



Large-scale fatigue testing of post-installed shear connectors in partially-composite bridge girders

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ABSTRACT

Many older bridges are constructed with structural systems consisting of a non-composite concrete deck over steel girders. A potentially economical method for strengthening these bridges is to develop composite action by attaching the existing concrete deck to the steel beams using post-installed shear connectors comprised of adhesive anchors. Because fatigue is one of the main concerns in designing bridges, investigating the fatigue properties of these post-installed shear connectors is crucial. Although the fatigue life of post-installed shear connectors has been explored through direct-shear tests, the actual fatigue performance of post-installed shear connectors in large-scale beam tests has not been extensively studied and may differ from component tests. The current paper describes the results of large-scale beam experiments on the fatigue performance of post-installed shear connectors. The results of this research indicate that adhesive anchor post-installed shear connectors have much better fatigue performance in beam tests compared to the previously reported fatigue test results based on direct-shear tests.

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1. Introduction

Many older bridges are constructed with floor systems consisting of a non-composite concrete deck over steel girders. Most of these bridges were designed using smaller loads than the standard design loads currently used for new bridges, as specified by the American Association of State Highway and Transportation Officials (AASHTO) [1]. The inadequate strength of these bridges can result in the need to limit truck loads through load posting or may require replacement of such a bridge. Alternatively, strengthening measures can be undertaken to increase the load rating of these non-composite steel girder bridges.

A potentially economical means of strengthening these floor systems is to connect the existing concrete slab and steel girders to permit the development of composite action. Composite action permits the existing steel girder and concrete slab to act together more efficiently than in the original non-composite condition. Connecting the steel girders and concrete slab using shear connectors can increase the load-carrying capacity of the girders by more than 50% in the ultimate limit state compared to that of non-composite girders, in which the two structural components act separately in flexure [5,11].

For new bridges, composite action is achieved by welding shear studs to the top flange of the steel girder prior to casting the concrete

slab. In the case of an existing bridge, however, this approach is not practical because the concrete slab is already in place. Consequently, economical and effective methods for post-installing shear connectors are needed to take advantage of composite action in existing bridges.

2. Background

2.1. Post-installed shear connectors (PISC)

Over the past 15 years, researchers at the University of Texas at Austin [8,9,14,19,21] investigated 14 different types of post-installed shear connectors (PISCs). Results of small-scale component testing show that PISCs can perform better than conventional welded studs in fatigue and strength limit states. The strength limit states include the ultimate strength of the girder under static loading as well as the performance under the shakedown behavior with cyclic loading. This means that partially-composite design can be used to achieve the necessary static strength using fewer PISCs than welded studs, while maintaining adequate fatigue strength for the expected remaining life of the bridge. To take advantage of partially-composite design, PISCs can be used in the positive moment regions, where composite action is highly efficient [14]. For older bridges with inadequate flexural capacity in the negative moment regions, adding PISCs in these regions may not be effective due to the limited amount of deck reinforcing steel. However, recent research has shown that limited flexural yielding can be permitted in

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the negative moment regions to redistribute moment to the strengthened positive moment regions [5,12,13]. Moreover, previous research by Kwon et al. [14] indicates that grouping the shear connectors near the points of zero moment rather than distributing them evenly along the girder improves ductility of the beams. As described in Kwon et al. [14], this observation is supported by theoretical studies by Oehlers and Sved [16], by finite element studies, and by experimental studies.

Partially-composite beams are described by the composite ratio. The composite ratio is defined for partially-composite beams by $\frac{N}{N_f}$, where N is the number of connectors used and N_f is the number of shear connectors required for fully-composite behavior. A key difference between partially and fully-composite design is that the relative horizontal displacement of the concrete deck with respect to the steel girder, i.e. “slip”, is significant under service loads in a partially-composite girder and should be considered in design. Fig. 1 compares non-composite, fully-composite, and partially-composite girders.

2.2. Analysis of shear connectors in partially-composite beams

The analysis of partially-composite beams can be challenging because plane sections do not remain plane. This means that the elastic shear flow equation $\tau = \frac{VQ}{I}$ is no longer valid for obtaining the force and slip demands on shear connectors when designing for fatigue. Therefore, an alternative method of analysis is required to design shear connectors in partially-composite beams.

Newmark et al. [15] developed an approach to analyze shear connectors in partially-composite beams. This method is based on equilibrium and compatibility of displacements and is applicable only to partially-composite beams in which the shear connectors are uniformly spaced along the beam and where there is a linear relationship between the shear force and slip in the connectors. However, these two limitations may not be satisfied when using PISCs to strengthen an existing bridge girder.

Proctor [20] developed an approach to address these limitations of Newmark's method. The partial interaction theory behind Proctor's method is nearly identical to Newmark's method, though it is iterative and requires two initial guesses: 1) location of the point with zero slip

and 2) the magnitude of horizontal interface shear at the point of zero slip. These two initial guesses are correct if the interface shear turns out to be zero at both ends of the beam, which are the two boundary conditions (Fig. 2). A limitation of Proctor's method is that no guidance is given for guessing the two initial values and performing iterations.

For the research described in this paper, Proctor's method was modified to predict slip of shear connectors before any experimental testing. Proctor's modified method was used to determine the location and magnitude of the load for the fatigue tests to achieve the desired connector slip ranges for testing. One of the main modifications was reducing the number of initial guesses as well as boundary conditions from two to one by guessing only the slip value at one end of the beam and making sure the interface shear is zero at the other end. Moreover, detailed guidelines were provided on how to make the first guess and how to perform the iterations with a fast rate of convergence. A detailed summary of how Proctor's method was revised and used in the test program is provided in [6].

It is important to note that PISCs are being added to improve the strength of bridges which already perform adequately under service and fatigue loads without PISCs. Thus, adequate fatigue behavior of PISCs from this perspective means that PISCs do not fracture in fatigue and can resist strength level loads after being subjected to fatigue loads.

2.3. Adhesive anchor shear connector

Patel [19] suggested that the adhesive anchor shear connector is an attractive alternative among PISCs in terms of strength, ductility, fatigue performance, and constructability. In addition to its excellent strength and fatigue performance, the adhesive anchor shear connector is installed entirely from the underside of the bridge deck and thus requires minimal disruption to bridge traffic during anchor installation. The adhesive anchor shear connector is shown in Fig. 3.

Patel [19] performed small-scale component fatigue testing on the adhesive anchor shear connector and determined a 0.2-mm slip endurance limit as well as a 100-MPa stress endurance limit, which means adhesive anchor shear connectors subjected to a slip range greater than 0.2 mm or a stress range more than 100 MPa may eventually fail under fatigue loading. Based on these data, the fatigue life of adhesive

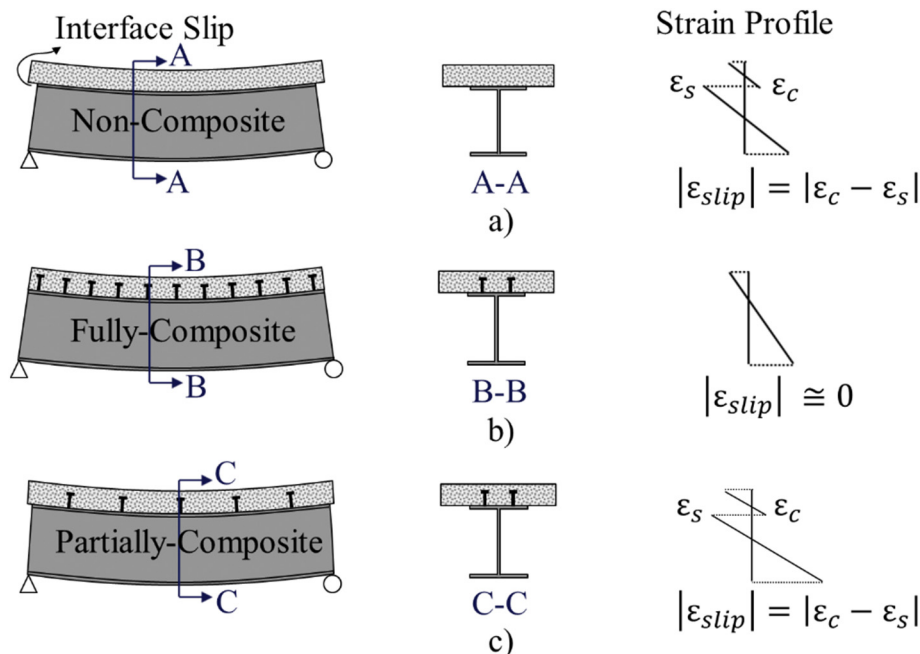


Fig. 1. Girders that are: a) non-composite, b) fully-composite, and c) partially-composite.

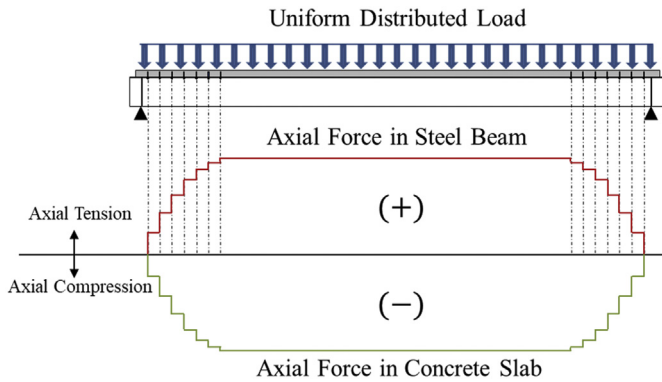


Fig. 2. Conceptual illustration of proctor's method.

anchor PISCs may still be inadequate for economical retrofit in some cases.

