FATIGUE RESISTANCE OF CLUSTERED SHEAR STUDS

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BIOGRAPHY

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SUMMARY

Accelerated bridge construction (ABC) is an increasingly common bridge construction technique in which large modular bridge elements are fabricated off-site and then are connected on-site to form the bridge. One such technique is the use of full-depth precast concrete deck panels placed on top of steel girders and connected via shear studs. The panels typically have pockets to fit around the shear studs, which are then filled with grout to form a composite connection with the girder. When using precast concrete panels, it is advantageous to cluster shear studs closer together and increase the distance between these clusters. Reducing the number of pockets in the panels simplifies panel fabrication and constructability.

This paper describes large- and small-scale fatigue tests constructed with steel beams and precast concrete deck panels. The shear stud configurations on the large-scale tests range from a more typical detail with studs spaced every 12 or 24 inches, to a detail more conducive to precast panels with cluster spacings of 36 or 48 in., which are currently not permitted by AASHTO. The small-scale tests are similar to other historical tests on shear studs will provide a comparison to the large-scale tests. The testing described herein will be used to evaluate the current LRFD shear stud provisions for the fatigue limit state as well as the shear stud spacing limits to recommend if either can be adjusted to more effectively accommodate the use of precast deck panels.
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Introduction

A common accelerated bridge construction practice is the use of large prefabricated bridge elements, which are fabricated off-site and then connected together on-site to construct the bridge. One such construction method of this practice is using full-depth precast concrete deck panels placed on top of steel girders and connected with shear studs. The concrete panels typically have pockets, which are formed to fit around the shear studs. These pockets are then filled with grout in the field to form a composite connection between the concrete deck and the steel girder.

On a typical bridge with a conventional cast in place deck, shear studs are spaced in regular intervals along the length of a girder, with a maximum longitudinal spacing of 24 inches according to the AASHTO LRFD Bridge Design Specifications (1). When precast concrete panels are used, it is advantageous to place individual shear studs close together in clusters and increase the distance between these clusters. This reduces the number of pockets in the deck panels, which helps to simplify panel fabrication and constructability.

To provide full composite action between the concrete deck and steel girder, shear studs must be designed for the fatigue and strength limit states (1). For short spans (ie. 120’ or less) and near the supports, the fatigue limit state typically governs the number of studs and can require a significantly larger number of studs than the strength limit state. Depending on the length of the span, a large number of shear studs can lead to a very small longitudinal spacing between studs, making it difficult to use precast deck panels. The current AASHTO LRFD shear stud fatigue specifications will be compared to three international shear stud fatigue design provisions to determine if there are opportunities for improvement to better accommodate precast decks.

Shear Stud Fatigue Design Provisions

The current AASHTO shear stud fatigue design provisions are based on 44 small-scale tests conducted by Slutter and Fisher (1966). These tests, called push out tests, are a common alternative to large-scale fatigue testing of shear studs. A diagram of Slutter’s and Fisher’s push out specimens are shown in Figure 1. In these tests, studs were welded onto one side of the steel beam, resulting in a “one face” push out test. Both 3/4 and 7/8 in. diameter shear studs were used in these tests (2). The small-scale test results were compared to beam tests (3 and 4) and the lower limit of dispersion (taken as twice the standard error of estimate) of the beam tests was approximately equal to the mean behavior of the push out tests. Therefore, the mean results from the push out tests were used to develop shear stud fatigue design equations (2).

A constant amplitude fatigue limit (CAFL) of 7.0 ksi was later added to the AASHTO Specifications in 1977 (5), though no test results were cited for this addition, to produce the provisions that are currently being used. The AASHTO design fatigue resistance of a single shear stud is expressed in terms of a shear force range, \( Z_r \) (in kips), rather than a stress range which is more common to the other AASHTO fatigue details, and is determined using the following equations (1):

For Fatigue I (infinite life):

\[
Z_r = 5.5d^2
\]  

(1)

For Fatigue II (finite life):

\[
Z_r = ad^2
\]  

(2)

in which,
\[ \alpha = 34.5 - 4.28 \log N \]  

(3)

where, \( d \) = shear stud diam. (in.), and \( N \) = number of cycles.

When compared to the international shear stud fatigue design equations, the AASHTO provisions are quite different. Figure 2 shows the stud fatigue design curves according to AASHTO (1), the Eurocode 4 (6), Australian Standard (7), and the Japan Society of Civil Engineers (JSCE) (8). The AASHTO fatigue detail Category C is also included in the figure for reference. Since the JSCE design equation depends on various geometric and material properties, the following typical values were assumed: shear stud diameter and height of 7/8” and 6”, respectively, and a concrete compressive design strength of 4.0 ksi.

