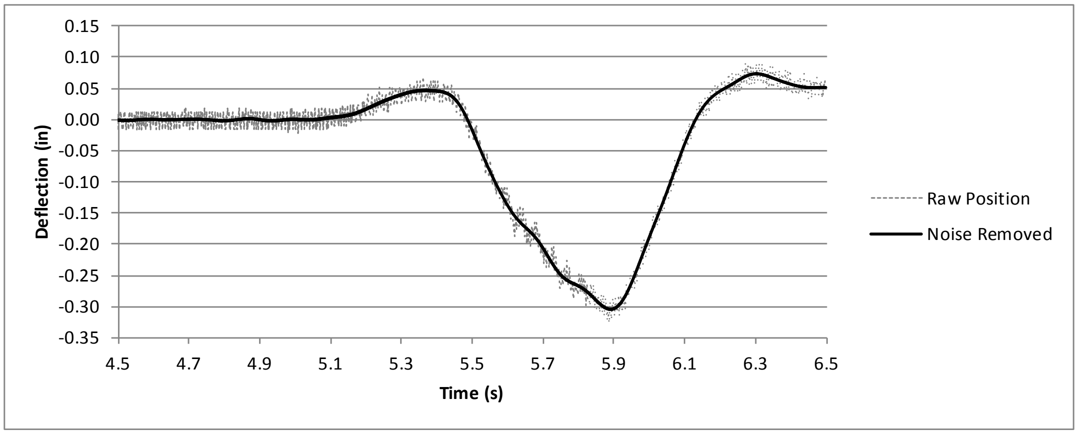

Figure 5.6 Noise removed from position data of record shown in Figure 5.3.

For the GLIDDEN SUB 69.11 bridge it was possible to observe the relative change in measured stiffness as the change in deflection before spiking and after spiking. In this case, that was not possible as the different locomotives were of different weights. Therefore, to be able to compare the measured stiffness of the stringers for the various trains, the load on the bridge also needed to be determined. As noted earlier, an approximate weight was available from the UPR; however, with the given information it was not possible to determine when or where the weight was measured, and thus, to estimate what fuel load loss would have occurred during its transit from that location to the bridge site. The fuel load onboard the locomotive at the time it crossed the bridge is part of the loading induced on the bridge. Therefore, there is appreciable uncertainty in the in the load given. However, in general, a fuel load is less than 10% of the total weight of the locomotive. The fuel load at the time of arrival at the bridge site would be lower by some unknown amount. In the subsequent analysis of the displacement results, the UPR's approximate weights of the locomotive were used and the fuel load was neglected. It was assumed that the weight of the lead locomotive was evenly distributed among all axles and equally shared between the rails. It was also assumed that the axle spacing was such that when the center axle in the first truck (comprised of three axles) was at mid-span of the center span of the bridge, the front and rear axles on that truck were very near the supports, though still on the center span. At this point, due to the long spacing between the front and rear trucks of the locomotive, there were no other axles on any span of the bridge.

Consequently, the "test load" on each span was calculated as 1/12 the total weight of the locomotive, which corresponds to the load being transferred by each axle to each chord of the bridge. The "test load" of one axle only is considered because it is centered at mid-span at the time of maximum deflection while the other two axles are very close to the supports. Most of the deflection is due to the load of the center axle. For analyzing the results, it is assumed the contribution of other two axle loads of the truck to the mid-span deflection is negligible. If the outer axles are over the piers then, indeed, only the center axle contributes to the measured displacement.

The measured stiffness of each stringer under the test load of each train was calculated as the applied load divided by the measured mid-span displacement. It was then possible to compare the measured stiffness values at different times and determine a percent increase (or decrease) in measured stiffness throughout the spiking process and the time following the spiking. It was found that when the individual stringers were considered separately, the results exhibited high variability, and it was difficult to distinguish a trend in the data. This was attributed to an unbalanced load sharing between the two chords on the bridge. If the train cars were either rocking or not loaded uniformly, the stringers under each of the rails could display inaccurate results based on the assumption of uniform load sharing. To account for the rocking, it was decided to combine the individual measured stiffness values of all eight stringers, yielding a net measured stiffness of the bridge (measured bridge stiffness).

Figure 5.7 shows a plot of percent increase in bridge measured bridge stiffness versus time. There is noticeable variability in the plotted results; however, possible contributing factors need to be considered. First, there were different types of locomotives on the trains that crossed the bridge. The various types of locomotives encountered likely have different axle spacing in their trucks. As stated above, to quantify a measured bridge stiffness, it was assumed that there was only one axle at mid-span of the center span and the front and rear axles of the front truck were very close to or directly above the supports of the bridge. For shorter spacing of the truck axles, the outer axles could be far enough into the center span to add a non-negligible measurable contribution to the mid-span displacement. Another factor is the direction in which the train was traveling. This manifests itself in two ways. First, there may be dynamic effects associated with the train crossing the bridge. As the condition of each end span was different, the dynamic effects would differ depending on which end span was in play as the locomotive came on to the bridge. Consequently, the observed deflection of the center span could be affected by the direction of movement of the locomotive. The direction of train travel could also affect the weight of the train. The provided weights were measured at some unknown location. It is possible that the fueling locations on each side of the bridge were significantly different distances from the bridge, which would have an effect on the actual weight of the locomotive when it reached the bridge. Another variable that could affect the measured bridge stiffness is the speed of the train. Some materials exhibit a dynamic impact effect, whereby larger deflections occur in beams for moving loads than for static loads of the same magnitude. In such cases, as a train moves across a bridge, the observed displacements are higher for any position of the train axles than would be observed if the train was standing still in that position. The dynamic impact effect is quantified by an "impact factor," which is used to magnify the static loads used in ordinary bridge design computations. Typically, as the rate of speed of a moving load is increased, so is the impact factor. It is generally accepted that wood has a negligible impact factor, meaning the speed should not affect the results of this study. However, it is possible that the impact factor of the steel rail affects the transmission of axle wheel loads down to the cross ties and, thus, to chords in some way.