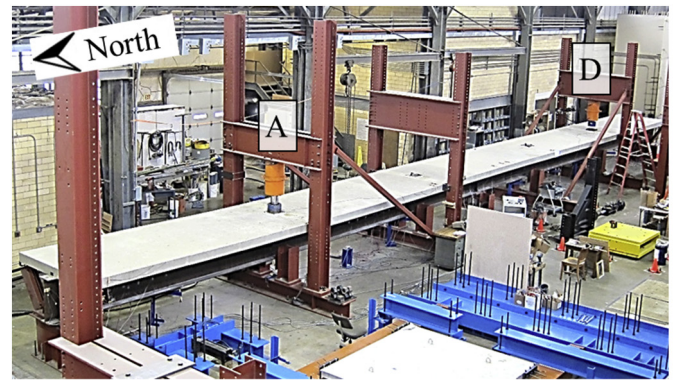
Because previous research [4,7,17,18] on conventional welded shear studs indicates that fatigue performance is better in large-scale beam tests than in small-scale component tests, large-scale beam fatigue tests were performed on adhesive anchor PISCs. Although there are some data on the fatigue life of adhesive anchors based on direct-shear tests, the actual fatigue performance of adhesive anchor PISCs in large-scale beam tests had not previously been investigated [6]. Direct-shear tests are less costly and faster to perform in comparison with large-scale beam tests and are therefore more commonly used. However, beam tests better represent the actual behavior of shear connectors in a bridge girder more accurately compared to direct-shear tests. The current paper describes the results of large-scale beam tests on the fatigue performance of adhesive anchor PISCs.

3. Experimental program

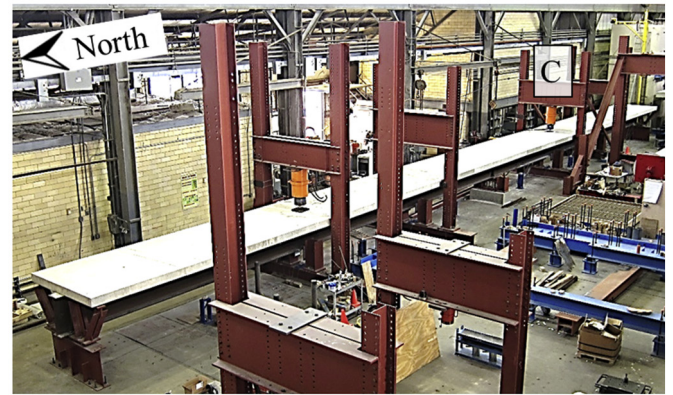
The fatigue performance of adhesive anchor PISCs was investigated through three different tests on two large-scale specimens. This testing was part of a larger project that also included elastic testing, large repeated loads requiring moment redistribution, and monotonic loading to failure. [12,13].

3.1. Test setup

Two large-scale girder specimens, referred to as Specimen No. 1 and Specimen No. 2, were constructed. Fig. 4 Shows a photo of both specimens, each of which consisted of a two-span continuous girder. The spans of the specimens are referred to as “north” and “south”, with reference to the north arrow shown in the figure. Not all the loading frames in these pictures were used for fatigue testing. The loading frames used



a)



b)

Fig. 4. Test setup: a) Specimen No. 1, and b) Specimen No. 2.

for fatigue testing are labeled as “A”, “C”, and “D”. Each load frame was used for one fatigue test. Therefore, a total of three fatigue tests were conducted.

3.2. Specimen details

Specimen No. 1 consisted of a two-span continuous girder with symmetric 12.8-m spans, and Specimen No. 2 was a two-span continuous girder with symmetric 15.9-m spans. These spans were selected so that the resulting span-to-depth ratios were similar to those of typical bridges that were candidates for this retrofit technique in Texas. Space limitations in the laboratory were another factor in determining the span length. Moreover, both specimens had a bolted splice in the steel girder, which was located 2.1 m away from the interior support in the south span of both specimens.

Fig. 5 shows the cross section of both specimens. ASTM A992 steel with a specified minimum yield stress of 345 MPa was used for the

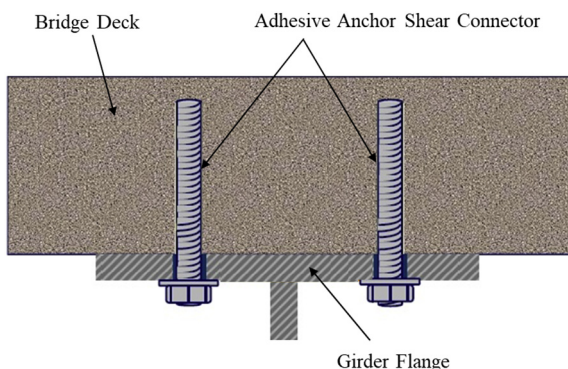


Fig. 3. Adhesive anchor shear connector tested by Patel [18], Kwon [14], and Kayir [9].

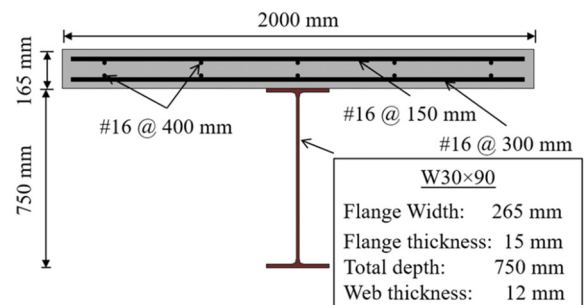


Fig. 5. Cross section of the specimens.

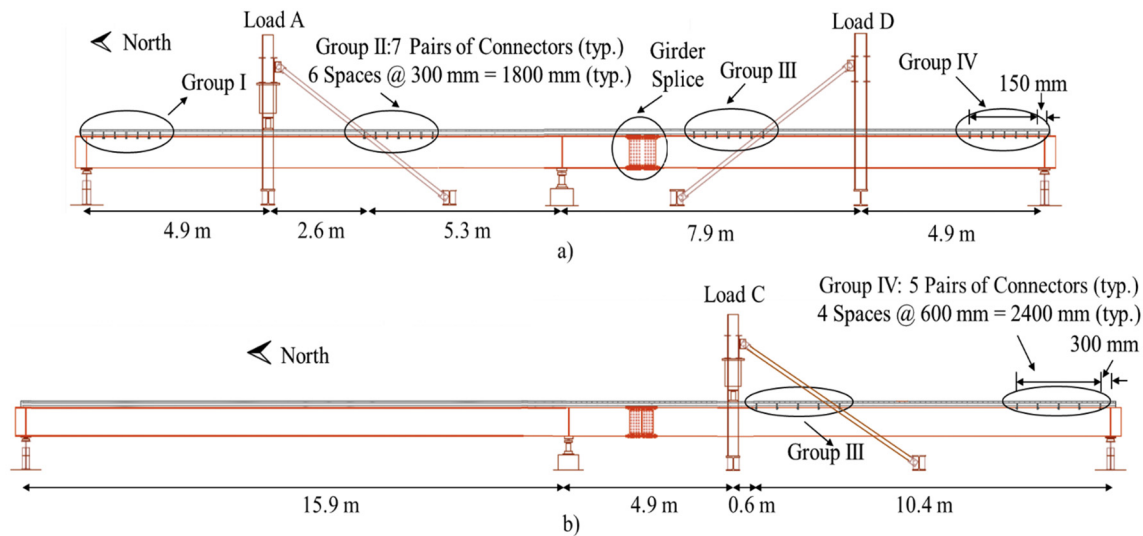


Fig. 6. Elevation view of the specimens: a) Specimen No. 1, and b) Specimen No. 2.

steel girders, and concrete with a specified compressive strength of 20 MPa was used for the deck.

Fig. 6 provides elevation views of the test setup for Specimen Nos. 1 and 2. Specimen No. 1 was strengthened with four groups of shear connectors, whereas Specimen No. 2 was strengthened with two groups of shear connectors in the south span only.

Each group of connectors had seven pairs of shear connectors in Specimen No. 1 and five pairs of shear connectors in Specimen No. 2, making Specimen No. 1 approximately 30% composite in both spans and Specimen No. 2 approximately 20% composite in the south span and non-composite in the north span. The shear connector groups are referred to as “Group I” through “Group IV”, as indicated in Fig. 6. Furthermore, load was applied at locations denoted “A”, “C”, and “D” for the various fatigue tests conducted, and diagonal braces were incorporated into the test setup to stabilize the load frames used for the fatigue tests.

3.3. Material properties

To characterize the material strengths of the components comprising the specimens, several material tests were conducted on the steel beam, concrete deck, reinforcing bars, and threaded rod used for the shear connectors. The results from these tests are summarized in Table 1 for the two specimens. Material tests were conducted following ASTM C39/39M-13 [2] and ASTM A370-13 [3]. For Specimen No. 1, the steel beams on either side of the splice were manufactured in different heats, and thus had somewhat different material properties. The entire length of the Specimen No. 2 was constructed of steel beams from the same heat as the south portion of the first specimen.

