STRENGTHENING CONTINUOUS STEEL GIRDER BRIDGES WITH POST-INSTALLED SHEAR CONNECTORS

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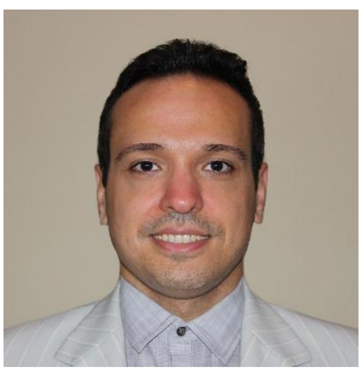

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BIOGRAPHY

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SUMMARY

Many continuous steel girder bridges constructed in the mid-1900s have non-composite floor systems with no shear studs. One method of strengthening such bridges is to “post-install” shear connectors to attach the existing concrete deck to the steel girders to create composite action. Increases in load rating of up to 60% can be achieved by installing a relatively small number of connectors, which have significantly improved fatigue strength over welded shear studs so that partially composite design is feasible. Because composite behavior provides little strength increase in negative bending, inelastic moment redistribution can be used to improve the load rating at the interior supports.

This paper summarizes the results of an experimental investigation on a strengthened girder tested under both fatigue and strength loading conditions, describes the design process and connector installation procedure, and discusses construction considerations for strengthening bridges in this manner.

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Introduction and Background

Many bridges constructed in the mid-1900s have non-composite floor systems consisting of a concrete deck on top of steel girders with no shear connectors. Although these bridges are nearing the end of their expected design life, many are still in good condition and can potentially remain in service for many more years. However, because these bridges were typically designed for smaller live loads than are used today, some may need to be strengthened to maintain a safe load-carrying capacity for current and future demands.

An efficient method of strengthening such bridges is to "post-install" shear connectors to connect the existing concrete deck to the steel girders and create composite action. This is most effective in regions of the bridge primarily subjected to large positive bending moments, where the concrete is in compression and can contribute a significant amount of strength and stiffness to the composite section. To address strength deficiencies in regions dominated by negative bending, inelastic moment redistribution from the interior supports can be considered following the provisions in the AASHTO *LRFD Bridge Design Specifications* (1).

An experimental program was carried out to investigate the behavior of a large-scale representative bridge girder strengthened with post-installed shear connectors at fatigue and strength limit states. The results of these tests, along with findings from previous research, were used to develop a design procedure for strengthening continuous non-composite steel girder bridges using post-installed shear connectors. This paper gives an overview of the experimental testing and results, along with a description of the design procedure, connector installation procedure, and construction-related items to consider when strengthening existing bridges in this manner.

Post-Installed Shear Connectors

The post-installed shear connectors used in this study were comprised of adhesive anchors, developed in previous research and shown in Figure 1 (2). Small-scale direct-shear testing of this connector indicated good static and fatigue performance, while large-scale simply supported beams strengthened with adhesive anchor connectors exhibited good overall performance (2).

The researchers that conducted this previous testing recommended the use of partial-composite design, which is commonly used in building structures as an efficient way of minimizing the required number of shear connectors to attain the particular strength needed (3). A partially composite girder is characterized by the composite ratio, or the ratio between the number of connectors provided to the number of connectors required for fully composite behavior. Because these post-installed connectors have very good fatigue strength as compared to conventional welded shear studs, strength limit states will often control the design over fatigue considerations, even for girders with low composite ratios.

The researchers also recommended that the connectors be placed in concentrated groups near regions of low moment, rather than spaced uniformly along the girders as is typically done with conventional welded studs. This improves the ductility of the strengthened girders.

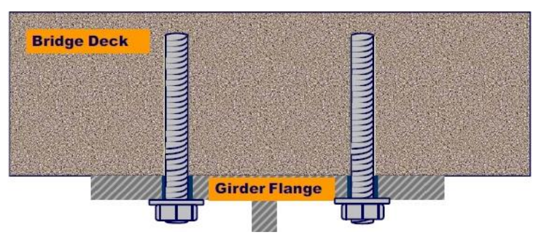

Figure 1 – Adhesive Anchor Post-Installed Shear Connectors (2)

Inelastic Moment Redistribution

Inelastic design concepts for steel bridges have been included in the AASHTO specifications since the 1970s by defining the flexural strength of compact, well-braced sections as the plastic moment capacity and by allowing for moment redistribution to occur from interior pier sections that are permitted to yield under the application of large loads. The appropriate limit state for yielding under a repeated load pattern is termed "shakedown," and refers to the stabilization of permanent deformations after several cycles of a particular load pattern (4). This stabilization occurs due to the formation of residual moments in the statically indeterminate structure which counteract the moments from the applied load. If the residual moments are of such a magnitude that when combined with the moments from the applied load, the flexural capacity of every section along the girder is not exceeded, elastic behavior will ensue for all future cycles of equal or lesser load.

This behavior serves as the basis for Autostress Design, also called Alternate Load Factor Design (ALFD), a bridge design method developed in the 1970s that incorporates inelastic analysis to determine the redistribution moments for design (5). A simpler procedure based only on elastic analysis has since replaced the Autostress Design provisions for moment redistribution and is covered in Appendix B6 of the AASHTO *LRFD Bridge Design Specifications* (6).