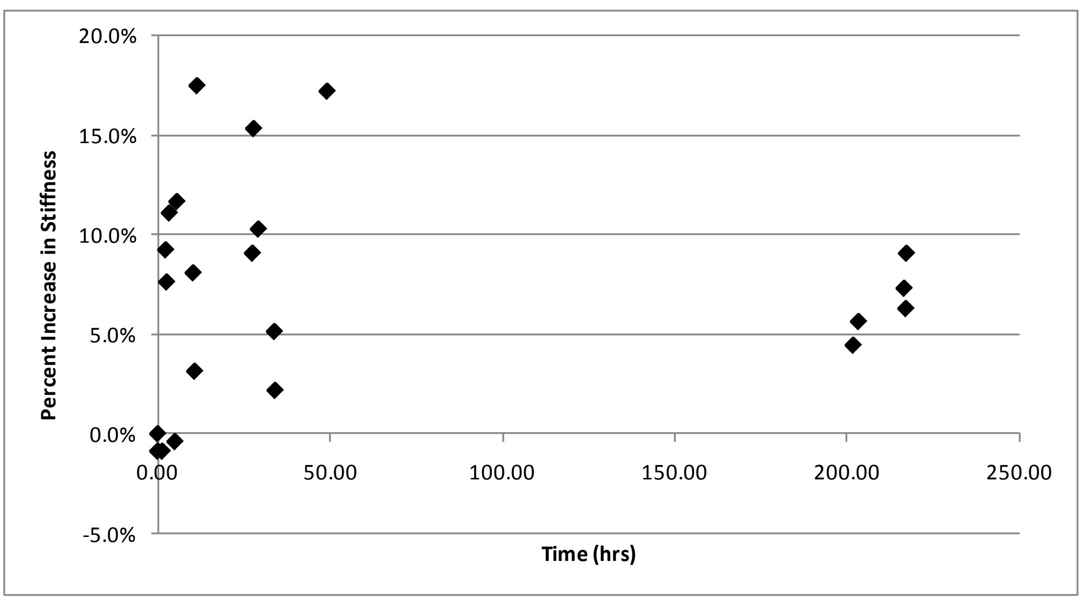

Figure 5.7 Percent increase in Measured Bridge stiffness vs. Time for all trains.

Due to the large amount of variability in the locomotive loading and the limited number of trains available for data collection, it was determined that averaging the data over limited time intervals would provide the most meaningful description of the results. Table 5.4 below provides a listing of the locomotives with their locomotive type, direction, speed, and the calculated increase (or decrease) in measured bridge stiffness relative to the baseline load test. It should be noted that in the table, there is no percent change in measured stiffness for trains 7 and 13. Train 7 is not included as it was a small locomotive that had only two axles on the truck; therefore, resulting in a significantly different loading of the bridge. Train 13 was not included due to an error in the data collection for that train, which resulted in missing data.

The averaging of the data was done as follows. Trains 1 and 2 were averaged, as both of those data points were before any spiking was done on the bridge. Trains 3–6 and 8 were not averaged at all, since these loadings took place during the spiking and the stiffness of the bridge was being influenced by the spiking process. After the spiking was completed, trains 9–11 were averaged, as those loadings all took place within a little over an hour. Next, trains 12 and 14–17 were averaged, as these loadings all took place within seven hours and would have occurred about one day after the spikes were installed. Lastly, trains 19–23 were averaged. These loadings took place approximately one week after the spiking was completed. At this time, the epoxy would be fully cured and there should be no changes in the stiffness of the bridge. Figure 5.8 provides the results of averaging the measured bridge stiffness. The observed changes in measured bridge stiffness over a one-week time period are plotted. During the period of the spike insertion, there was a steady increase in measured stiffness with a maximum of 12% greater than the initial value. The next two data points show a slightly lower stiffness over the course of the next 24 hours as the measured stiffness was approximately 10% higher than the initial value. Then, after a week, the measured stiffness had again decreased, but was still 7% higher than the initial value measured.

Table 5.8 Measured Bridge Stiffness Changes with Loading Details

Train	Date & Time	Elaspsed Time (hr)	Rows of Holes	Rows of Spikes	Train ID	Locomotive Model	Speed (mph)	Direction	Percent Change in Measured Stiffness
1	June 8, 7:48	0	-	-	MWCDA-06	SD9043A	30	Westbound	0.000%
2	June 8, 7:57	0	-	-	ITIDI-06	SD70ACE	55	Westbound	-0.874%
3	June 8, 9:19	1.32	3	2	MTUFW-07	C45ACCTE	30	Westbound	-0.877%
4	June 8, 10:19	2.32	5	4	ZLCMN2-07	SD70M	59	Westbound	9.261%
5	June 8, 10:36	2.6	5	5	ZLAMQ2-07	C45ACCT	67	Westbound	7.641%
6	June 8, 11:23	3.38	7	7	IMNLB-07	C45ACCT	37	Westbound	11.117%
7	June 8, 13:00	5	8	7	LBD78-08	GP15-1	52	Eastbound	-
8	June 8, 13:34	5.57	9	8.25	STXFB-07	SD70M	49	Westbound	11.690%
9	June 8, 18:18	10.3	9	9	IDILB-08	C45ACCTE	42	Eastbound	8.101%
10	June 8, 18:44	10.73	9	9	ZAILC-08	ES40DC	52	Eastbound	3.149%
11	June 8, 19:25	11.42	9	9	ILXMN1-06	SD70M	52	Westbound	17.525%
12	June 9, 11:31	27.52	9	9	ILBDI-08	C45ACCT	30	Eastbound	9.092%
13	June 9, 11:43	27.72	9	9	MTUFW-08	SD70ACE	15	Westbound	-
14	June 9, 11:52	27.87	9	9	IMNLB-08	C45ACCT	20	Westbound	15.365%
15	June 9, 13:15	29.25	9	9	ZLAMN-08	SD70ACE	62	Westbound	10.305%
16	June 9, 17:54	33.9	9	9	ZMQLC-09	C45ACCT	52	Eastbound	5.149%
17	June 9, 18:06	34.1	9	9	MFWWC-09	C45ACCT	49	Eastbound	2.185%
18	June 10, 9:13	49.22	9	9	MODFW-09	SD70M	42	Westbound	17.248%
19	June 16, 18:02	202.03	9	9	ZMQLC-16	C45ACCTE	22	Westbound	4.460%
20	June 16, 19:35	203.58	9	9	ILBMN-16	C45ACCTE	50	Eastbound	5.651%
21	June 17, 8:49	216.82	9	9	ILXMN1-17	SD70M	25	Eastbound	7.327%
22	June 17, 9:20	217.33	9	9	MWCDA-??	SD70ACE??	16	Eastbound	6.306%
23	June 17, 9:31	217.52	9	9	IDILB-16	C45ACCTE	25	Westbound	9.087%