3.4. Instrumentation

Fig. 7 illustrates the instrumentation layout of the fatigue testing on the south span of Specimen No. 1, fatigue testing on the north span of Specimen No. 1, and fatigue testing on the south span of Specimen No. 2. Vertical deflections were measured at intervals of approximately 2.4 m to 3 m along the specimens. Slip measurements and strain profile measurements were generally conducted in the vicinity of the shear connectors to capture the behavior of these regions throughout the test program.

String and linear potentiometers with strokes ranging from 100 mm to 380 mm were used to measure vertical deflections along the specimens. Linear potentiometers (Fig. 8 (a)) with strokes of 50 mm and 25 mm, respectively, were used to measure slip, or relative movement of the underside of the concrete slab and the top of the steel girder. An 890-kN capacity load cell (Fig. 8 (b)) was placed between the piston of the hydraulic ram and a steel plate attached to the concrete deck using hydro-stone gypsum cement. To minimize slight alignment errors (especially as the girder deformed under the applied load) a load button, which is a relatively small steel cylinder with a slightly curved bottom surface was threaded into the bottom of the load cell which was threaded directly into the piston of the ram. At all support locations, pairs of 445-kN (end supports) or 2220-kN (interior support) load cells were sandwiched between steel plates to measure the reaction forces.

Strain gauges with a 6.35-mm gauge length were installed at discrete locations through the depth of the steel girder to evaluate the extent of composite action through monitoring of the neutral axis location. Strain gauges were placed on the top and bottom flanges on both south and north spans of each of the specimens. Strain gauges were also

Table 1
Material properties of the two specimens (units in MPa).

Specimen	Span	Beam flange ^a	Beam web ^a	Concrete ^b	Connector ^c	Reinforcing bar ^a
1	North	360 (495)	375 (500)	30	605 (925)	425 (675)
1	South	385 (535)	375 (540)	30	605 (925)	425 (675)
2	Both	385 (495)	375 (500)	20	605 (925)	425 (695)

^a Yield strength, ultimate strength in parentheses.

^b Compressive strength based on 28-day compression cylinder test.

^c Ultimate Shear strength, ultimate tensile strength in parentheses. (The data reported here represents the threaded rod of the connector and not the entire connector assembly with the epoxy in the concrete.)

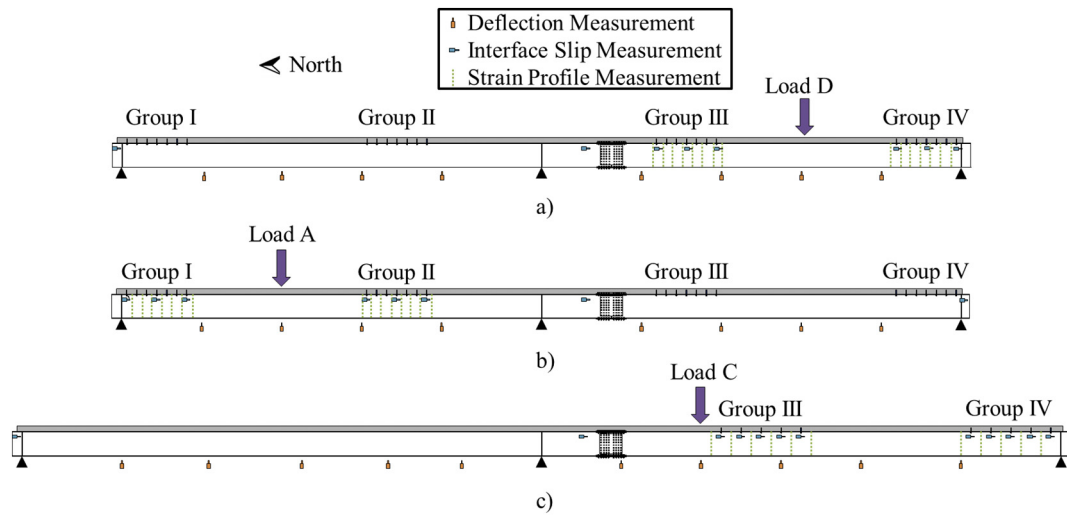


Fig. 7. Instrumentation: a) fatigue testing on the south span of Specimen No. 1, b) fatigue testing on the north span of Specimen No. 1, and c) fatigue testing on the south span of Specimen No. 2.

placed on either side of the web 240 mm above the bottom flange on the north span of Specimen No. 1.

In addition to tracking the locations of the neutral axis of the steel girder throughout the testing, strain gauges were used to estimate the shear stress in the PISCs. This was accomplished by drawing the strain profile of the steel girder on both sides of a connector pair and converting the strain profiles to equivalent axial force in the steel

girder on both sides of the connector pair. It was then assumed that the difference between the calculated axial force on both sides of the connector pair would be the shear force in the connector pair. This shear force was divided by twice the effective shear area of the adhesive anchor PISC, which was taken as 80% of the gross shear area [14], to obtain shear stress in a single connector. This method of estimating the shear stress in the PISCs is approximate, in that it

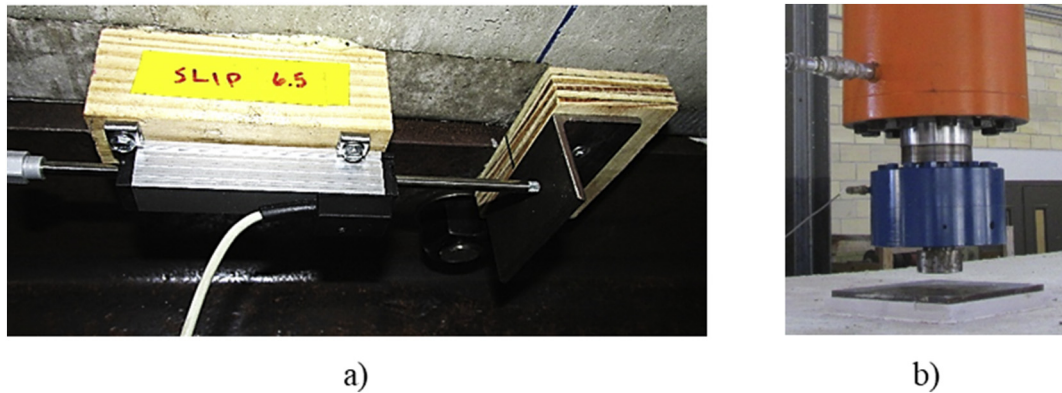


Fig. 8. Instrumentation: a) linear potentiometer as slip transducer, and b) 890-kN load cell.



Fig. 9. Materials used for adhesive anchors: a) threaded rods, and b) adhesive.

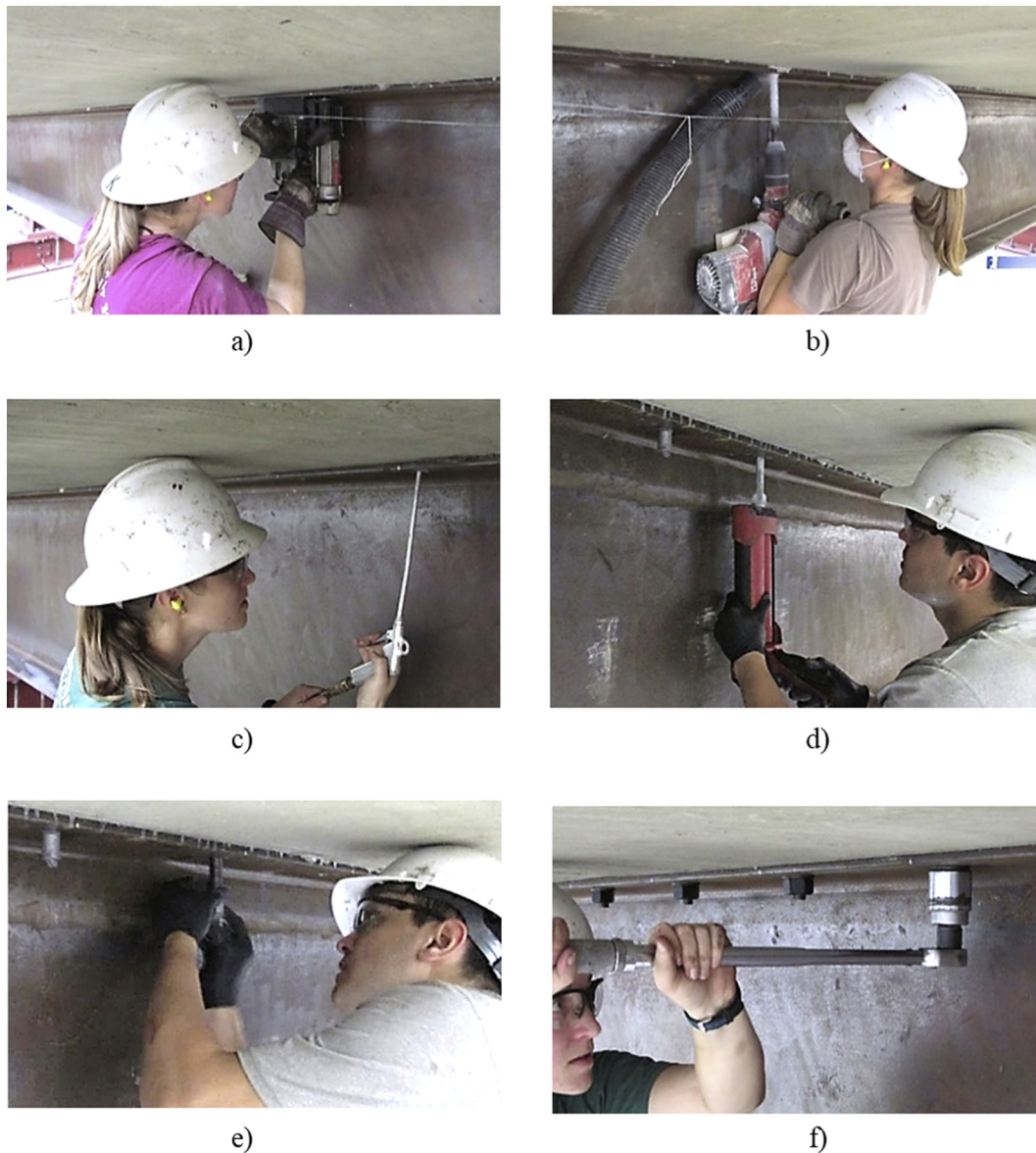


Fig. 10. Installation process of the adhesive anchor shear connector.

neglects any shear transfer due to friction between the steel girder and concrete slab.