The capability of composite girders to "shake down" has been questioned over the years due to the lack of ductility of the concrete deck and the complex interactions of the shear connectors with the deck and the steel girders. Several experimental tests of composite girders and multi-girder systems have shown that the ALFD provisions are conservative (7-9). However, a few more recent tests of composite and partially composite girders indicate that the deflections may never truly stabilize in a composite system under shakedown-type loading (10-12). This is possibly due to the small-scale of these experiments, as none were larger than ½-scale models with deck thicknesses not exceeding 4 inches.

Survey of Non-Composite Bridges

Original plans and recent inspection reports were reviewed for 25 non-composite steel girder bridges in Texas to investigate the typical properties, geometry, and condition of bridges that may be candidates for strengthening with post-installed shear connectors (13). With a few exceptions, these bridges were constructed between 1955 and 1965 and have many common features.

The bridges contain two- to five-span continuous steel girder units with three to ten girders across the width of the bridge, carrying two to six lanes of traffic. Span lengths vary from 40 to 270 feet, with the spans not exceeding 100 feet long comprised of 27- to 36-inch deep rolled wide flange sections and longer spans made up of 4- to 10-foot deep plate girders. Cover plates are commonly welded on the top and bottom flange of the rolled wide flange girders at the interior piers and in some cases in the middle of the spans as well. All flanges of both the rolled wide flange and plate girder sections are compact, assuming a yield stress of 33 ksi corresponding to A7 grade steel, which was common for the time period (14).

Concrete deck thickness varies from 6 to 7.25 inches, with #4 or #5 bars for transverse and longitudinal reinforcement. Design compressive strength for all decks was 3000 psi.

The surveyed bridges are generally in acceptable condition with the most common problems being minor rust on all steel elements, over-rotation of bearings, and minor to moderate cracking and spalling of the deck and concrete substructure. Superstructure inspection ratings range from 5 to 7, or fair to good, while substructure ratings vary from 4 to 8, or poor to very good.

Analysis of nearly half of the surveyed bridges both before and after strengthening indicates that up to a 60% increase in the load rating can be achieved with this strengthening method. The majority of these bridges can be strengthened to reach a load-carrying capacity exceeding that required by current design standards with a composite ratio of only 30% and minimal moment redistribution.

Experimental Program

Laboratory testing of a two-span continuous girder strengthened with post-installed shear connectors was conducted to examine the structural behavior of such a system under different types of loading. A summary of the test program and results is presented here, and more details can be found elsewhere (13).

Test Specimen and Setup

The specimen, pictured in Figure 2, has symmetric 42-foot long spans and is comprised of a W30x90 steel beam with a 6.5-foot wide, 6.5-inch thick concrete deck reinforced with details typically found in the surveyed bridges. After erecting the steel and casting the deck, a total of 56 adhesive anchor connectors were installed in pairs in four groups along the girder, as shown in Figure 3, resulting in a composite ratio of approximately 30%. The test setup consisted of three load frames which supported 400- or 500-kip capacity hydraulic rams and could be configured to apply point loads at the four locations labeled "A" through "D" in Figure 3. As illustrated in Figure 4, these loads were used to approximate the peak force effects from a typical moment envelope derived from the bridge live load. The application of Load A causes a large positive moment and engages the shear connectors in the north span, while Load D (not shown in this figure) results in similar behavior in the south span. Applying Loads B and C simultaneously creates a large negative moment at the interior support and does little to engage the connectors in either span.

The applied loads and reaction forces were measured using 100- or 500-kip capacity load cells. Vertical deflections were measured at eight equally-spaced points along the girder using linear and string potentiometers. Interface slip was also measured using linear potentiometers at varying locations along the girder, with a general focus on the locations of the connectors. More than 200 strain gages were installed on the steel beam in several sections along the girder to monitor the location of the neutral axis, which is an indication of the level of composite action, and to estimate the force carried by a pair of shear connectors. The difference in the axial force in the steel beam on either side of a pair of connectors, as computed using the measured strains, provides an estimate of the force transmitted by those connectors into or out of the concrete deck.