![Figure 2. Shear stud S-N design provisions from AASHTO and various international codes](image)

As shown in Figure 2, the AASHTO stud fatigue curve is in a semi-log format, while the other three specifications follow a log-log format, with slopes ranging from -8 to -9.5. These slopes are much shallower than the slope of -3 for the typical AASHTO fatigue details. This difference in slope is expected due to the different types of stress ranges used for these different detail types: when designing shear studs, shear stresses are used, while typical AASHTO details are designed based on normal stresses. Since the AASHTO stud curve is semi-log, the fatigue design stress range decreases more rapidly as the number of cycles increase. This is clearly evident at approximately 6.0 million cycles, where the infinite life fatigue equation controls, and where most new bridges are likely designed. The AASHTO stud CAFL of 7.0 ksi also appears to be generally conservative since the international provisions do not have CAFLs for shear studs.

**Research Objectives**

Based on a review of the current domestic and international shear stud specifications, it is clear that the AASHTO provisions warrant further examination. The shear stud fatigue design provisions appear too conservative, thereby limiting the potential benefits of using precast concrete deck panels. This paper will discuss large- and small-scale fatigue tests to evaluate the fatigue behavior of shear studs. The large-scale tests are comprised of a 30 ft. long rolled steel beam with shear studs welded in one of four different configurations of shear pocket spacings, along with a precast concrete deck. The shear stud configurations vary from a typical cast-in-place deck construction detail with studs every 12 or 24 in. to configurations more conducive to precast panels with clustered shear studs spaced at 36 and 48 in. The small-scale tests are used as a means of comparison to the large-scale tests, as well as a comparison to the historical push out test data including those used to develop the current AASHTO shear stud fatigue provisions. An additional motivation for the small-scale tests was to evaluate the CAFL of the AASHTO provisions since the small-scale tests could be loaded at a faster rate than the large-scale tests, and thus could be tested at lower stress ranges for longer fatigue lives.

**Large-Scale Fatigue Tests**

**Specimens & Test Setup**

The large-scale tests consist of a 30 ft. long W27x84 rolled steel beams and two concrete deck panels fabricated by a PCI certified precaster. Pockets were cast into the deck panels and were sized depending on the number of studs per pocket. Table 1 shows the experimental test matrix with the four different shear stud cluster spacings.

Regardless of stud cluster spacing, each of the beams has 12 studs per shear span, which corresponds to 38% composite action. Although AASHTO only allows for fully composite beams, these partially composite beams were chosen to ensure failure of the shear studs. Since the beams are partially composite, section properties were determined using AISC specifications (9).
Table 1. Large-scale experimental test matrix

<table>
<thead>
<tr>
<th>Stud Cluster Spacing</th>
<th># Longitudinal Shear Studs / Cluster</th>
<th>Total # Shear Studs / Shear Span</th>
<th># of Fatigue Tests</th>
</tr>
</thead>
<tbody>
<tr>
<td>12”</td>
<td>1</td>
<td>12</td>
<td>3</td>
</tr>
<tr>
<td>24”</td>
<td>2</td>
<td>12</td>
<td>3</td>
</tr>
<tr>
<td>36”</td>
<td>3</td>
<td>12</td>
<td>3</td>
</tr>
<tr>
<td>48”</td>
<td>4</td>
<td>12</td>
<td>3</td>
</tr>
</tbody>
</table>

Shear studs are placed longitudinally along the length of the beam in clusters at a pitch of 3.5 in. (4 times the stud diameter, \(d\)), which is less than the minimum AASHTO spacing of 6\(d\). This smaller pitch was selected to allow for smaller pockets and is currently allowed in Texas (10). The shear studs were welded onto the steel beams in accordance with AASHTO/AWS D1.5 specifications (11).

Before setting the concrete deck panels in place on top the steel beams, the top flange is coated with a thin layer of grease to reduce any friction between the flange and the deck panel so that nearly all of horizontal shear force transferred between the steel beam and the deck panels is carried by shear studs. Grout with an expected strength of approximately 8.0 ksi is used to fill the pockets, haunch, and transverse joint in the center of the beam. Slutter and Fisher concluded that the strength of the concrete around the studs does not significantly influence the fatigue resistance of the studs (2); therefore it is believed that these test beams can be representative of bridge girders with both pre-cast and cast-in-place concrete decks. Figure 3 shows a plan, elevation, and typical section view of the 36 in. cluster spacing specimen. The other specimens are similar to that shown in the figure, but differ in the number of studs per cluster and cluster spacing.