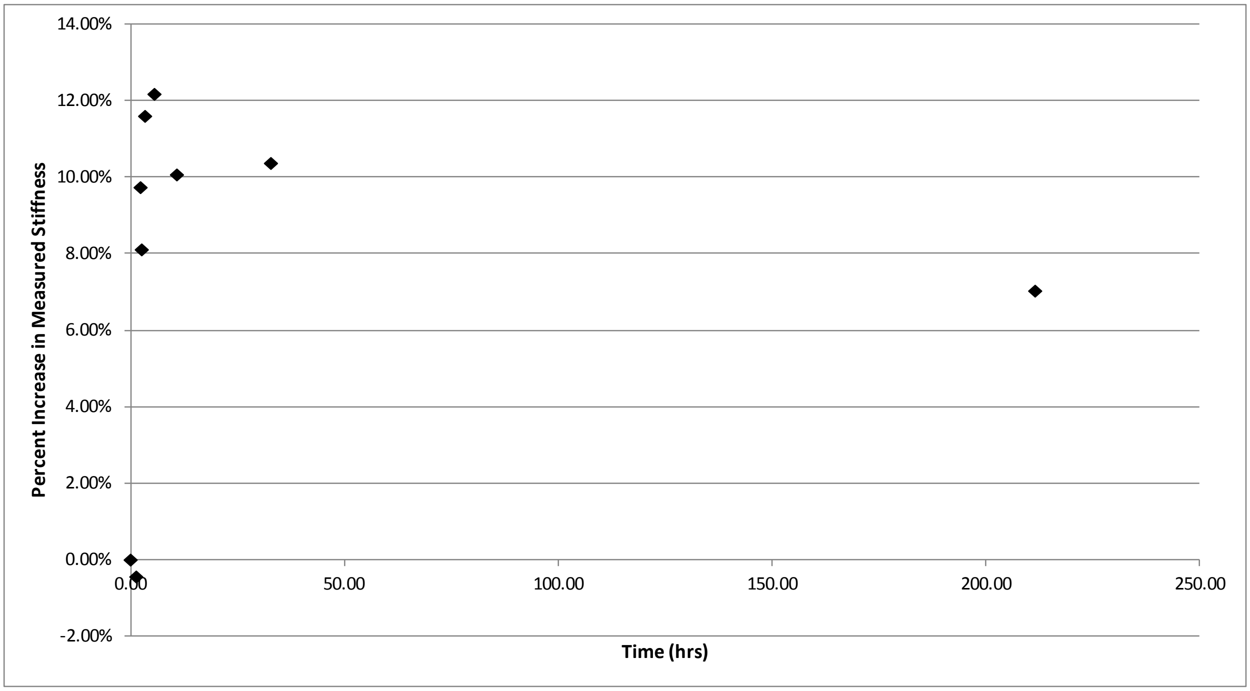

Figure 5. 8 Averaged percent increase in measured stiffness vs. time.

5.5.10.4 Observations

As detailed above, the spiking was completed on June 8, 2011, and load data were taken throughout that period and at various times after until mid-morning of June 10, 2011. For meal time and also overnight, the team left the bridge site. During those periods, additional trains passed over the bridge but were not documented as to number and make up of cars. Nevertheless, as the bridge was on a main line track, there had to be several each time and with a high number of cars involved for each. Thus, many train cars passed over the bridge during absences from the site. Despite that, in a general visual observation sense, the bond of the spiking had evidently occurred within about 4-6 hours after the last spike was placed and had been retained until leaving Midland on June 10. This was evidenced by the lack of any visible breakage in the lines of urethane spray foam sealer present along the sides of the stringers where it had been used to seal cracks.

Some hairline cracks, that were visible in stringer 5 in the initial reconnaissance visit and before spiking, were completed on the second visit were no longer evident the day after spiking. This may have been a result of or been contributed to by the installation of the check bolts that was done to restrain the shear crack in stringer 4, despite that stringer being in the other chord, as the members were adjacent, albeit separated by the chord spacing. However, under passing trains on June 10, 2011, the check bolt in span 1 was apparently no longer active. Visually, the extended long length of the shear crack within span 1 of stringer 4 was noticeably opening and closing repeatedly as train cars passed over, including at the location of the check bolt in that span. On the other hand, the crack in span 2 of stringer 4, which had been spiked, showed no apparent visible evidence of opening and closing.

Despite the installation of the additional shim board under stringer lines 5-8 at the pier between spans 1 and 2, stringer 5 was noticeably pumping at the pier between spans 1 and 2 during the test loadings. Indeed, subsequently it was seen in a photograph taken of the new shim (see Figure C.54), that one of the stringers in that chord was sagged below the level of the others. Apparently (similar to stringer 3 in the other chord) there was compressive deformation in bearing above the pier cap between span 1 and span 2, which caused that stringer line to deform into a position below the adjacent other stringers in that chord. It is not known if there was a two-span continuous stringer crossing the cap at that point or if two stringer ends were butted to each other. Also, which stringer is sagged is not discernible in the photo. However (unlike in sagged stringer 3 of the other chord), it is evident in the photograph of Figure C.54 that wood material fractured in the bottom of the displaced member as well. Perhaps these effects, having not previously been noticed in reconnaissance, were a result of the shim added above that cap and inherent lifting and resetting of the chord involved in that shimming process.

5.6 Overall Observations

Despite the outcome of apparently non-useable displacement data for the GLIDDEN SUB 69.11 bridge, the sequence of events experienced there provided considerable useful learning. Above all, the challenges of conducting a shear spiking installation and load test evaluation of effectiveness in the field compared with the laboratory were encountered. Among them are:

a) Field maintenance and repair work on bridges occurs in scheduled time periods, but crews face the "on-the-fly" nature of performing it. Crews must intersperse work stints between passing trains. The vastly predominant traffic is that of freight trains. Unlike passenger trains, which move along closely on schedules, arriving and departing at closely known and dependable times, freight trains move along at highly unpredictable timing. While train locations are always known in computer systems (similar to air traffic controllers), their travel time between any two points is unpredictable. Hence, there are procedures in place for various options for control of access to bridge work sites. Availability of these depends on the nature of the work, it varying between normal everyday maintenance to more or less immediate emergency repairs. In the usual routine, on a request basis, dedicated time intervals for control of a work site line can be sought prior to given work days, but may not be granted if higher priority need (e.g., traffic is too high to interrupt or another work site on the line has one approved, and an additional need is too interruptive) exists to deny a request. Such requests are not decided until the morning of the particular work day. Full days can be made available if a curfew is approved in advance, but these are much harder to arrange and for longer needs, such as repairs along a long line of track. Spontaneous requests for control of a track or bridge work site on an alert basis can be made via real-time request to a central dispatcher for the entire regional railroad system of the company (in this case, a dispatcher of the UPR – located in Omaha, Nebraska). In such a case, when the dispatcher sees no train traffic within a certain distance of the work, the dispatcher can alert and grant the crew chief a period of open access (perhaps an hour or two). But the spiking work was done on a intermittent in-and-out basis, whereby the crew jumped in between train arrivals and conducted a burst of work. Train locations within a long vicinity of the location can be tracked on computer, but nonetheless designated safety observers must be constantly on visual watch of the line.
b) Central supply rooms for tool and materials are not readily available. Tools travel with the work crews on train cars or in trucks. Any special needs that arise, can delay work for a period of time.
c) Only safety trained and qualified personnel are allowed on top of the bridge during any ongoing work. Beyond the placement of the instrumentation, preparation of the epoxy, and operating the data acquisitions system, the research tasks were in the hands of UPR personnel. Hence, the spiking procedure could not be either closely observed or guided by the research team members. The static load test involved the UPR staff controlling the placement of the test train. Nonetheless, the by-product of having to prepare and train UPR personnel about the in-depth aspects of the background development of the spiking technique, issues of cure time etc., is a positive step.