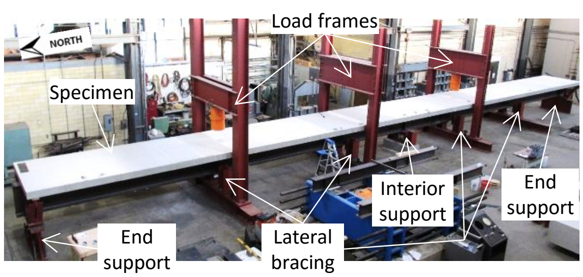

Figure 2 – Test Specimen and Setup

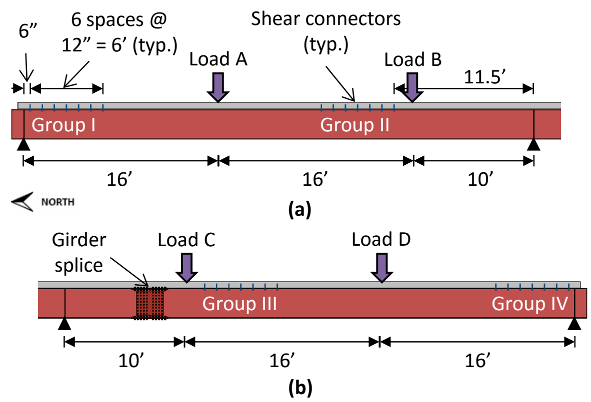

Figure 3 – Elevation View of Test Specimen – (a) North Span and (b) South Span

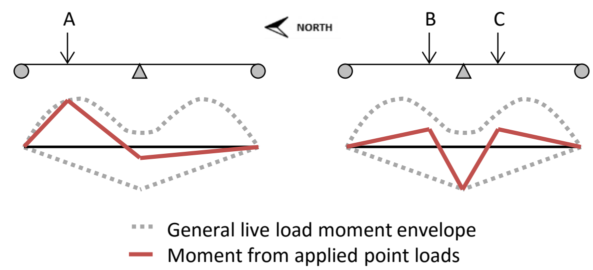

Figure 4 – Loading Scheme for Test Specimen

Loading History

To represent the variety of demands that may be placed on a strengthened bridge, testing was conducted over approximately a 6-month period under many different load types and magnitudes to observe the behavior under elastic, fatigue, shakedown, and ultimate strength-level loads. The following describes the phases of testing, listed in chronological order:

1. Elastic testing of the non-composite girder in the north span prior to installing any shear connectors (Load A = 40 kips) to break the natural bond at the steel-concrete interface and provide a baseline for non-composite behavior. Steel stresses did not exceed 35% of the yield stress in this phase.

2. Elastic testing of the composite girder in the north span (Load A = 40 kips) to evaluate the stiffness increase from installing connectors and provide a baseline for composite behavior.

3. Shakedown testing in the north span (Loads A, B, and C), consisting of repeated cycles of the load pattern shown in Figure 4 to simulate the

effects a large truck crossing one-half of the bridge. Cycles were repeated at the same magnitude of load until the change in deflection from one cycle to the next was less than 0.01 inches. At this point, shakedown was deemed to have occurred and the magnitude of the loads was increased for future cycles.

4. Fatigue testing in the south span (Load D = 50-kip range for 2 million cycles) at a load level that caused connector slips that would be expected to occur under HL-93 fatigue loading in a typical strengthened bridge. A closed loop control system was used to apply a sinusoidal load with a frequency of 0.85 Hz.
5. Fatigue testing in the north span (Load A = 75-kip range for 330,000 cycles), at a load level 50% greater than would be expected to occur under HL-93 fatigue loading in a typical strengthened bridge using a sinusoidal load with a frequency of 0.45 Hz.
6. Shakedown testing in the south span (Loads B, C, and D) in a similar manner to the shakedown testing conducted previously in the north span.
7. Ultimate strength testing in the south span (Load D = 233 kips maximum), conducted under monotonic load through a maximum deflection of more than 14 inches.
8. Ultimate strength testing in the north span (Load A = 240 kips maximum), conducted in a similar manner as in the south span to a maximum deflection of nearly 9 inches.

Experimental Results

Throughout all phases of testing, the girder exhibited good structural performance and resilience. No shear connector failures occurred in either span until the ultimate strength testing was conducted in the final phases.

Elastic Testing

Small levels of load were sufficient to break the bond at the steel-concrete interface along the majority of the girder with the exception of the region near the load point. The remaining bond was broken at this point by directly lifting the deck with small hydraulic rams placed under the deck on either side of the girder. The elastic behavior of the girder, both before and after installing the connectors, was reasonably well-predicted by finite element modeling, as shown in Figure 5. The deflections of the composite girder are nearly half of those of the non-composite girder, indicating that installing the connectors increased the flexural stiffness by a factor of nearly two.

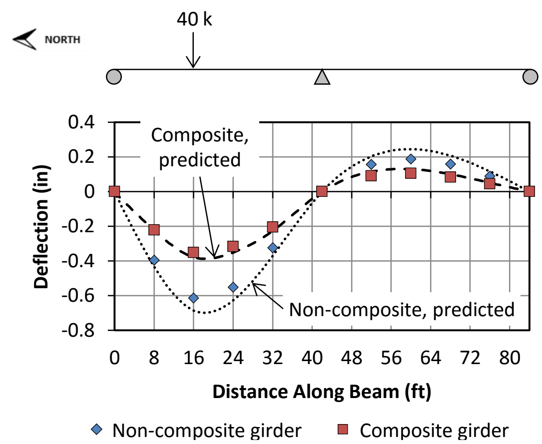

Figure 5 –Elastic Testing Results

Fatigue Testing

The results for fatigue testing in both the north and south spans are summarized in Figure 6 and Figure 7, which show the variation of the stress range and slip range, respectively, for the connectors in the tested span throughout the duration of the tests. These ranges were calculated as the difference between the maximum and minimum values of stress and slip during a single cycle of load. Additionally, Figure 8 illustrates the force-slip behavior of the connector closest to the interior support in each span at four points throughout the tests. Note the difference in the range of the horizontal axes in Figure 8, as significantly larger slip ranges were observed in the north span test. The stress range is calculated by dividing the force range by the effective shear area of the connectors (0.48 square inches).