3.5. Post-installing shear connectors

The adhesive anchor shear connectors were installed after the concrete deck was placed and cured. The connectors used in both specimens consisted of 22-mm diameter ASTM A193 B7 threaded rod with a corresponding structural nut and washer, and Hilti HIT-HY 200-R structural adhesive. The threaded rods were cut to approximately 170 mm long for a 115-mm embedment depth into the concrete deck. This resulted in a 50-mm top cover for the connectors as required by the AASHTO LRFD Specifications [1]. Before installation, the cut rods were lightly cleaned and degreased to improve adhesion. The two-part adhesive was injected using the compatible Hilti HDM 500 manual dispenser and mixer (Fig. 9).

Fig. 10 shows the installation process of the adhesive anchor shear connector. First, a 25-mm diameter hole was drilled through the top flange of the steel beam using a magnetic drill with an M2 HSS annular bit (Fig. 10 (a)). Next, a 24-mm diameter hole was drilled into the concrete deck to a depth of 115 mm, through the hole in the steel beam,

using a rotary hammer drill (Fig. 10 (b)). The resulting hole was then cleaned with an air hose and a brush as specified by the adhesive installation instructions (Fig. 10 (c)). After using compressed air to clean debris from the hole, a slightly over-sized round brush was inserted and removed twice using a twisting motion. Compressed air was used again to remove debris from the hole. After cleaning, the adhesive was injected into the hole (Fig. 10 (d)) and the connector rod was inserted (Fig. 10 (e)). The adhesive had enough viscosity to remain in the hole without running out and to keep the threaded rod in place immediately after insertion. Thus, the rod did not need to be held while the adhesive cured. The threads below the underside of the top flange were wrapped with duct tape to prevent any adhesive from reaching that area. Excess adhesive was wiped off immediately with a rag. After allowing the specified 1 h for the adhesive to cure, the tape was removed, and a washer and nut were placed on the threaded rod of the connector. The nut was then tightened to a torque of 170 kN-mm using a calibrated torque wrench, as specified by the adhesive installation instructions (Fig. 10 (f)).

The anchor was installed in accordance with the manufacturer's recommendations, with the exception that the 24-mm hole in the concrete was 1 mm smaller than that required by the manufacturer. This

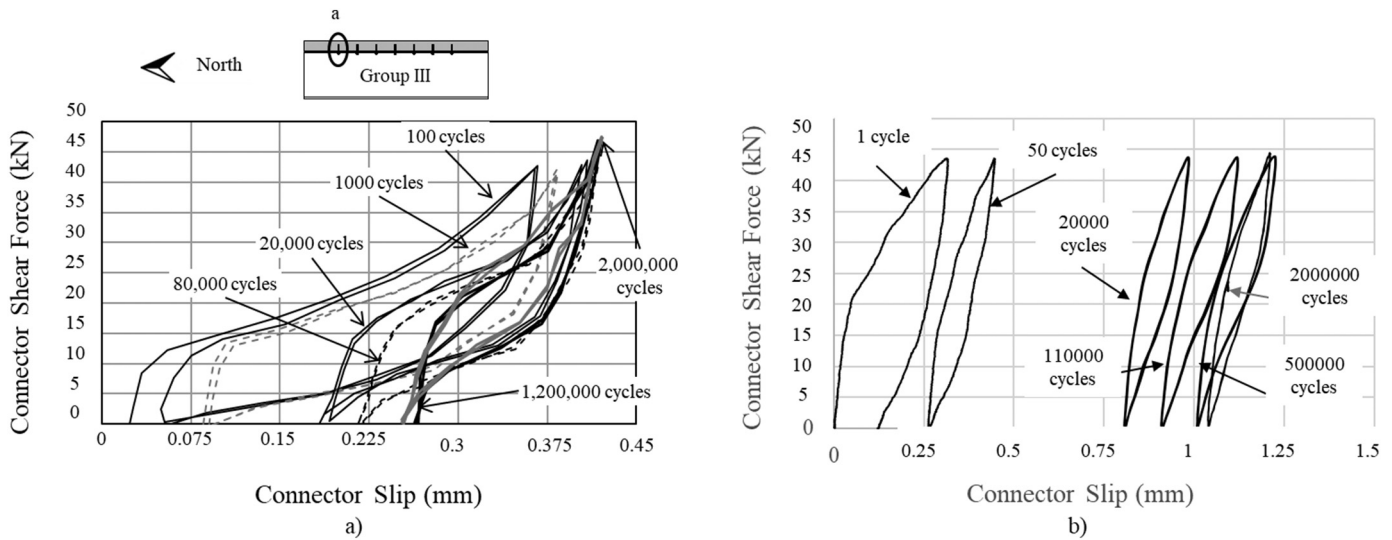


Fig. 11. Typical connector shear force-slip behavior: a) during south span fatigue testing of Specimen No. 1, and b) based on direct-shear tests by Patel [18]. (Note that the horizontal axes have different scales.)

modification was made to ensure the rotary hammer drill could properly fit in the 25-mm hole in the top flange of the steel beam.

3.6. Testing protocol

Data from the strain gauges, linear potentiometers, string potentiometers, and load cells were collected using an Agilent data acquisition system with 144 channels and LabVIEW software. While the fatigue tests were run 24 h a day using a servo-controlled hydraulic system, the tests were stopped generally at least once per day to collect data during one or two cycles in which the load was manually applied as a static load.

An extensive test program was developed for both specimens. As mentioned previously, the test program included high-cycle fatigue loading, low-cycle repeated loading into the inelastic range to investigate moment redistribution and the ability of the girders to achieve shakedown, and monotonic loading to failure. For each specimen, these tests were often performed separately in each of the two spans. Various testing sequences were used to evaluate the fatigue strength both before and after the repeated loading into the inelastic range,

which induced large forces in the connectors. Most importantly, the different sequence of loading for the two specimens was intended to clarify how important the loading sequence and simultaneous loading were in the performance of the specimens. For both spans of each specimen, the natural bond at the concrete-steel interface was broken through the application of relatively low loads prior to the installation of the connectors.

4. Test results

4.1. South span fatigue testing of Specimen No. 1

Fatigue testing was conducted on Specimen No. 1 by applying a load range of 220 kN at the location of load frame D (Fig. 6) which was intended to approximate the AASHTO HS-20 fatigue loading. This type of loading mainly engaged the connectors in groups III and IV in the south span of the girder. Within the south span, group III connectors were located nearest to the interior support, while group IV connectors were located nearest to the exterior support (Fig. 6). For each group, the

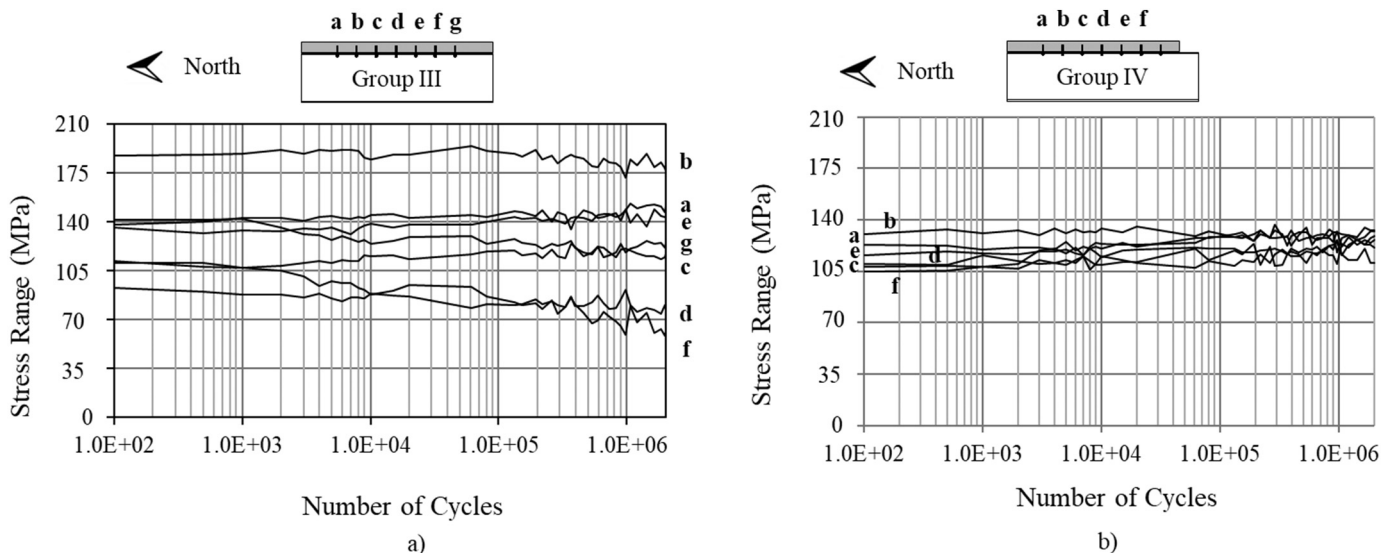


Fig. 12. Stress range variation of connectors in south span fatigue test of Specimen No. 1: a) group III, and b) group IV.