d) In the case of the GLIDDEN SUB 69.11 bridge, a gang crew cut all the holes and was able to complete that and all spiking before a curfew ended and trains passed over the bridge. About four hours of curing had occurred for the last spike placed. Earlier spikes cured longer, the first spike curing perhaps two hours longer. By contrast, in the laboratory, to incrementally install spikes and periodically do a load test to observe developing effective flexural stiffness took over several hours or even an entire day or two

In the case of the TOYAH SUB 534.35 bridge, dramatic changes were made in the conduct of the work and load test compared with procedures used for the GLIDDEN SUB 534.35 bridge. These were done from a desire to adjust to the everyday work environment of the railroad crews so as to facilitate that aspect as critical to the future use of the spiking technique as a routine repair technique. On the other hand, the load test approach was adjusted from different perspectives. Specifically,

a) Drilling and spiking was done by a team of two UPR personnel, minimizing the crew size but extending the time needed to about four hours compared with the one to two hours involved for the GLIDDEN SUB 69.11 bridge.
b) The holes were drilled laterally across the bridge, one eight-row line at a time and spiked before moving to the next line. As trains were in normal service, they arrived irregularly during drilling and spiking and after all spiking was done. Thus, up to eight empty holes could exist as a train passed over the bridge. Spiked holes might also have cured for a very short time period, and not yet have bonded in place. The presence of holes in a stringer reduces its flexural resistance. The calculations mentioned in Section 5.3 indicate that the loss in section due to the drilling of the holes has negligible effect on the deflection of the chords even if there is no bond between the spikes and timber members. However, assuming two holes laterally across the solid stringer width (size and depth used in the field), a high order structural analysis of a stringer done after the field work using 3-D finite element modeling showed the moment of inertia of the cross-section is reduced by about 17%, resulting in an increase in extreme fiber flexural stress of about 18%. For a two-chord, eight-stringer bridge, two holes in one stringer constitutes a 4.5% (18%/4) drop in the combined stringer flexural resistance. For two stringers with two holes in each, there is 9% drop. That increases to 13.5% and 18% for two holes in each of three and four stringers, respectively. With spikes in place and bonded there is no drop. Until sufficient bond occurs, even spiked holes are essentially the same as empty holes.
c) The load test aspect is needed for initial examination of the spiking technique in the field but would not be an aspect of spiking for maintenance and repair needs. Use of actual in-service trains avoided the need for the UPR to schedule curfew time on site and involve the accessing of a train car to serve as the load and also involve its crew.
d) Instrumentation performed very well, but involved verbal signal and manual triggering. This caused risk of triggering too late, which nearly occurred in one of the train passes.
e) Train car loads were not known. While the dispatch logged information provided a total gross weight being carried by the combined locomotives and cars, the actual axle loads were not known. Despite attempts to obtain them, no path to do so was established. Hence, only the locomotive type was known. While this allows some tracking as to its specifications, the fuel load was not known and could not be established for any of the locomotives.

5.7 Conclusions

The load test of the GLIDDEN SUB 69.11 bridge was flawed due to the field expedients done in the installation of instrumentation for data collection, so all measured results are suspect and the test is thus deemed inconclusive.

Based on the previous trials on the GLIDDEN SUB 69.11, the hardener used in the bonding epoxy for the TOYAH SUB 534.35 was changed to decrease the cure time. Between this change and the relatively high ambient temperatures (>100F), the resin developed a large degree of cure in the time between sequential trains. Visual inspection verified that a high degree of cure was developed in the time period between trains. This was felt to be an important consideration as there was some concern after the repair of the GLIDDEN SUB 69.11 that the large amount or train traffic prior to cure completion of the repair could have compromised the stiffness improvement. For the TOYAH SUB 534.35 there were clear and sequential gains in bridge stiffness as spikes were added and measurements taken shortly after spiking completion, suggesting that resin cure was sufficient to transfer the stresses planned. This result supports the visual inspection of the state of cure of the resin.

The short-term stiffness gain (within 24 hours of initiating the repairs) was significantly different for eastbound versus westbound traffic as seen in Table 5.4. The maximum short-term stiffness gain shown for eastbound traffic was approximately 9%, while for westbound traffic the maximum short-term stiffness increase approached 17.5%. No confirmed reasons for this disparity have been determined, as mentioned elsewhere in this report. Retesting 10 days later showed approximately 8% increase over the averages of span stiffness measured prior to shear spiking, with little variation that could be related to the direction of travel of the train traffic. Thus, over the 10-day period, the measured stiffness increase for the eastbound traffic remained essentially unchanged with time, but that for the westbound traffic dropped to half the highest measured stiffness. If the east and westbound data are averaged together, then the stiffness increase after 24 hours is approximately 10% and after 10 days is 7.5%. While this suggests a reduction in stiffness associated with the traffic over the 10-day period, the disparity between the stiffness response for eastbound versus westbound traffic makes it difficult to conclude any relationship between stiffness and service time. Thus, within the limitations of the current field test, it can be said that the shear spiking did increase the middle-span stiffness almost immediately and that a significant stiffness increase was still evident after 10 days of service. The observed outcomes concerning gains in measured bridge stiffness were for spiking only the exterior stingers (half the stringers) of each chord in the center span of the three-span bridge. The actual condition of the interior stringers was unknown. Assuming the extreme case that interior stringers were collectively in the same condition as the collective exterior stringers, the gains would be doubled had those stringers been spiked. For example, the short-term gain in average bridge stiffness would have been 20% (2 x 10%).

The evaluation of the TOYAH SUB 534.35 bridge involved the insertion of a pair of shear spikes between each set of cross-ties in the outer stringers of each chord of the middle span. This meant that 50% of the bridge stringers in the middle span were modified. Overall, it is clear that despite spiking only half the span timbers, the stiffness of the middle bridge span did increase, and that a stiffness improvement was retained over the two-week period of the investigation. Since the difference in stiffness between the bridge span at time of original installation and immediately prior to repair is unknown, it is difficult to surmise what fraction of the original span stiffness was achieved with the shear spiking. Thus, if additional field testing was to be performed, it would seem important to have both a better understanding of the loads that are crossing the bridge when deflection measurements are being made and have measurements over a longer period of time. Measurements over a longer period of time, both before repair and after, would improve the confidence in the measured stiffness and help improve the understanding of the long-term durability of such repair.

Before shimming, due to a vertical gap between it and the cap at mid span, stringer 8 was a single 30 foot member until the gap closed under loading. Adjacent stringer 7 was likely similar, so until the gaps were closed under load, these members would have behaved as if only half the length of those members was spiked. The consequence would be some decrease in observed gain in bridge stiffness, relative to no gap existing. Thus, shimming was a beneficial repair.