The south span test, which was conducted at a load level expected to induce connector slips similar to the effects of an HL-93 fatigue truck on typical bridges from the survey, was run for a duration of approximately two million cycles. The connectors in the south span, which had not been previously loaded to any significant level, exhibited very good fatigue performance during the test with no failures nor significant damage to the connectors occurring. Throughout this test, the stress range remained fairly constant on each individual connector, varying from approximately 10 to 25 ksi over all of the connectors in the south span, as indicated in Figure 6. However, the slip range measured in each connector decreased steadily as the test progressed, reaching a value of nearly half of the original slip by the end of the test, as can be seen in Figure 7. This decrease in slip range is primarily due to the increase in the minimum value of slip measured within a single cycle, rather than change in the maximum slip, as indicated in Figure 8(a). Coupled with the constant stress range, the decreased slip range indicates that the effective stiffness of the connector is increasing with the number of cycles. One possible explanation for this behavior is that small permanent deformations in the adhesive surrounding the threaded rod of the connector accumulate with increasing number of cycles so that the connector is not forced back into its original position upon unloading, and the compressed adhesive provides a stiffer response.

Because of the very good behavior observed in the south span test, the load magnitude was increased by 50% for the test in the north span in attempts to induce a fatigue failure. Additionally, this test was conducted after shakedown testing in the north span so the connectors had previously been subjected to very large force demands. While no fracture of the threaded rod was observed in any of the connectors, degradation of the adhesive between the threaded rod and the hole in the top flange of the steel beam occurred as the test progressed. This degradation progressed to the point that the response was essentially equal to that of a non-composite girder, as the rod of each connector simply slipped within the oversized hole in the steel flange without coming into bearing. This can be seen in Figure 7, in which a large increase in the slip range occurs after an initial constant trend. The hole in the steel flange is nominally 1/8-inch larger than the threaded rod, corresponding approximately to the maximum slip range values indicated in the graph. The test was stopped after approximately 330,000 cycles because the connectors were no longer carrying a significant amount of load, as illustrated in Figure 6 and Figure 8(b).

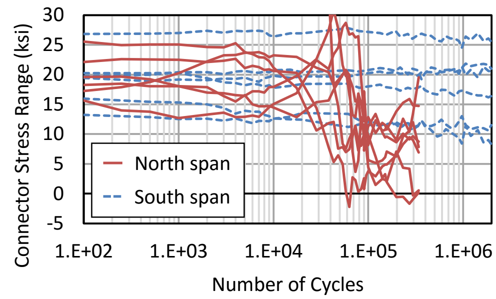

Figure 6 – Variation in Connector Stress Range during Fatigue Testing

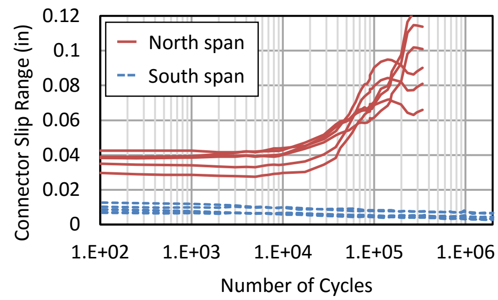

Figure 7 – Variation in Connector Slip Range during Fatigue Testing

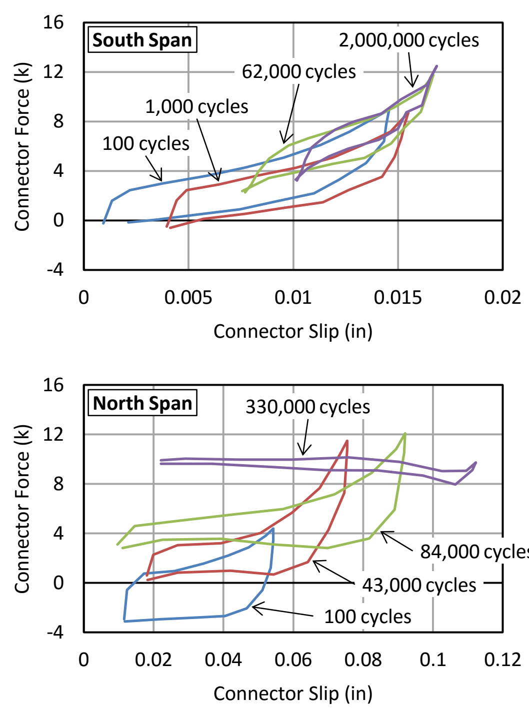

Figure 8 – Behavior of Connectors Closest to Interior Support during Fatigue Testing

Shakedown Testing

A comparison of the results from the shakedown tests for both spans is given in Figure 9. A total of 89 cycles (alternating between Load A and Loads B & C) at 19 load levels were applied during the north span test, which was stopped after shakedown was achieved at a load level 5% higher than the predicted shakedown limit load to preserve the specimen for future phases of testing. The south span test consisted of 84 total cycles (alternating between Load D and Loads B & C) at 20 load levels and was stopped after significant local buckling of the web occurred at the interior support at a load level 15% higher than the predicted shakedown limit load. The predicted shakedown limit load for the partially composite girder is nearly 50% higher than that of the non-composite girder.