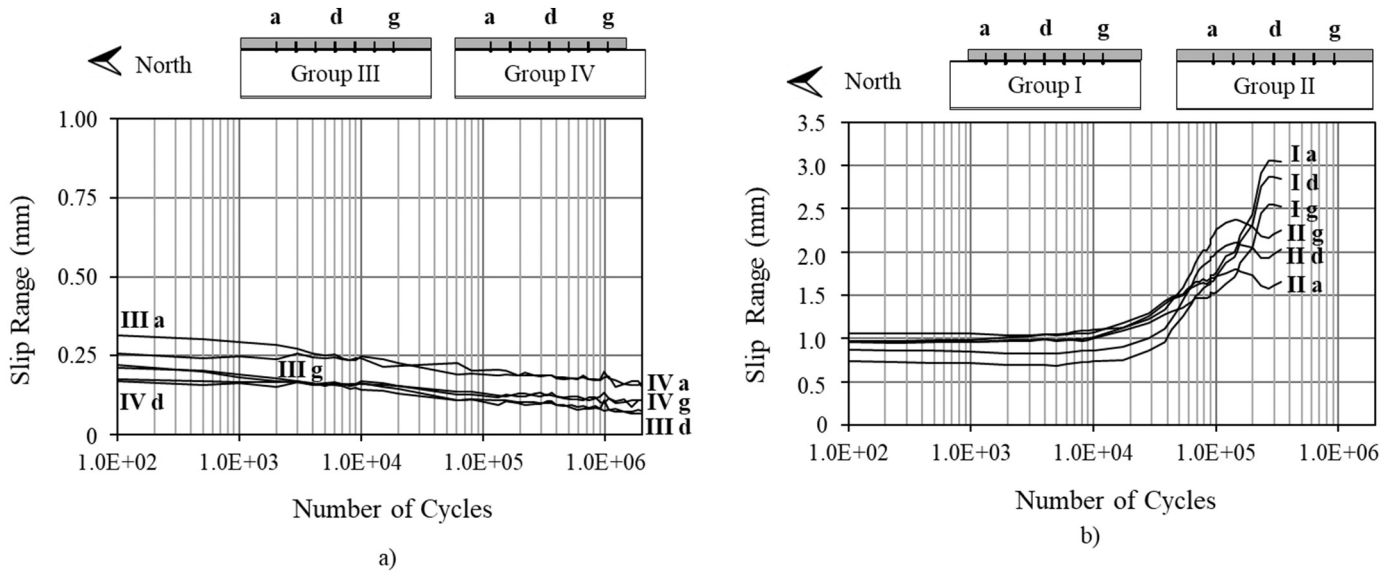


Fig. 13. Slip range variation during fatigue testing of Specimen No. 1: a) south span test, and b) north span test. (Note that the vertical axes have different scales.)

slip range of the first, middle, and last connector was monitored during fatigue loading.

Fig. 11 (a) shows the force-slip behavior of a single connector at several points during the fatigue test. These data are from the northernmost connector in group III (shown as connector “a” in Fig. 11), which was subjected to the largest slip range during the test. The decreasing slip range and approximately constant connector force with increasing number of cycles are both evident in the figure. However, the figure also indicates an increase in the residual slip, or the slip measured when the connector is unloaded. One possible explanation for this behavior is that the adhesive that fills the space around the threaded rod in the deck and in the top flange of the steel beam undergoes slight permanent deformations with each cycle.

The increase in residual slip was also observed during direct-shear tests performed by Patel [19]. However, the magnitude of the residual slip was three to four times higher after 2,000,000 cycles in the direct-shear tests in which the shear connector did not fail under a 45-kN force range applied as a direct-shear force on the connector (Fig. 11 (b)). Note that the horizontal axes in these figures do not have the

same scale so that the trends can be properly illustrated. This difference between the direct-shear tests and large-scale test suggests that the shear connectors in an actual bridge may perform much better in fatigue than what might be predicted by direct-shear tests. In direct-shear tests, nothing prevents the plate from sliding relative to the slab except for the single shear connector. In beam tests, however, slip is caused by beam deflection and the maximum slip for each connector has an upper bound of the slip for the non-composite condition. Moreover, compatibility between slip at other connectors is required. Nevertheless, both beam tests and direct-shear tests confirmed the fact that most of the residual slip occurs in the beginning of fatigue loading and the residual slip increases by a smaller amount after each cycle. For instance, in both Fig. 11 (a) and (b), more than 75% of the total residual slip at the end of two million cycles occurred after only 20,000 cycles, whereas the residual slip increased very slowly from 20,000 cycles to 2,000,000 cycles.

Fig. 12 (a) and (b) show the variation in the estimated stress range with number of load cycles for all connectors in groups III and IV, respectively. Contrary to the slip range, the stress range remained almost

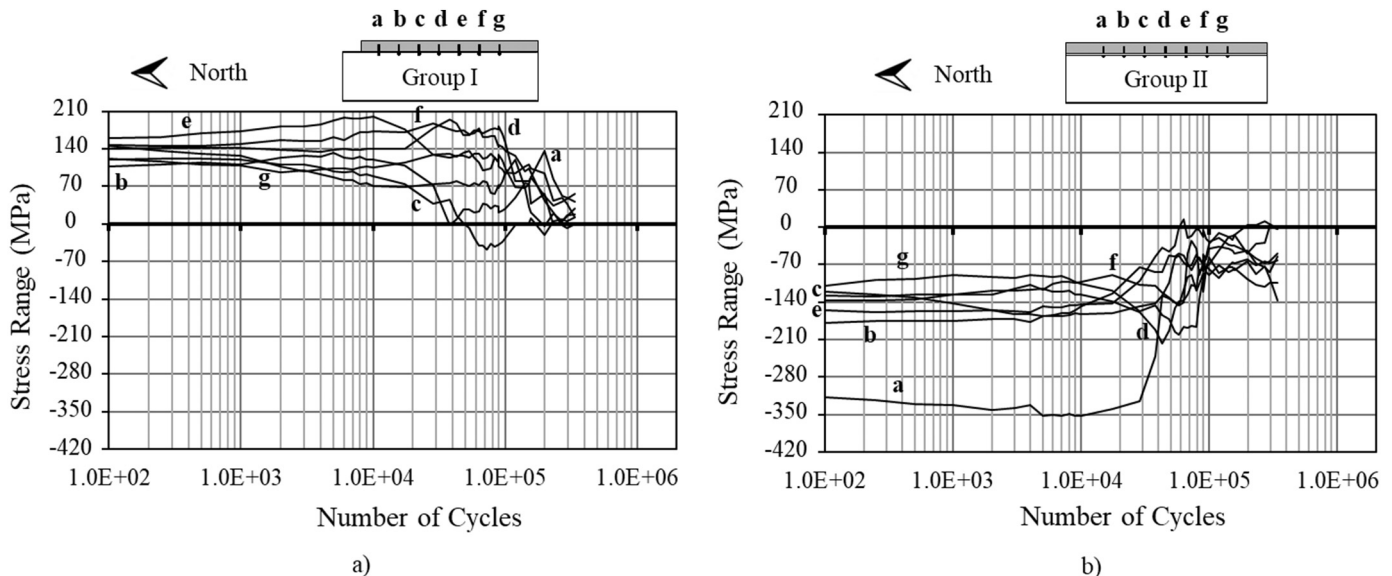


Fig. 14. Stress range variation of shear connectors during north span fatigue test for Specimen No. 1: a) group I, and b) group II.

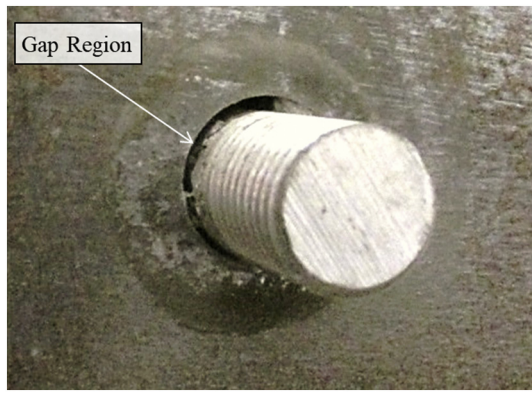


Fig. 15. Visual observation of adhesive degradation through the gap around connector.

constant throughout the testing for all the connectors. Coupled with the observed decrease in slip range, this result suggests an increase in the connector stiffness as the test progressed. One possible explanation for this phenomenon is that as the adhesive surrounding the connector degraded, the connector shifted towards one side of the hole as indicated by the residual slip values and compressed the adhesive, which might have made the adhesive appear to respond with greater stiffness.