At face value, the observed gains in stiffness are similar to levels achieved when stringers are not damaged enough to split in half over their entire length (as evidenced above by reference to the past studies of Burgers et al. [3], and Gutkowski and Forsling [6]). However, there are important differences. In most of the laboratory research that preceded the field testing, individual stringers or isolated chords were tested and directly observed. In the field, the stringers of a bridge are just one component of the superstructure, thus not isolated. Wheel loads transmit through the steel rail, into the cross ties and then to the stringers. Stringers of a chord are physically constrained to deflect together due to the cross-ties above them. Thus, the differing physical states and dispersion of spiking of the individual stringers get diffused into the measurements of a highly statically indeterminate structural system. Thus, despite separate measurements of the individual stringers, the displacement data were already physically merged and only the measured overall bridge stiffness was examined. So comparison with laboratory results should be taken in that light. However, the internal fractured condition of the stringers was indeed not known for either exterior or interior stringers of the monitored chords. Only removal and examination of the stringers can determine their actual condition and extent of any lateral separations evident only on the outer surfaces (sides) observed for some exterior stringers and reveals the unknown state of the interior stringers. What is certain is that some exterior stringers had only minor damage, if any, and indeed had not split across their full width.

5.8 Recommendations

Short Term:

During the time of this study, it was anticipated that the stringers on both bridges would be removed subsequent to the spiking as a matter of planned replacement. Thus, in a sense, this was a pilot study of sacrificial stringers not a trial implementation into practice. Should those stringers be removed and provided to the researchers for laboratory investigations, several important steps could be taken.

a) Inspect the physical condition of the interior stringers versus that of the exterior stringers.
b) Retest the spiked stringers in the laboratory for stiffness levels.
c) Add additional spikes and observe any resulting gains in stiffness.
d) For some stringers, saw through spike locations to determine integrity of spikes and bond conditions. Compare observations for those done in the field versus those added in the laboratory.

Intermediate Term:

Should a third bridge become available for spiking research, it should reflect the following:
In choosing a follow-up bridge for further testing, it seems that the form of damage is much more important than frequency of rail traffic. As currently implemented, the shear spiking approach is primarily intended to rejuvenate stringers with 4A damage. At the same time, after the two field trials performed to date, a field process of shear spike insertion has evolved that allows relatively rapid preparation and application of the technology. After the first field trial, there was some concern related to the relatively slow rate at which the resin cured, in relation to the allowable frequency of rail traffic. However, the substantially increased rate of cure developed, through the change in curing agent and in ambient temperature resulted in measured stiffness increases within less than one hour of shear spike insertion. Thus, it seems possible to perform the rejuvenation by shear spiking even with relatively high rail traffic.

Of course, a low train traffic situation would likely allow increased cure time between the passages of trains in the critical early stages of curing should temperature conditions be much lower than experienced on the TOYAH SUB 534.35 site. It would also reduce the time interruptions during the spiking process, lessening the crew time involved. The latter would increase in importance as the number of spans and stringers to be spiked increased.

In applying the shear spiking technology to a third field test, some adjustments should be considered that may help address stiffness differences measured for rail traffic traveling in opposite directions. Since this difference may be related to the interaction of multiple spans due to dual span stringers, it is suggested that all suspect (4A) stringers of the bridge be shear spiked, even if stiffness measurements are only performed on a single span. In the second field test (TOYAH SUB 534.35), shear spikes were only inserted in the middle span, even though 4A conditions were noted in span 1 as well. As the middle span deflections were measured for the moving rail traffic, the different forms of damage in spans 1 and 3 could have led to span 2 stiffness values apparently being related, in some indeterminate way, to the direction of train travel.

The two investigations of spiking involved bridges made available on a "short fuse" basis. Thus, the window of time for conducting research tasks was limited leading to condensed approaches. A limited budget existed as well, on both sides. The UPR dedicated considerable personnel in number and in time – each time diverting them from normal workloads and other needs. A fixed budget for the CSU research team was in place, leading to tightly managed use of travel funds, personnel costs, materials, etc. Working within those limits, much was accomplished. In the ideal case of much more expanded resources (time and money), even more extensive and productive outcomes could be realized. Of prime consideration, the data acquisition technology used was able to accurately record spikes in deflection response but the loads were not known, only estimated. However, actual loads could be determined with more extensive instrumentation. By installing a load cell at some point along the track, the individual axle loads could be measured as the axles pass over that point. Also, on-off switches could be installed along the side of the track rails to detect the spacing of the axles, given the measured speed of the train. The use of switches would also serve to detect the times at which these axles pass over selected load points for the load test. This could be done over time during and after spiking was completed producing numerous axle load-deflection responses in real time as point events, but also collectively as an extensive set of load tests under moving train loads. After spiking, personnel need not be present if a means to determine train speeds is configured. This could simply be the first axle moving over two sensors set apart by a known distance, and when one is crossed, it starts a clock and the other stops the clock. Data could be collected for long time periods before and after spiking, to examine durability of the rejuvenation. Security of electronic equipment left on site would be a concern due to possible damage by weather, vandalism, or theft. In-place instrumentation (such as string potentiometers) could also be disturbed by human or other disruptions.

Comprehensive laboratory studies of the effect of repeated loading, at levels and repetitions consistent with typical field train load conditions, on bond and stiffness gains in deteriorated railroad bridge stringers could be conducted. Ramp load tests to failure of such specimens should be subsequently conducted on them and results compared to those observed for replicated specimens not subjected to repeated loading. Similar comparison should be made between those results and results for replicated new stringers, some of which are spiked and others which are not spiked. However, the conduct of the field program as described in the preceding paragraph would be more productive and directly meaningful.

In implementing spiking for bridges on in-service rail lines, the order of spiking should be to move along individual stringers one at a time, not laterally in a line across all stringers. If possible, to maximize cure time, spiking could be done incrementally, outer stringers on one day and interior stringers the next day.

Long Term:

Spiking even good-condition members likely provides a reinforcement effect that prevents shear failures and increases the longevity of a stringer. A comparative study could be performed in the field, in which a large sample of in-place stringers in acceptable to good condition is identified. Half the sample could then be spiked. Over time, via normal regularly conducted inspections, the longevity (evidenced by changing inspection ratings) could be compared with similar other members that are not spiked. If practical, more frequent inspections could be done on these members to more steadily and quickly observe the trend in each half sample.

There is strong evidence that shear spiking, as described in this report, can be successfully applied to field service of existing timber rail bridges with stringers compromised by type 4A damage. The field results clearly seem poorer than the laboratory test results, but this is not unexpected due to the greater variability of actual damage found in the field. During the scale-up from the laboratory, through the first field test to the second most recent field test, modifications have been made to the insertion process and to the resin cure schedule, which improve the potential to perform shear spiking without the need for substantial periods of bridge closure. Long-term performance of the shear spike repair has not been performed, but a substantial stiffness improvement is retained over the one week of evaluation on the TOYAH SUB 534.35.