The beams have two load points with 11’-6” shear span on each end of the beam. Load was applied using a 220-kip servo-hydraulic actuator at each load point. The actuators were cycled under a constant load range to produce the desired average stress range at the base of all the studs in each shear span, while still maintaining elastic behavior in the beam. The beams were cyclically loaded until it was clear that at least one of the precast decks had completely separated from the beam.

**Instrumentation**

There are a total of 54 strain gauges used along the steel beam and concrete deck for each test. Six strain gauges are used at each of four cross sections on each shear span and one cross section at midspan. Each section contains of three gauges installed on the steel beam and three gauges on the concrete deck. Of the four cross sections with strain gauges along each shear span, two are located at a pocket and two are halfway between pockets.

An additional 32 strain gauges are used for some of the large-scale tests and were installed on selected shear studs in each shear span prior to grouting the concrete deck in place. For these tests, all 12 of the shear studs within the east shear span and the 4 endmost studs in the west shear span had strain gauges installed on them. For each of these studs, strain gauges were installed on the east and west side near the base of the stud. These strain gauges were used
in hopes of determining the order in which shear studs failed in fatigue during testing.

Twelve linear variable differential transducers (LVDTs) are used on each beam. Nine are used to record the relative slip between the deck panel and the top flange of the beam, and are mounted at the same sections as the strain gauges. Two LVDTs measure the relative uplift, or vertical separation, between the deck and the top flange. These uplift LVDTs are located in the mid-point of each shear span. The last LVDT measures vertical deflection at midspan of the beam.

A unique instrumentation feature used for the large-scale tests is the FARO Laser Tracker ION; this laser system uses a centralized head unit and a prism to record highly accurate 3-D coordinates on demand. In this case, the FARO is used to determine the relative slip and uplift between the haunch and the top flange of the steel beam. This is achieved by mounting aluminum spacers to the haunch and to the edge of the flange every 6 in. along the length of the beam, as shown in Figure 5. FARO measurements were recorded by stopping the fatigue tests at predetermined cyclic intervals (depending on expected fatigue life of the test) and applying a static load equal to the maximum fatigue load. The FARO prism was then nested into each of the aluminum spacers and measurements could be recorded. From the measurements, the relative slip and uplift between the haunch and the top flange could be calculated.

**Test Results & Discussion**

To date, 10 of the 12 anticipated fatigue tests are complete, with another test currently underway. Selected stress ranges for the tests include 20, 16, 12, and 10 ksi. The larger stress ranges (20 and 16 ksi) were selected to produce a fatigue failure in the shear studs within a reasonable timeframe, while the smaller stress ranges (12 and 10 ksi) were selected in hopes of producing “long-life” fatigue failures. Selected data from some of these tests, as well as criterion for defining a fatigue failure, will be presented in the following sections.

**Strain Gauge & LVDT Results**

During testing, strain gauge measurements from the steel beam were used to determine the location of the neutral axis (N.A.). The calculated N.A. locations were used to determine how composite action was affected during cycling. Figure 4 shows an example of this for beam 4F2, which has a stud cluster spacing of 48” and was tested at a stress range of 16 ksi. The plot shows the location of the N.A., referenced from the bottom of the bottom flange of the steel beam, at three different cross sections over the duration of the cyclic loading. The cross sections shown include one from the east shear span (E4), west shear span (W4), and midspan (M). The composite section and bare steel beam N.A. are also provided for reference. The composite N.A. in the figure refers to the partially composite section N.A.

As shown in the figure, the N.A. for all three sections starts near the composite section N.A. As the beam is cyclically loaded, the location of the N.A. for each section gradually drops down into the steel beam. The rate at which the N.A. decreases is relatively similar for all three sections until about
600k cycles. This shows that gradual fatigue damage is occurring at the shear studs along the entire length of the beam. After about 600k cycles, the N.A. for the west shear span (W4) drops rapidly, whereas the N.A. for the east shear span (E4) continues to decrease at the same rate. The midspan (M) N.A. also drops rapidly, similar to W4. This is likely because the studs in the west shear span had failed so the studs near midspan were now carrying the shear force that was once carried by the studs in the west shear span. These studs at midspan failed rather quickly due to the large shear stress range.

At first glance, this descent of the N.A. in W4 may not appear to be significant, especially since the N.A. is still well above that of the bare steel beam. However, the test was stopped at 700k cycles and, indeed, it was clear that the shear studs in the west shear span had failed such that the concrete deck was completely separated from the steel beam. This shows that the rapid shift in the N.A. in W4 does signify a fatigue failure of the shear studs within a shear span. The N.A. of W4 likely did not approach the bare steel beam N.A. because shear force is still being transferred between the deck and the beam by friction of the cracked stud surfaces and, to a lesser extent, any friction which may be present between the greased top flange and the bottom of the concrete deck. Field studies have shown that even a very small amount of friction, such as rivet heads against a cast-in-place concrete deck, can provide full composite action under service level loads (12).