As the fatigue test progressed, a reduction in the measured slip range was observed for constant cycles of the same magnitude of applied load. After 2,000,000 cycles and no connector failures, a 50% reduction in the slip range was observed for almost all the connectors. This phenomenon is illustrated in Fig. 13 (a) for three connectors in each of groups III and IV in the south span. Furthermore, an increase in the residual slip of the shear connectors was observed.

These results indicate a significant difference from direct-shear fatigue test results reported previously by Patel [19], Kwon [14] and Kayir [9] where the slip range remained essentially constant during the entire fatigue life for tests in which the connector was loaded in only one direction. These direct-shear tests also indicated a fatigue slip range endurance limit of 0.2 mm, suggesting that connectors subjected to slip ranges lower than this limit have effectively an infinite fatigue life. Because of the significant reduction in slip range with cycles in the large-scale tests, it is difficult to evaluate this proposed endurance limit. However, the results of the large-scale beam tests imply that the actual fatigue slip range endurance limit of the adhesive anchor shear

connector is most likely much higher than the reported 0.2-mm limit derived from the direct-shear tests.

4.2. North span fatigue testing of Specimen No. 1

Once the fatigue testing in the south span was complete, fatigue testing in the north span began. Inelastic testing, which loaded the partially-composite girder well into the inelastic range, had been conducted in the north span prior to fatigue testing. While residual slips of up to 1.5 mm were accumulated in the north span connectors, no connector failures were observed during the inelastic testing [10]. It is important to note that the 1.5 mm value for residual slip is more than three times the maximum connector slip observed during the south span fatigue test of the first specimen. Because of the very good fatigue performance of the south span connectors under a 220-kN load range, project researchers decided to increase the load range to 335 kN for fatigue testing in the north span, which was conducted using the Load A configuration. This 335-kN load range applied slip and force demands on the shear connectors approximately two to three times greater than what would be expected from the AASHTO HS20 fatigue loading.

Fig. 13 (b) shows the slip range variation of the connectors in the north span fatigue testing of Specimen No. 1. Note that in Fig. 13, the vertical axis of the two graphs do not have the same range so that the decreasing trend of slip range in Fig. 13 (a) can be properly illustrated. Fig. 14 (a) and (b) show the stress range variation with the number of cycles of load for fatigue testing in the north span of Specimen No. 1.

Note that very different trends were observed in this test compared to the south span test. The slip range remained almost constant through approximately 10,000 cycles and then increased with the number of cycles to large values. Note that the corresponding slip values for the non-composite specimen would be much larger in the region of group I connectors compared to group II, and this may explain the different increasing trends in slip between group I and group II connectors. Also, rather than remaining constant throughout the entire test, the connector forces began to vary greatly with a general trend of approaching zero as the number of cycles increased beyond 10,000 cycles. The adhesive in connector “a” in group II was the least damaged from prior inelastic testing based on the initial slip range values of the connectors as observed in Fig. 13 (b). Therefore, this connector was able to take significantly more force initially than the other connectors in this fatigue testing.

This very different behavior compared to the south span test can be explained by degradation of the adhesive around the threaded rod in

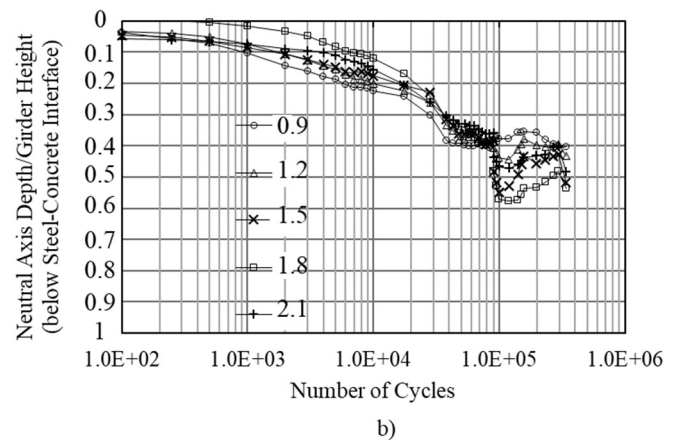
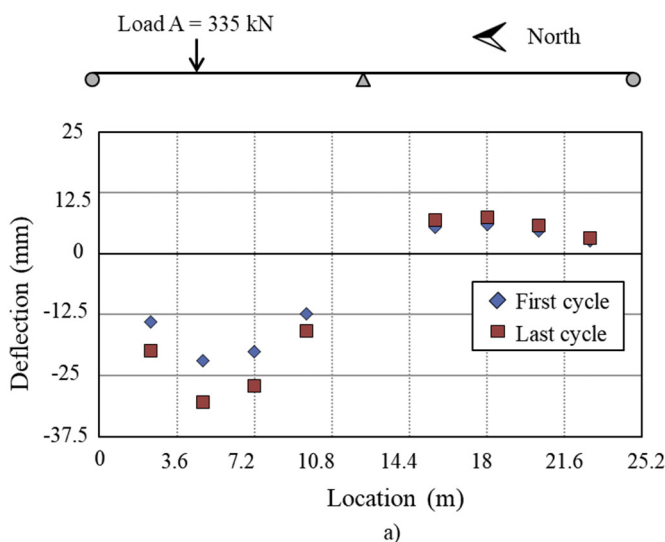


Fig. 16. North span fatigue test of Specimen No. 1: a) deflected shape of the specimen, and b) neutral axis depth vs. number of cycles at 0.9 m, 1.2 m, 1.5 m, 1.8 m, and 2.1 m away from the north support.

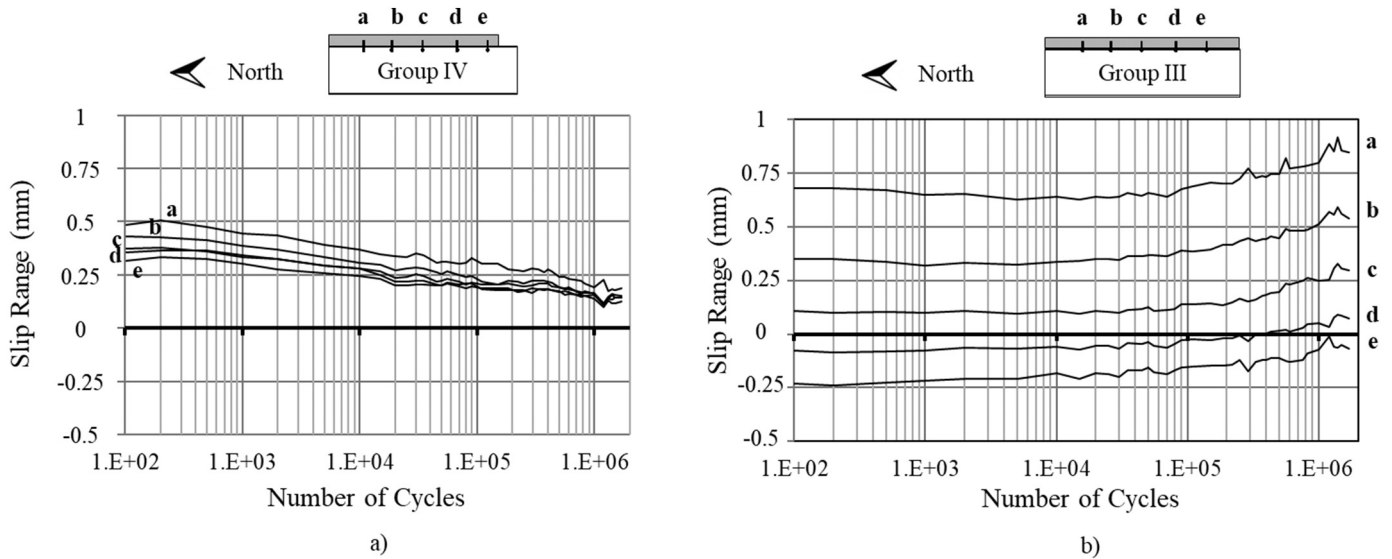


Fig. 17. Slip range variation of connectors during the fatigue testing of Specimen No. 2: a) group IV, and b) group III.

the top flange of the steel beam, a phenomenon that was also observed in the direct-shear fatigue testing. As the north span test progressed, the adhesive in the annulus between the connector and the top flange of the steel beam slowly disintegrated into a powder and accumulated on the bottom flange of the steel beam, effectively forming a “gap” region, through which the threaded rod of the connector could slip without coming into bearing on the flange of the steel girder. Fig. 15 shows the gap region of a north span connector in a photograph taken after all testing was complete. The maximum size of this gap region due to adhesive degradation was roughly 3 mm, which is the difference between the 22-mm diameter of the threaded rod and the 25-mm diameter of the hole. The maximum slip range observed by the end of the north span fatigue test reached nearly this value.

Because the degradation took place more quickly in some connectors than in others, the load distribution among the connectors changed consistently during the test, as illustrated in the stress range variation in Fig. 14 (a) and (b). These figures also suggest the specimen behaved almost like a non-composite specimen because the shear connectors took little force after 300,000 cycles. This observation was also verified with deflection measurements and strain profile measurements.

As the adhesive degraded and the connectors were able to slip without transmitting any force, the behavior of the girder trended towards what would be expected for a non-composite girder. This response is illustrated in Fig. 16 (a), which compares the measured girder deflections near the beginning and end of the fatigue test. The north span fatigue test was stopped after approximately 330,000 cycles because the connectors were taking very little force, and the girder was exhibiting essentially non-composite behavior.