There has been some discussion of the potential for this technique to be applied to new timber stringers as a method for improving service life by reducing the likelihood of damage initiation. While this may be effective, it has not been tested. However, during the repair of the TOYAH SUB 534.35 it was observed that the new replacement stringers were glued-laminated (glulam) beams rather than solid sawn timber beams. Since the shear resistance and durability of a glulam beam is only as good as the adhesive bond between its laminations, it may be found that the insertion of shear spikes in glulam beams prior to installation in the field offers a significant durability gain. There is independent work, focusing on glulam beams, which use a similar technique and has been shown to be very promising [9]. Thus, based on the preliminary field tests, it seems that increased duration field evaluation would be a logical next step for existing timber bridge stringers. It would also seem that durability/lifetime gains for glulam beams may be possible through the installation of shear spikes at the time of fabrication of the glulam beams.

Overall:

Presently, more research is needed as detailed above. Thus, a broad step toward applying shear spiking in practice is not yet advocated. In pursuing a path toward implementation, the approach should be as follows. Regular inspections of bridges lead to a bridge maintenance program based on prioritizing the implementation of repairs. Major repairs are scheduled for extended time, unless an emergency exists and immediacy is dictated by the situation. However, some repair needs discovered during an inspection or other site visits can lead to action at those times. Examples are the replacement of a shim and the installation of check bolts such as occurred in Bridge No. 2. When Type 4A shear cracks are observed and are of a serious and worrisome nature, shear spiking is not yet recommended; instead, it is perhaps best to replace the stringer. However, for more minor horizontal crack situations for which near- to intermediate-term stringer replacement is not yet warranted, shear spiking could be done as a stopgap measure to arrest the cracking and extend the life of the member. Spiking need only be done in the region(s) of the visible crack or cracks. Thus, trained individuals could even perform the work on the fly when the situation is first observed. On the other hand, the selective application of shear spiking to existing stringers of observed good condition could be done as the exploratory research described above to compare the life span of spiked (reinforced) vs. non-spiked stringers. Another alternative to conduct such a comparative investigation is to add spikes to replacement stringers before they are installed in a scheduled work time.

Thus, some newly installed stringers could be a mix of spiked and non-spiked stringers. Indeed a stock pile of such reinforced stringers could be prepared and maintained for such purposes. That would avoid the time constraint conditions of application of spiking directly to in-place stringers of an in-service bridge.

As only the exterior stringers of a chord are visible, the structural gain from spiking interior stringers is indeterminate. If flexural cracks are evident in the bottom surface of an interior stringer, then shear spiking would not help that condition. If bottom surface flexural cracks are not evident, then the shear spiking would either be on stringers that are either a) undamaged, b) partially damaged by horizontal shear cracks, c) fully split by horizontal shear cracks, or d) some other condition. Thus, shear spiking would either provide reinforcement to a good or partially cracked stringer or serve to achieve the intended direct repair of a horizontally split member. In any case, the life of the member would likely be extended. It is recommended that when a chord is replaced from a bridge, the condition of the exterior versus interior stringers be examined for evidence of possible commonality of damage. By keeping an ongoing record of these comparisons, that database might lead to a useful way to sense, hopefully predict, interior stringer condition from observed condition of exterior stringers.

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2. Burgers, T.A., R.M. Gutkowski, D.W. Radford & J. Balogh. “Composite Repair of Full-Scale Timber Bridge Chord Members through the Process of Shear Spiking,” Report No. 05-173, Mountain-Plains Consortium, North Dakota State University, Fargo, ND; December 2005.

3. Burgers, T.A., R.M. Gutkowski, D.W. Radford & J. Balogh. “Composite Repair of Full-Scale Timber Bridge Chord Members through the Process of Shear Spiking,” Report No. 05-173, Mountain-Plains Consortium, North Dakota State University, Fargo, ND; December 2005.

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APPENDIX A: BRIDGE INSPECTION REPORTS

GLIDDEN SUB 69.11

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GLIDDEN SUB: Data last refreshed on 11/5/2009 8:34:04 PM.

B>>> 69.11(1)   [SIMN] EAGLE LAKE   (COLORADO, TX)   OVER: Creek
Segment A        75'   TSTOD           5 spans   1994

Total            75'

MGT:        45          TPD:  18

ACCESS:     Road        PHOTO/DOC(S):  2    (06/30/06)
LATITUDE:   29:35:43    LONGITUDE:  096:20:23

(5)TST-OD (Timber Stringers)     LENGTH: 74'7"   BUILT: 1946 / 1994    CS: 01620
INSPECTED:              10/29/2009

1     T T   O O O O O                          O O    8     S               DMG  110   2 : 110  N/A
                                                                            RTG  144   2 : 144  N/A

   T  4B O  O  4A             :    : O  O  O  O                   14 4   PR DMG  51  45 113  68  45 113
                                                                            RTG  62  60 149  83  60 149

2     T T   O O O O O O                        O O  O 10    X S             DMG   76   3 :  76  N/A
                                                                            RTG  102   3 : 102  N/A

   T  4V O  O  2A             :    : O  O  O  O                   15     PR DMG  44  30 109  61  42 109
                                                                            RTG  54  41 143  75  56 143

3     T T   O O O O O O                        O O  O 12    X               DMG   77   3 :  77  N/A
                                                                            RTG  102   3 : 102  N/A

   T  4B O  O  4A             :    : 4A O  O  4A                  15        DMG  46  42 109  28  42 109
                                                                            RTG  56  56 143  35  56 143

4     T T   O O O O O O                        O O  O 12    X S             DMG   77   3 :  77  N/A
                                                                            RTG  102   3 : 102  N/A

   T  4A O  O  4A             :    : 4A O  O  4A                  15        DMG  28  42 109  28  42 109
                                                                            RTG  35  56 143  35  56 143

5     T T   O O O O O O                        O O  O 10    X               DMG   77   3 :  77  N/A
                                                                            RTG  102   3 : 102  N/A

   T  4A O  O  4A             :    : O  O  O  2A                  14 1      DMG  33  46 114  51  33 114
                                                                            RTG  40  62 150  63  45 150

6     T T   O O O O O                          O O  O 8                     DMG  113   2 : 113  N/A
                                                                            RTG  148   2 : 148  N/A