In Figure 9, the individual data points represent the peak deflections for each cycle of load applied to girder. At a given load magnitude, larger peak deflections were observed in the north span test than in the south span test, especially at larger loads. This is primarily a result of the order in which the testing was conducted, as the north span was tested prior to the south span. The load pattern applied during both tests consisted of the simultaneous application of Loads B and C, which causes a large negative moment at the interior support, as shown in Figure 4. A significant amount of yielding at the interior support occurred during the north span test, causing the formation of residual moments in the girder. When the same magnitudes of Loads B and C were applied later in the south span test, elastic behavior was observed at the interior support because the residual moments developed during testing of the north span counteracted the applied loads so that the net moment on the section at the interior support remained within the elastic range. Although the two spans were considered separately for the purposes of conducting these phases of testing, it is clear that shakedown behavior and moment redistribution affects the entire girder.

While the peak data points generally create a backbone curve shaped similarly to that which would be expected in a test consisting of a monotonic load, the behavior under each cycle of load was essentially elastic. This is indicated by the two solid lines which show the load-deflection behavior for the last cycle of the largest load that was applied to both spans in positive bending (191 kips). Throughout the test, small inelastic deformations were accumulated with each cycle of load, and the magnitude of these permanent deformations decreased with each cycle at the same load level. This behavior is shown in Figure 10, which plots the change in peak deflection between positive bending load cycles at four different load levels during the north span test. Once the change in deflection for both the positive and negative bending cycles at a load level dropped below 0.01 inches, shakedown was considered to have been achieved and the loads were increased for the following cycle.

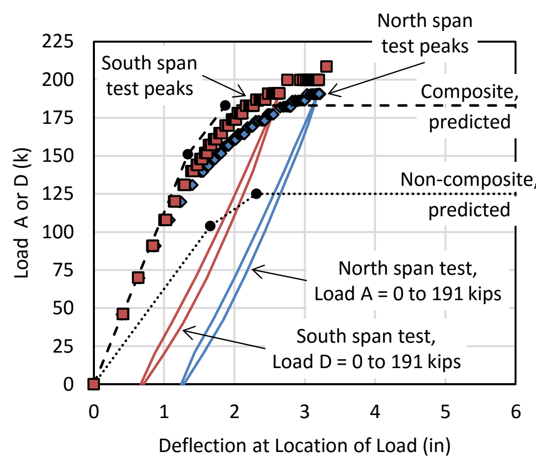

Figure 9 – Load-Deflection Behavior during Shakedown Testing

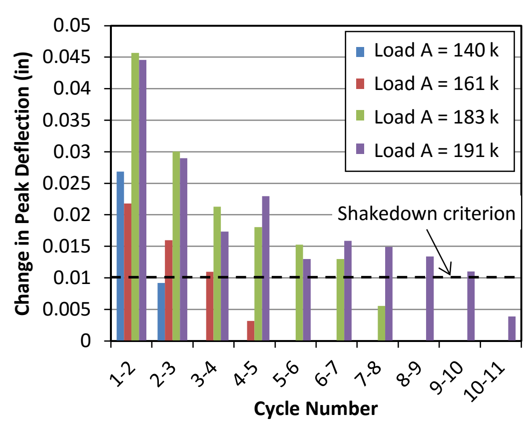

Figure 10 – Stabilization of Deflections during Shakedown Testing in the North Span

Ultimate Strength Testing

The load-deflection behavior of each span of the girder during ultimate strength testing under monotonic load is shown in Figure 11. The initial deflection reflects the residual deflection at the load point prior to the ultimate strength testing of each span. Elastic behavior was observed in both spans up to a load of approximately 200 kips, corresponding to the maximum load in positive bending applied during shakedown testing. However, the elastic stiffness of the north span is significantly lower than that of the south span because of the adhesive degradation that occurred during fatigue testing at large loads, which allowed the connectors to slip through the oversized holes in the top flange of the steel beam without coming into bearing. This decreased elastic stiffness lies between the predicted composite and non-composite stiffness.

Despite the adhesive degradation around the connectors in the north span, the peak load in both spans exceeded the predicted partially composite strength by approximately 10% and the predicted non-composite strength by nearly 50%, as predicted by a simple plastic hinge model. This indicates that at strength limit states, the connectors with degraded adhesive will slip to an extent that the threaded rod comes into direct bearing with beam flange to provide composite action and the associated composite strength.

In both spans, a sharp drop in load occurred at a deflection between 6 and 8 inches, at which point all of the connectors in one group in the tested span fractured suddenly. All connectors in Group I failed during the north span test, while the connectors in Group III failed during the south span test. After this failure, however, the load continued to increase with increasing deflections, indicating that some composite action was still being developed in the girder, possibly through friction at the steel-concrete interface and the remaining shear connectors in Groups II and IV. The south span test was pushed to a total deflection of more than 14 inches with only small amounts local buckling occurring in the steel beam at the load point. Unloading and reloading after connector failure occurred elastically. The north span test was concluded shortly after connector failure to prevent damage to the test setup.