To verify the non-composite behavior of the specimen as the fatigue test progressed, the neutral axis depth at different locations was determined from strain gauge readings. As illustrated in Fig. 16 (b), the neutral axis depth moved towards the mid-depth of the steel girder as the fatigue test progressed, thereby verifying the non-composite behavior of the specimen. It is important to note that the initial almost fully-composite behavior as observed in Fig. 16 (b) is because the specimen had seven pairs of shear connectors in the locations where neutral axis measurements were taken; therefore, the specimen exhibited a behavior closer to that of a fully-composite beam in these locations.

The observed slip behavior may be quite beneficial, as it reduces the fatigue demands on the shear connectors, thereby extending their

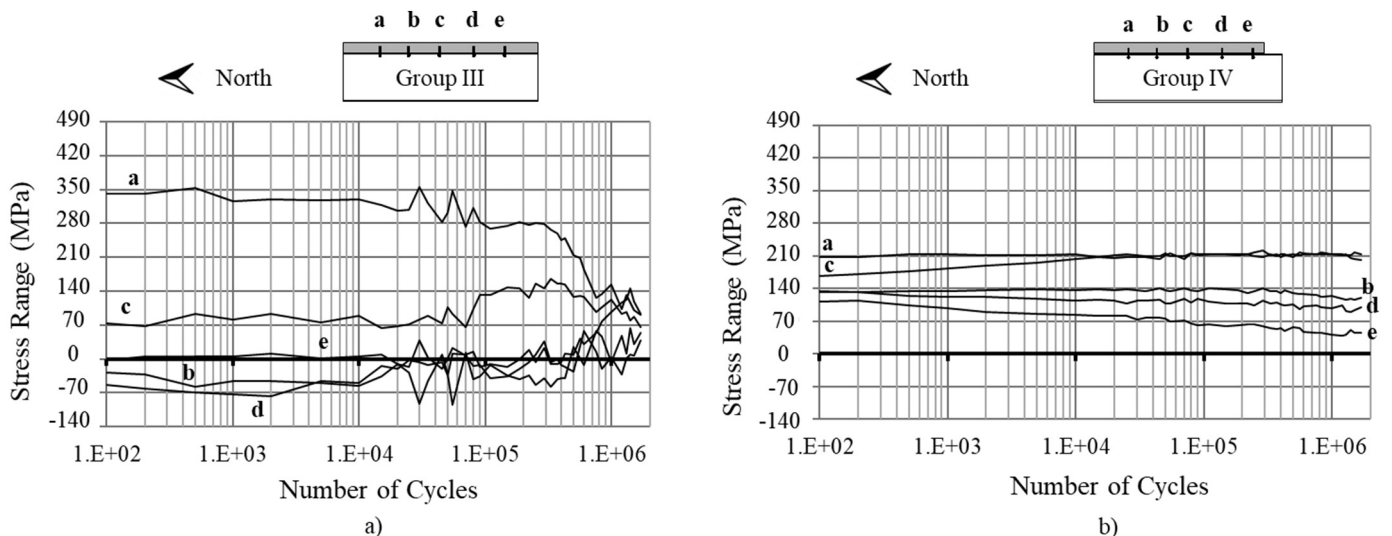


Fig. 18. a) Stress range variation of connectors during fatigue testing of Specimen No. 2: a) group III, and b) group IV.

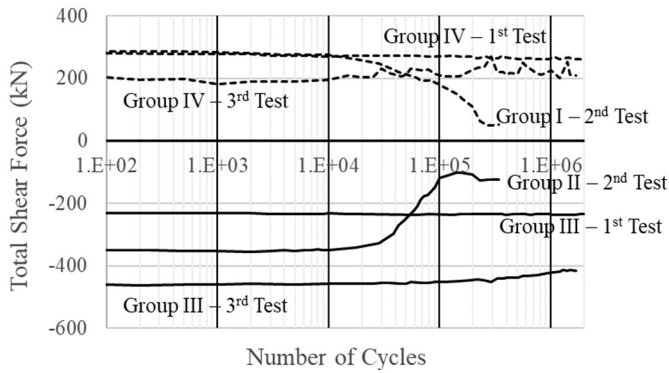


Fig. 19. Variation of total shear with number of cycles for all groups of connectors for all fatigue tests.

fatigue life [6]. Rather than repeatedly stressing the threaded rods until fatigue-induced fracture occurs, the adhesive degradation softens the shear connector and effectively reduces the stress range the connectors must resist. As observed in the ultimate strength testing results [6], the connectors can slip through the gap region and re-engage at large loads so that the composite strength can be developed if necessary. Thus, ultimate strength is not negatively influenced by degradation of the adhesive.

4.3. Fatigue testing of Specimen No. 2

The fatigue testing of Specimen No. 2 started prior to any inelastic testing on this specimen. Because no shear connector failure was observed during fatigue testing of Specimen No. 1, project researchers decided to increase the demand on the shear connectors in Specimen No. 2 even further to see if a connector failure could be observed during fatigue testing. Therefore, one of the spans (south span) was designed to be 20% composite (five pairs of connectors instead of seven pairs in the first specimen), and the other (north span) was left non-composite for the duration of the fatigue testing. Because Specimen No. 2 had longer spans with fewer shear connectors, much larger slip and stress demands were expected on the shear connectors.

Because both fatigue tests on Specimen No. 1 were conducted by placing the load between the two groups of connectors in each span, it became of interest to investigate what would happen if the load was

closer to the middle support, causing slip in opposite directions within the group. Therefore, load frame C was selected for the fatigue testing of the south span of Specimen No. 2.

The applied load range was chosen such that the maximum slip range measured in the connectors would exceed the maximum tested by Patel [18] in the direct-shear tests. Applying a load of 335 kN at load frame C resulted in 0.81 mm of slip measured in the northernmost connector (connector “a”, which was the nearest to the interior support). This slip range was significantly higher than the 0.64 mm maximum slip range tested in the direct-shear tests by Patel [19]. Therefore, fatigue testing of Specimen No. 2 was conducted under a 335-kN load range using load frame C, which induced slip demands roughly twice as large as that induced by AASHTO HS20 fatigue loading. After 1,700,000 cycles and no signs of a fatigue failure in the shear connectors, the fatigue test was stopped.

Fig. 17 (a) shows how the slip range of the group IV connectors (located at 28.5 to 30.9 m away from the north support every 0.6 m) throughout the fatigue test. The trends in this figure indicate that all the group IV connectors had a 50% (or more) reduction in the slip range by the end of the fatigue test. This trend is quite similar to that of Fig. 13 (a), which illustrated the slip range variation of the connectors in the south span fatigue testing of Specimen No. 1. This similarity was expected because both figures represented test results prior to any inelastic testing on the corresponding span.

Despite the agreement in the trends between the slip range results for Specimen No. 1 and those of the group IV connectors in Specimen No. 2, the slip range results of group III connectors (located at 21 m to 23.4 m away from the north support every 0.6 m) indicate a different type of behavior. Fig. 17 (b) illustrates how the slip range in these connectors changed throughout the fatigue testing of Specimen No. 2. As illustrated in the figure, the group III connectors experienced a slight reduction in the slip range initially. However, the slip range became nearly constant as the test progressed. After approximately 100,000 cycles, the slip ranges of connectors “a”, “b”, and “c” started to increase, whereas the slip range of connectors “d” and “e” started to decrease. The negative sign in the slip range of connectors “d” and “e” means that these connectors were slipping in the opposite direction of connectors “a”, “b”, and “c”. This behavior, which was not observed previously, can be attributed to the fact that the point of zero-slip changed as the test progressed (Fig. 17 (b)) resulting in force redistribution among these connectors.

To evaluate the hypothesis of force redistribution among the connectors, it was necessary to look at the connector stress range variation

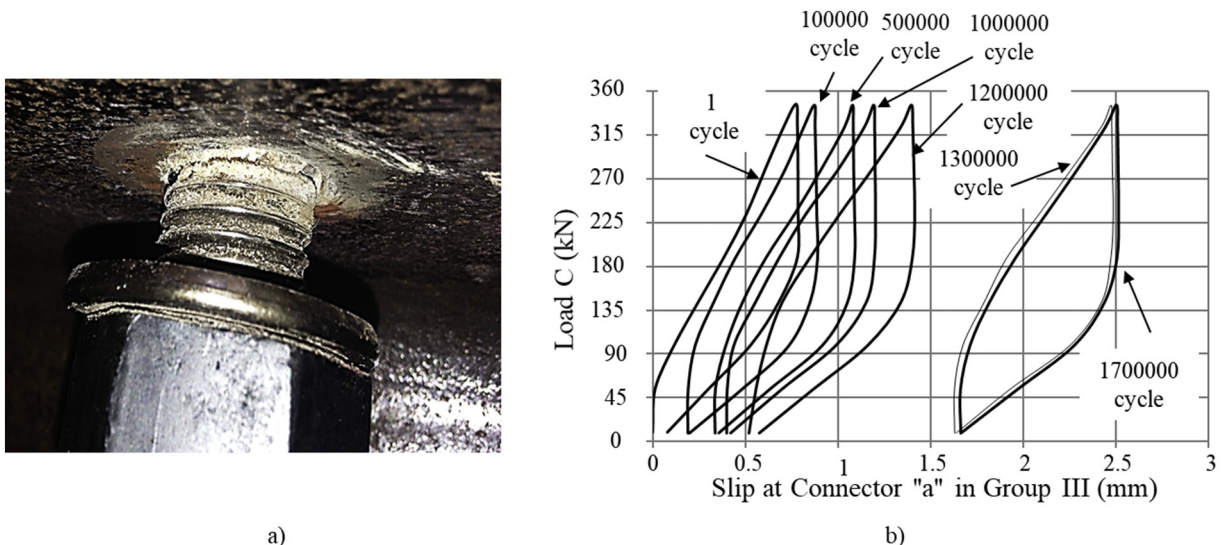


Fig. 20. Fatigue testing of Specimen No. 2: a) degraded adhesive in the connector, and b) load C vs. connector slip.