PILING:             1,6                      (5)  SIZE: 14" (44")  T
                    2-5                      (6)  SIZE: 14" (44")  T
CAPS:               1-6                      (2)  14.0" X  28.0" X  12.0'  T
SHIMS:              2                        (1)  0.50"  T
                    3                        (1)  0.80"  T
                    4-5                      (1)  1.00"  T
                    6                        (2)  1.50"  T
STRINGERS:          7.50" X  17.25"          Timber         YEAR: 1994
WELD JOINT COUNT:   1
WALKWAY:            LEFT: Steel;  RIGHT: Steel
HANDRAIL:           LEFT: Steel Cable;  RIGHT: Steel Cable
APPROACHES LOW:     NO            RC: P
TIES:               10.00" X 8.00" X  10.00'   TIMBER     2008    0 BAD / 60 TOTAL
SPACER BLOCKS:      NO
GUARD TIMBERS:      YES
PANDROL PLATES:     NO
RAIL ANCHORS:       PER STANDARD
END PATTERN:        PER STANDARD
RAIL:               136 LB Continuously Welded
SPEED:              50 MPH (FREIGHT)  50 MPH (PASSENGER)
FRA CLAS:           5
ALIGN:              TANGENT
```

Data last refreshed on 11/5/2009 8:34:04 PM. Page 1

GLIDDEN SUB 69.11

NO GUARDRAIL ON BRIDGE SEGMENT TRACK
APPROACH TIES NON-STANDARD
BRIDGE PROFILE: FAIR
NO STANDING WATER
UNDERWATER INSPECTION NOT REQUIRED

	INSP DATE	INIT	AREA	INSPECTION FINDING	INSP # | COMMENTS
I	10/29/2009	BTM	A1	Sub, Bent, Backwall, Missing, [3]	#64 | Backwall 1 top board missing and board 2 is out of place
A	06/26/2006	BTM	A1	Sub, Bent, Backwall, Working Out, [3]	#42 | Top board out of place and leaking ballast
A	06/02/2009	RP	A1	Super, Footwalk/Handrail, Panel #1 Both Sides, Walkway Grating, Missing, Severe, [2AS]	#57 | Wallkway Grating Missing on Panel # 1 Both Side
A	03/12/2002	JLW	A1	Super, Span, Span #ALL, Hardware, Fasteners Defective, [3]	#22 | 11 CHORD BOLTS MISSING IN BRIDGE
A	06/02/2009	RP	A2	Super, Footwalk/Handrail, Panel #2 Both Sides, Walkway Grating, Missing, Severe, [2AS]	#58 | Wallkway Grating Missing on Panel # 2 Both Sides
A	06/02/2009	RP	A3	Super, Footwalk/Handrail, Panel #3 Both Sides, Walkway Grating, Missing, Severe, [2AS]	#59 | Wallkway Grating Missing on Panel # 3 Both Sides
A	06/12/2007	BTM	A4	Sub, Bent, Piling, Insufficient Height, [3]	#48 | Bent 4 insufficient height both sides .75 inch
A	03/12/2002	JLW	A4	Sub, Bent, Bent #ALL, Cap Plate, Fasteners Defective, [3]	#23 | 7 SADDLE PLATE ANCHOR BOLTS MISSING IN BRIDGE
A	06/02/2009	RP	A4	Super, Footwalk/Handrail, Panel #4 Both Sides, Walkway Grating, Missing, Severe, [2AS]	#60 | Wallkway Grating Missing on Panel # 4 Both Sides
A	06/02/2009	RP	A5	Super, Footwalk/Handrail, Panel #5 Both Sides, Walkway Grating, Missing, Severe, [2AS]	#61 | Wallkway Grating Missing on Panel # 5 Both Sides
I	09/18/2002	JLW	A6	Sub, Bent, Bent #6 Both Left & Right, Wingwall, Insufficient Height, [3]	#25 | WINGWALLS WEST END BOTH SIDES ARE 20 INCHES LOW

	INSP DATE	INIT	AREA	INSPECTION FINDING	INSP # | COMMENTS
I	10/29/2009	BTM		Bridge, Inspection	#63 |
A	06/02/2009	RP		Bridge, Inspection	#62 |
A	07/02/2008	BTM		Bridge, Inspection	#53 |
I	10/21/2008	RP		Bridge, Inspection	#56 |

TOYAH SUB 534.35

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TOYAH SUB: Data last refreshed on 3/30/2011 6:27:47 PM.

B>>> 534.35(2)    [SIMN] STANTON   (MARTIN, TX)    OVER: Waterway
Segment A        42'    TSTOD          3 spans    1962

Total            42'

MGT:        37          TPD:  14

ACCESS:     Road        PHOTO/DOC(S):  2   (06/25/06)
LATITUDE:  32.12832    LONGITUDE:  -101.7913
ASBESTOS:  MAY CONTAIN - Codes F

(3)TST-OD (Timber Stringers)      LENGTH: 42'2"   BUILT: 1962 / 1962
INSPECTED:          02/16/2011

1      T T   M  M  M  M  M                      M    O  4               DMG  ***   *** : ***  N/A
                                                                        RTG  ***   *** : ***  N/A

     T  2A 2V 5A 4A              :     : 4A O  2B 4A              13 6     DMG  28  19  93  30  42 126
                                                                           RTG  38  26 123  40  56 166

2      T T   O  O  O  2V O                      O    O  8               DMG   49     4 :  49  N/A
                                                                        RTG   64     4 :  64  N/A

     T  4A 5V 2V 4A              :     : 4A O  O  5A              14 2     DMG  26  39 122  48  28  90
                                                                           RTG  35  53 160  64  38 119

3      T T   O  O  O  O  O                      O    O  6               DMG   82     2 :  82  N/A
                                                                        RTG  107     2 : 107  N/A

     T  2B O  O  4V              :     : 4B O  O  O               13 4     DMG  56  37 128  58  43 128
                                                                           RTG  74  50 168  77  58 168

4      T T   M  M  M  M  M                      M    O  4               DMG  ***   *** : ***  N/A
                                                                        RTG  ***   *** : ***  N/A

PILING:             1-4                          (5)  SIZE: 13" (41")  T
CAPS:               1-4                          (1)  13.5" X  13.5" X  14.0'  T
SHIMS:              1                            (4)  5.50"  T
                    2-4                          (2)  4.50"  T
STRINGERS:          8.00" X  15.25"        Timber        YEAR: 1962
WALKWAY:            LEFT: Steel;  RIGHT: Steel
HANDRAIL:           RIGHT: Steel Cable
APPROACHES LOW:     NO          RC: P
TIES:               8.00" X  10.00" X 10.00'   TIMBER      2011     0 BAD / 45 TOTAL
SPACER BLOCKS:      NO
GUARD TIMBERS:      YES
RAIL ANCHORS:       PER STANDARD
END PATTERN:        PER STANDARD
RAIL:               136 LB Continuously Welded
SPEED:              70 MPH (FREIGHT)
FRA CLAS:           4
ALIGN:              TANGENT
NO GUARDRAIL ON BRIDGE SEGMENT TRACK
APPROACH TIES NON-STANDARD
BRIDGE PROFILE:     GOOD
NO STANDING WATER
UNDERWATER INSPECTION NOT REQUIRED
```

	INSP DATE	INIT	AREA	INSPECTION FINDING	INSP # | COMMENTS
I	10/06/2010	ROD		Bridge, Inspection	#44 |
A	02/16/2011	JDP		Bridge, Inspection	#45 |

APPENDIX B: PHOTOS FROM GLIDDEN SUB 69.11

Figure B.1 Bridge No. 1 – Glidden Sub 69.11

Figure B.2 Displacement measurement Set up for Glidden Sub 69.11

Figure B.3 String Potentiometers hung below bridge – Glidden Sub 69.11

Figure B.4 Alternate base for String Potentiometers (Glidden Sub 69.11)