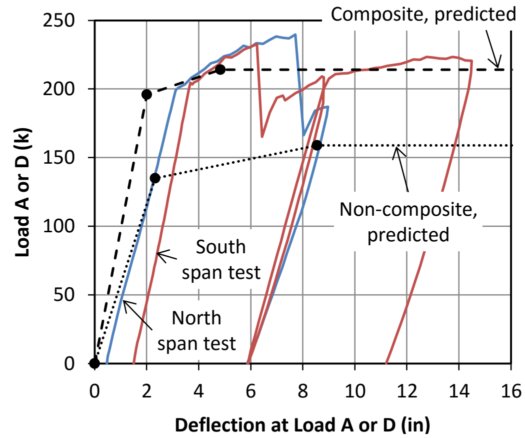

Figure 11 – Ultimate Strength Testing Results

Summary of Experimental Behavior

Throughout all phases of testing, the strengthened girder exhibited resilient and ductile behavior. The fatigue performance of the connectors was very good under force and slip demands similar to those expected to be caused by an HL-93 fatigue truck. The predicted capacity was exceeded at both shakedown and ultimate strength limit states, despite some degradation of the adhesive observed in connectors subjected to repeated large loads.

Recommended Design Procedure

The strengthening design procedure will generally include the following steps, which are discussed in more detail below:

1. Evaluate the existing bridge
2. Set strengthening targets
3. Check negative moment regions of strengthened bridge at strength limit states, and redistribute moments if needed
4. Design connectors for positive moment regions of strengthened bridge for strength limit states
5. Locate connectors along girder
6. Check fatigue limit states

Evaluate Existing Bridge

Evaluating the existing structure generally will consist of conducting a load rating using any available design drawings and material properties that are representative of the current condition of the bridge. Guidelines and recommendations for conducting this evaluation can be found in the AASHTO *Manual for Bridge Evaluation* (14). The evaluation can be done using any magnitude of live load, including specific permit loads, and any type of load rating, including allowable stress, load factor, or load and resistance factor rating at either the inventory or operating levels. It is recommended to use the same live load analysis results for both the existing non-composite and strengthened partially composite bridge, as finite element modeling indicates that post-installing the connectors does not significantly change the stiffness distribution along the girders.

Set Strengthening Targets

Targets for the strengthened bridge should include both strength and fatigue considerations. For strength limit states, targeting a particular increase in load rating is recommended. To address fatigue limit states for the strengthened bridge, the desired remaining life of the bridge in years should be determined, along with an estimate of the average daily truck traffic in a single lane ($(ADTT)_{SL}$).

Negative Moment Regions – Strength

The design of the strengthened bridge begins with the negative moment regions to determine whether or not moment redistribution is required from the interior supports, as this will affect the flexural demand used to design the positive moment regions. A load rating of the negative moment regions is conducted using the moment envelopes and member capacities corresponding to the live load chosen as a strengthening target.

If the load rating of the existing negative moment regions falls below the target, inelastic moment redistribution can be used to increase the load rating at the interior supports. It is recommended to use the provisions in Appendix B6 of the AASHTO *LRFD Bridge Design Specifications* for moment redistribution, as they have been greatly simplified from earlier provisions based on autostress design principles. After computing the "redistribution moment diagram," these redistribution moments are added to the elastic moment envelopes for the remainder of the design.

These provisions require that the bridge and bridge girders meet certain criteria to ensure that the provisions are not applied to cases without adequate experimental validation and that the steel section at each interior support has enough plastic rotation capacity to accommodate the redistribution. The surveyed bridges generally meet most of these criteria, although it is important to note that bearing stiffeners may need to be installed at interior supports. Additionally, moment redistribution is not allowed by the current specifications for bridges with any horizontal curvature or for bridges with supports skewed more than 10°.

Positive Moment Regions – Strength

To increase the strength of the positive moment regions in each span, post-installed shear connectors are added to create composite behavior. The number of shear connectors required in each span to meet the target strength demands, including redistributed moments if applicable, is determined using simple plastic cross sectional analysis (3). The strength of a single adhesive anchor connector, Q_n (kips), is (2):

$$Q_n = 0.5A_{sc}F_u \qquad \textit{Equation 1}$$

where A_{sc} is the effective area of the connector (square inches), taken as 80% of the gross area to account for the threads in the shear plane, and F_u is the nominal ultimate tensile strength of the connector material (ksi). It is not recommended to use a composite ratio lower than 30%.

Locate Connectors

Parametric studies were carried out to investigate the optimal layout of post-installed connectors along a single girder line to minimize the demand under elastic-level fatigue loads. The following guidelines for choosing a connector layout are recommended based on these studies:

- Place connectors in pairs within a cross section with one on either side of the web. Requirements for the spacing, cover and edge distance from the AASHTO *LRFD Bridge Design Specifications* should be followed.
- Concentrate connectors in groups near regions of low moment using a longitudinal spacing of approximately 12 inches, but equal to a multiple of the spacing of the transverse deck reinforcement. This will help to prevent drilling into the rebar during construction.