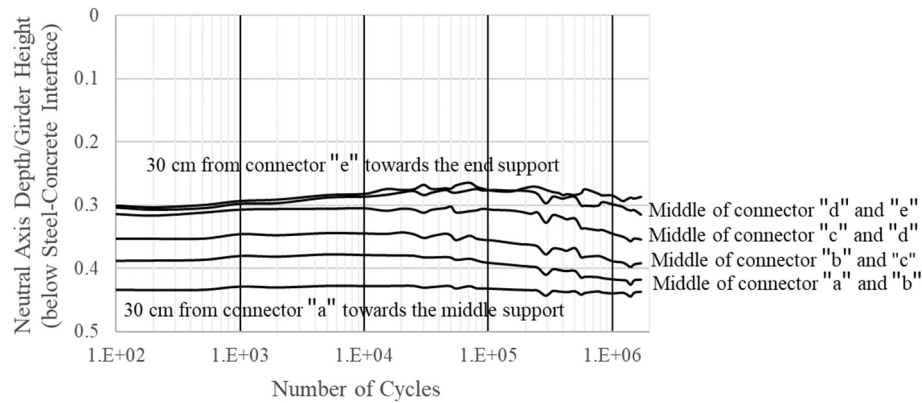


Fig. 21. Neutral axis depth vs. number of cycles during fatigue testing of Specimen No. 2.

as the fatigue test progressed. Fig. 18 (a) and (b) show the stress range variation for connectors in groups III and IV, respectively.

Fig. 18 (a) shows that the group III connectors have redistributed the total force acting on the group and that each connector carried approximately a 70-MPa shear stress at the end of the fatigue test. Note that connectors “a” and “b” are within a distance d (overall beam depth) from the applied load and this may affect the response of the connectors. This positioning may also explain the significantly higher stress values in connector “a”. As shown in Fig. 18 (b), the group IV connectors had a fairly constant stress range (similar to the results in Fig. 12 (a) and (b) for Specimen No. 1) with the exception that some force redistribution between the middle shear connector and the end shear connector (connectors “c” and “e” in the figure) can most likely be deduced from the trends in this figure based on the fact that connector “c” exhibited the most increase in the stress range and connector “e” exhibited the most decrease in the stress range initially.

Fig. 19 shows how the total shear force changed in groups of connectors for all the three tests as the fatigue test progressed. The total shear force remained almost constant throughout all the fatigue tests (except for the second test where the specimen started behaving like a non-composite specimen after 10,000 cycles) which suggests that if a connector in a group took less force, the other connectors in the group would have to carry the force that was originally being carried by that connector.

The total shear force in the two different groups of connectors in each test would have the same magnitude with a different direction based on horizontal equilibrium assuming that there is no friction at the interface. This is relatively true for the first and second fatigue tests. For the third fatigue test however, the difference between the magnitude of total shear force in group III and group IV connectors is significant which can be attributed to the fact shear connectors “a” and “b” in group III connectors were located at a distance less than the beam overall depth to the loading point.

Because the stress range variation in the connectors implied that force redistribution was taking place, it became necessary to investigate if the connectors had lost stiffness. This investigation was conducted by removing the washer and nut in the northernmost connector (connector “a”). This connector had the highest slip demand and had redistributed the most force based on Fig. 18 (a).

Fig. 20 (a) shows connector “a” after the washer and nut were backed off from the flange of the girder. As seen in this figure, the adhesive in the shear connector had degraded substantially, supporting the force redistribution hypothesis. The adhesive degradation was also clear from the slip measurements. Fig. 20 (b) shows the applied load (Load C) versus slip at the location of the northernmost connector where the adhesive degradation was physically observed. The residual slip at the end of the test (1.7 mm) was roughly six times larger than the residual slips observed during fatigue testing on the south span of Specimen No. 1. Such a large residual slip indicates that the adhesive

is significantly degraded. Furthermore, most of this degradation took place between 1,200,000 cycles and 1,300,000 cycles.

Because the adhesive in some of the connectors degraded, it is possible that the specimen could have started behaving like a non-composite girder, as was the case in the north span testing of Specimen No. 1. However, this proved not to be the case as the deflection range of the specimen appeared to be nearly constant. This result is not consistent with that of Specimen No. 1 under north span fatigue testing, where the deflection range started to increase as the fatigue test progressed and approached that of non-composite girder. Fig. 21 shows how the neutral axis depth (normalized with respect to the girder height from the steel-concrete interface) changed as the test progressed. As this figure shows, the neutral axis around connector locations did not approach the mid-height of the girder, indicating that the specimen was still behaving like a partially-composite beam. Note that between connector “a” and all the way to the north support, there were no shear connectors. Therefore, it was expected that the specimen would have a behavior close to that of a non-composite beam at 30 cm away from connector “a”, towards the middle support.

5. Summary and conclusions

Generally, the adhesive anchor shear connectors behaved well in fatigue in the large-scale beam tests, with no fatigue failures occurring during any of the three fatigue tests. In the south span test of Specimen No. 1, which was conducted under a load range that placed similar force and slip demands on the connectors as would be expected in a typical strengthened bridge, consistent behavior was observed throughout 2,000,000 cycles of load despite a gradual increase in residual slip and a gradual decrease in slip range. During the north span test of Specimen No. 1, which was conducted at a load range 50% greater than that of the south span test and after inelastic testing of the north span, degradation of the adhesive between the threaded rod and the top flange of the steel beam was observed in all connectors. This degradation led to the formation of a gap region, allowing the connectors to slip without transmitting force. Different behavior was observed throughout 330,000 cycles of load with a sudden increase in both residual slip and slip range of connectors, as well as an abrupt decrease in the stress range of connectors, after approximately 10,000 cycles. With increasing cycles of load, the girder showed non-composite behavior.

The adhesive degradation observed in this study is not necessarily detrimental to the bridge performance. In fact, the adhesive degradation appears to be beneficial. Rather than repeatedly stressing the threaded rods until fatigue-induced fracture occurs, the adhesive degradation softens the shear connector and reduces the force in the connectors. As observed in the ultimate strength testing results, the connectors can slip through the gap region and re-engage at large loads so that the composite strength can be developed if necessary. Thus, ultimate strength is not negatively influenced by adhesive degradation.

The adhesive degradation does not necessarily indicate loss of composite action. The results of this study showed that adhesive degradation due to large repeated loads requiring moment redistribution in the inelastic region can cause loss of composite action under fatigue loading while adhesive degradation due to large fatigue loads in the elastic region does not cause the composite action to fail.

The fatigue tests conducted in this research indicate that the fatigue behavior of adhesive anchor PISCs is expected to be better in an actual bridge than that suggested by results of direct-shear tests. There are four possible explanations for this difference: 1) Connectors work together in a composite girder and can redistribute force among them; 2) contrary to component tests where the shear force is applied directly, in large-scale beam testing the shear force in the connectors is applied indirectly through beam deflection and interface slip; 3) adhesive degradation causes the connectors to slip without taking any significant force in large-scale beam tests, which allows for force redistribution away from connectors with degraded adhesive. In component tests, which are force-controlled, a certain shear force must be transmitted through the shear connector regardless of the formation of a gap region; and 4) in beam tests, the connector slip is driven by deflection of the girder such that the applied load has a more indirect influence on the slip compared to direct-shear tests. The degradation of adhesive in the most highly-loaded connectors leads to force redistribution that prolongs the fatigue life of the group of shear connectors. Force redistribution is not possible in the direct-shear tests with one connector. This can also be why the slip endurance limit from the direct-shear tests did not correlate well to the beam tests.

The fact that the shear connectors did not fail in fatigue in Specimen No. 2 indicated that the results from the direct-shear testing conducted previously by Kayir [9], Kwon [14], and Patel [19] on the fatigue life of adhesive anchor shear connector was conservative as the shear connectors did not fail in fatigue even when subjected to slip ranges higher than what was tested previously in direct-shear testing. Based on these direct-shear tests, the connectors with slip ranges above 0.64 mm or stress ranges above 350 MPa would have been expected to fail in less than 50,000 cycles, which in comparison with no failure through 1,700,000 cycles indicates how conservative the direct-shear tests can be in predicting the fatigue life of post-installed shear connectors in an actual bridge.

Moreover, the component tests appear to be conservative for fatigue design purposes because no shear connector failure was observed in any of the large-scale beam tests. All connectors exhibited significantly longer fatigue lives in the large-scale beam tests than what would be expected by comparing the force and slip demands to the results of the direct-shear testing. Additional study is needed to relate component tests to large-scale beam tests and predict the fatigue life of PISCs more accurately.

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