Figure B.5 Alternate base for String Potentiometers (Glidden Sub 69.11)

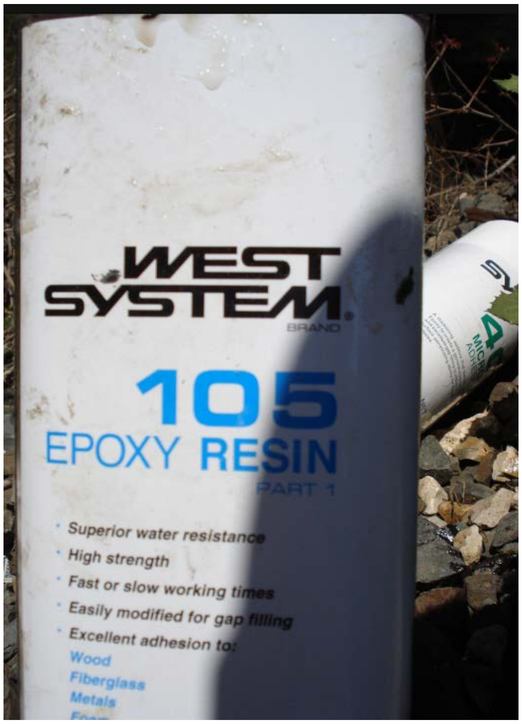

Figure B.6 Epoxy Resin Used for Spiking

Figure B.7 Hardener Used for Spiking


Figure B.8 Spiking Procedure

Figure B.9 Spiking Procedure

Figure B.10 Spiking Procedure

Figure B.11 Spiking Procedure

Figure B.12 Spiking Procedure

Figure B.13 Spiking Procedure

Figure B.14 Spiking Procedure

Figure B.15 Spiking Procedure

Figure B.16 Crane for loading bridge – Glidden Sub 69.11

Figure B.17 Crane for loading bridge – Glidden Sub 69.11

APPENDIX C: PHOTOS FROM TOYAH SUB 534.35

Figure C.1 Bridge Site – Toyah Sub 534.35

Figure C.2 Close up of crack in stringer 1, span 1. Coded 2A

Figure C.3 Span 1, Stringer 1, badly cracked due to large knots and split near abutment.

Figure C.4 Span 1, Stringer 1, badly cracked due to large knots and split near the abutment.

Figure C.5 Span 1, Stringer 1, badly cracked due to large knots and split near the abutment.

Figure C.6 Bad knots, flexural cracks - Span 1, abutment end of Stringer 1.

Figure C.7 Bad knots, flexural cracks - Span 1, mid span of Stringer 1.

Figure C.8 Bad knots, flexural cracks - Span 1, mid span of Stringer 1.

Figure C.9 Stringer 1, Span 2 - split in bottom of beam emanating from right end, notched bottom of beam at shimmed cap.

Figure C.10 Horizontal Shear Crack in Stringer 1, Span 2. Coded 4A

Figure C.11 Nathan pointing (2 fingers) to shear crack in stringer 4 , Span 1. Coded 4A

Figure C.12 Shear Crack in Stringer 4, Span 2. Coded 4A


Figure C.13 Split at mid depth between lateral tie rods at Pier 2, stringer 4, spans 1 and 2.


Figure C.14 Split at mid depth between lateral tie rods at Pier 2, stringer 4, spans 1 and 2.

Figure C.15 Stringer 5, Span 1 - Split between tie rods at interior pier. Coded 4A

Figure C.16 Shear crack in Stringer 5, Span 2


Figure C.17 Stringer 5, Span 1 – Horizontal crack looking away from the pier toward the abutment.


Figure C.18 Stringer 5, Span 1 – close up of horizontal crack.

Figure C.19 Stringer 5 – Two members butted together over pier.

Figure C.20 Stringer 5, Spans 1 and 2 – Two members butted together over pier.


Figure C.21 Stringer 5, Span 1 – Horizontal Crack


Figure C.22 Stringer 5, Span 1 – Horizontal Crack by abutment

Figure C.23 Stringer 5, Span 1 – Horizontal crack near abutment

Figure C.24 Stringer 8, Span 1 – Minimal damage, despite 4A coding in inspection report.


Figure C.25 View showing compressed Stringer 3, Span 1


Figure C.26 View of showing compressed Stringer 3, Span 2

Figure C.27 Instrumentation setup for Bridge No. 2

Figure C.28 Alternate view of Instrumentation Setup – Bridge No. 2

Figure C.29 Alternate view of Instrumentation Setup – Bridge No. 2

Figure C.30 Laptop and USB DAQ used for data acquisition.

Figure C.31 Stringer 8 – Gap over pier between Span 1 and Span 2

Figure C.32 Stringer 8 – Close up of damage over pier

Figure C.33 Stringer 8 - Shim added below end of Span 1, Span 2

Figure C.34 Stringer 8 - Shim added below end of Span 1, Span 2

Figure C.35 Stringer 8 - Shim added below end of Span 1, Span 2

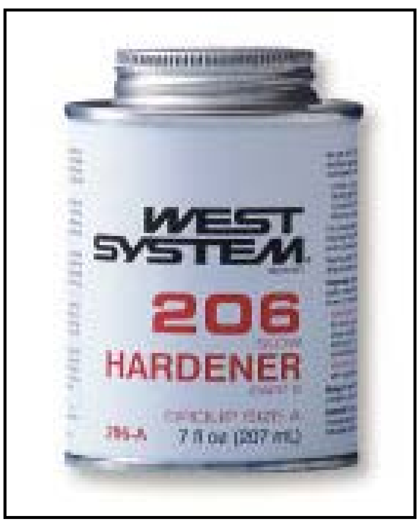

Figure C.36 West System 205 Fast Hardener (westmarine.com)


Figure C.37 Spray foam used to fill cracks


Figure C.38 Spray foam used to fill cracks

Figure C.39 Spray foam used to fill cracks

Figure C.40 Epoxy pushed out of cracks in beams


Figure C.41 Epoxy flowing through hole in bottom of beam.


Figure C.42 Cardboard expediently nailed to bottom of beam to stop flow of epoxy through hole.

Figure C.43 Top view of spikes inserted in beams.

Figure C.44 Top view of spikes inserted in beams.

Figure C.45 Top view of spikes inserted in beams.

Figure C.46 Spikes inserted by UPR employees.

Figure C.47 Spikes being driven in using a 4x4 board.

Figure C.48 Example train – double-stacked cars.

Figure C.49 Example train – empty cars

Figure C.50 Speed measured with Speed Gun.

Figure C.51 Locomotive crossing center span of bridge.

Figure C.52 Double stacked car crossing bridge.


Figure C.53 Empty car crossing bridge.


Figure C.54 Shimmed chord with stringer hanging below pier cap.