- Locate the connector closest to the end of a continuous girder unit at a distance of one-half of the longitudinal spacing from the centerline of the exterior bearing.
- Locate the interior connector groups so that the connector closest to the interior support is approximately 15% of the span length from the centerline of that support.

Check Fatigue

For partially composite girders, the fatigue demand on the post-installed connectors should be computed using an analysis technique that explicitly considers the slip between the underside of the concrete deck and the top flange of the steel beam. This slip can significantly reduce the force demands on the connectors as compared to a fully composite system (15). Additionally, because the connectors are not uniformly spaced along the girders, determining the connector force demand through the typical method for welded studs as the product of the interface shear flow and the connector spacing is difficult.

One method of accounting for the effects of slip in a partially composite girder is to follow an analytical procedure based on elastic beam theory and equilibrium (16-17). This procedure considers the steel beam and concrete slab as separate entities attached by discrete shear connectors, and involves an iterative solution procedure to satisfy force equilibrium. While this method is not suitable for hand calculations, it can be easily programmed into a spreadsheet. Alternatively, the fatigue demand can be determined computationally using a 3D model that represents the deck, steel beams, and shear connectors as discrete objects. In both of these cases, the adhesive anchor shear connectors are recommended to be represented by linear elastic springs with a stiffness of 900 kips per inch. This value of stiffness was determined empirically based on observations from laboratory tests.

The following equations represent preliminary fatigue design provisions for adhesive anchor shear connectors, presented in a parallel manner to the provisions for conventional welded shear connectors in the AASHTO *LRFD Bridge Design Specifications*. The fatigue resistance of a single adhesive anchor shear connector, Z_r (kips), depends on the expected daily truck traffic on the bridge over the remaining life of the structure and is defined by the following equations:

$$(ADTT)_{SL\,limit} = \frac{22600}{Y} \qquad \textit{Equation 2}$$

where Y is the desired remaining life of the strengthened bridge (years). If the $(ADTT)_{SL}$ is greater than this limiting value, the Fatigue I load combination is used to design for infinite fatigue life. The fatigue resistance is:

$$Z_r = 9.4d^2 \qquad \textit{Equation 3}$$

If the $(ADTT)_{SL}$ is smaller than the limiting value from Equation 2, the Fatigue II load combination is used to design for finite fatigue life with:

$$Z_r = (63.5 - 8.5\,log(N))d^2 \qquad \textit{Equation 4}$$

$$N = (365)(Y)(n)(ADTT)_{SL} \qquad \textit{Equation 5}$$

where d is the diameter of the connector (inches) and n is the number of stress cycles on the connector for a single truck passage, as defined in the AASHTO *LRFD Bridge Design Specifications*.

The fatigue demand, which is expressed in terms of the force range, ΔF (kips), that a particular connector is subjected to as a fatigue truck crosses the bridge, must not exceed the fatigue resistance of a single connector:

$$\Delta F \le Z_r \qquad \textit{Equation 6}$$

These equations were determined empirically from 17 small-scale fatigue tests on 7/8-inch diameter adhesive anchor shear connectors (17). Previous testing of 3/4-inch diameter connectors yielded slightly higher fatigue strength (18). Caution should be used in applying these design equations to connectors with diameters smaller than 3/4 inches or larger than 7/8 inches. Additionally, after completing the fatigue check on the post-installed connectors, other elements of the bridge should also be checked for fatigue.

Strengthening of Sample Bridge

The bridge shown in Figure 12 represents a typical structure that may be a candidate for strengthening with post-installed shear connectors. Sample calculations for the strengthening design of this particular bridge were carried out, and specific details can be found elsewhere (13). Evaluation of an interior girder of the existing bridge yields a load factor rating of HS 12.6 at the inventory level, considering both the Overload and Maximum Load limit states from the AASHTO *Standard Specifications for Highway Bridges* (19).

A strengthening target of an HS 20 inventory load factor rating can be attained by post-installing a total of 128 adhesive anchor shear connectors along a single girder line. This results in the outer spans becoming nearly 50% composite and the middle span becoming 30% composite. The shear connector layout, shown in Figure 13, was slightly modified from the recommended layout by shifting the interior connectors in the outer spans closer to the interior support to reduce the fatigue demand. A remaining life of 20 years was chosen as a strengthening target, with an estimated $(ADTT)_{SL}$ of 300 trucks per day over that time.

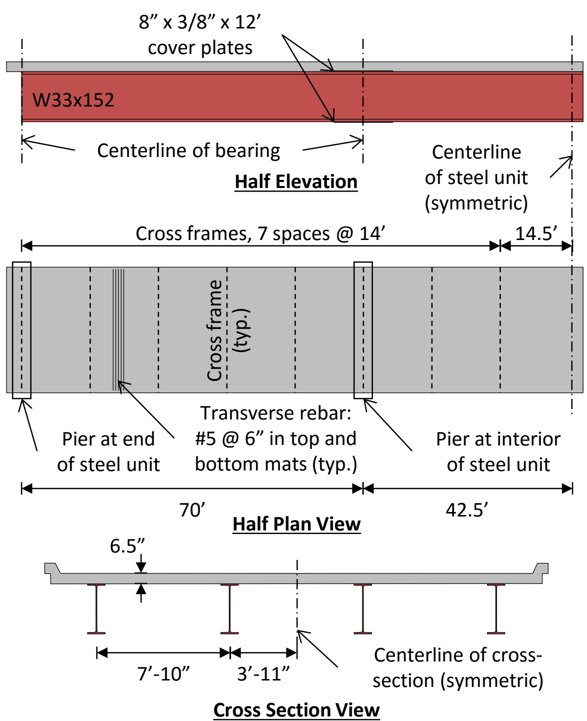

Figure 12 – Sample Bridge Geometry

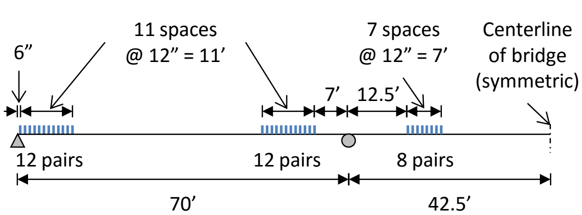

Figure 13 – Connector Layout for Sample Bridge

Connector Installation Procedure

The adhesive anchor shear connectors used in this study were comprised of 7/8-inch diameter ASTM A193 Grade B7 threaded rods and a structural adhesive. Figure 14 shows a photograph of a typical group of connectors after installation. The following steps describe the installation process:

1. Drill a 1-inch diameter hole through the top flange of the steel beam. This can be done using a portable drill with a magnetic base.
2. Through the hole in the flange, drill a 15/16-inch diameter hole into the concrete deck to the desired depth. This can be done using a rotary hammer drill. At least a 2-inch cover to the top of the concrete deck should be maintained, following the provisions in the AASHTO LRFD specifications for shear connectors.
3. Clean the hole with a wire brush and compressed air, or as specified by the adhesive installation instructions.
4. Inject the adhesive into the hole as specified by the installation instructions. Fill the hole from the top down so that no air bubbles are present.
5. Place the threaded rod into the hole using a twisting motion so the adhesive fills the threads. The threaded rod should be long enough so that it extends below the underside of the bottom flange an adequate distance to accommodate a washer and a nut.
6. Allow the adhesive to cure as specified in the instructions. After curing, install the washer and nut. Tighten the nut to the torque specified by the installation instructions.
7. Strike the threads below the nut with a grinder. This will prevent the unlikely event of any nuts that may loosen over time falling on traffic or pedestrians passing under the bridge.

Figure 14 – Installed Adhesive Anchor Connectors

Construction Considerations

Post-installed shear connectors have been used to strengthen a simple span bridge located near San Antonio, Texas (20). Experiences from field installation on this bridge and from laboratory work have indicated the following items to be considered when planning to post-install shear connectors in an existing bridge:

- When completing the design and choosing specific connector locations, accessibility and constructability should be taken into consideration. In some cases, it may be more economical to install slightly more connectors in a less efficient layout if the locations of the connectors are more easily accessible for installation. Slight modifications to the recommended connector layout will generally not have a significant effect on performance.

- Depending on the particular bridge, there may be some preparation work that needs to be done before the connectors can be installed. This may include general cleaning and rust and/or paint removal in the areas in which connectors will be installed.

- The annular cutters used to drill through the flange of the steel beam can become dull quickly when coming in contact with concrete. Care should be taken not to penetrate into the concrete deck when drilling through the steel beam. This is a more significant issue if the inner flange surface of the steel beam is sloped, as for S-shaped sections.

- The use of a rebar locator to determine the positions of the transverse reinforcing bars in the deck is highly recommended to avoid hitting rebar when drilling into the deck. If a bar is encountered during installation, shift away from that location approximately one-half of the rebar spacing and continue installing connectors from the new location.

- It is recommended to choose a high quality structural adhesive that is viscous enough to not run downwards after it is injected into the hole and to hold the threaded rod in place once it is inserted. The adhesive used in all laboratory testing (Hilti HIT-HY 150-MAX and 200-R) exhibited adequate viscosity and no issues were encountered during the installation process. However, a different type of adhesive was used to install connectors in the field which was less viscous and created some difficulties in completing the field installation.

Summary and Conclusions

This study investigated the static and fatigue behavior of continuous bridge girders strengthened with post-installed shear connectors and inelastic moment redistribution, and developed design recommendations for strengthening existing non-composite bridges in this manner. The proposed strengthening method provides an efficient way to extend the service life of an existing bridge and is a feasible alternative to load-posting or other strengthening measures, such as a full deck replacement to install conventional welded shear studs. Resilient and ductile structural performance was observed under a variety of fatigue and strength loading conditions during a 6-month long large-scale experimental study of a two-span strengthened girder. The design procedure is rational and is based largely on existing bridge design provisions, while also incorporating the efficiency of partial-composite design concepts and a type of shear connector with very good fatigue strength. The installation of the adhesive anchor connectors is straightforward and requires minimal traffic interruption.

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