It is even more apparent that the west shear span studs failed first when examining the slip data recorded from the LVDTs across two different cross sections of the same beam, shown in Figure 6. The two selected sections include one from the east shear span (E1) and west shear span (W1). Slip values are shown near the ends of the shear spans because this is where the slip is maximized. Both sections experience a moderate increase in slip from the beginning of the test until about 250k cycles, at which point the slip increase becomes more gradual. At about 600k cycles, the slip in W1 experiences a drastic increase, similar to how the N.A. saw a rapid shift. This again illustrates that the studs in the west shear span failed in fatigue before those in the east shear span.

The N.A. and slip results shown in the previous two figures were fairly typical of all of the large-scale beams tests. It was apparent that the N.A. results could be used to determine the shear span that had failed. More importantly, the N.A. results from the strain gauges and the LVDT results correlated well to the physical evidence of the concrete deck separating from the steel beam. For the larger stress range (20 ksi), the rapid increase in the N.A. and slip was more obvious than for the smaller stress ranges. The N.A. in the beams under the 20 ksi stress range also typically decreased until it was approximately equal to the N.A. of the bare steel beam. This likely occurred because the loads in these tests were large enough to overcome any friction causing composite action to remain in the beam after the studs had cracked.

Once it was clear which shear span had failed in a test, the N.A. from each of the sections in that shear span were examined. Figure 7 shows the N.A.

Figure 6. LVDT slip results from beam 4F2

Figure 7. Location of N.A. for west shear span on beam 4F2
location along three sections of the west shear span in the same beam, 4F2. Similar to the previous N.A. plot, the composite section and bare steel beam N.A. are also included for reference.

The figure shows that each of the three sections lose composite action in a similar fashion. The N.A. in all three sections experience an initial decrease from the beginning of the test until about 250k cycles. Clearly, there is some noise in the strain gauges in section W2 at the beginning of the test that is likely due to the small stresses in this section, which become magnified when the N.A. is calculated using these stresses. From about 250k-600k cycles, the N.A. in all three sections continues to decrease at a gradual rate. Although the three sections do behave similarly, the sections closer to the end of the beam do begin to lose composite action sooner than those closer to midspan. This is reasonable because sections near the end of the beam have to resist a larger horizontal slip to maintain compatibility than those closer to midspan. Although sections closer to the end of the beam do begin to lose composite action sooner, all of the sections lose composite action at a much faster rate after approximately 600k cycles. The N.A. for all of the sections also appear to level off just before 700k cycles.

All of the beams experienced similar N.A. behavior to that shown in Figure 7. In general, the sections closer to the ends of the beam lost composite action sooner than the sections closer to midspan, but typically all sections within a shear span reached a point of no composite action at about the cycle count. Typically, there was not a significant difference in the location of the N.A. between sections either at a stud cluster or between stud clusters. In other words, composite action along the length of a shear span did not seem to be affected by placing shear studs in clusters with a spacing of up to 4 ft. between clusters.

Using the strain gauge data to calculate the location of the steel beam neutral axis, a definition of failure for the fatigue tests was developed and is defined as the following: A beam is considered to have reached “failure” if one or more cross sections experience a complete loss in composite action. In beam 4F2, this was determined to have occurred at 680k cycles. This is when the neutral axis appears to have leveled out such that it was clear the shear studs in the west shear span had exhausted their fatigue lives. This definition of failure is somewhat similar to that used in some of the few other large-scale fatigue tests on shear studs. Toprac et. al. (4) defined failure as the point at which the rate of loss in composite action increases considerable. This is similar to the failure definition in this study since the rate of composite action remains fairly constant until just before the beam fails completely. Using the definition of failure developed for this study, stress range and fatigue life (S-N) data were constructed from the large-scale fatigue tests, and are shown in Table 2. In addition to S-N data, the table also includes the basic naming convention for each beam as well as stud cluster spacing. Stress ranges in the table represent the initial average stress range on all of the shear studs within a shear span.

Table 2. S-N data for large-scale fatigue tests

<table>
<thead>
<tr>
<th>Beam name</th>
<th>Stud cluster spacing (in)</th>
<th>Stud stress range (ksi)</th>
<th>Cycles to failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>1F1</td>
<td>12</td>
<td>20</td>
<td>175,000</td>
</tr>
<tr>
<td>2F1</td>
<td>24</td>
<td>20</td>
<td>174,000</td>
</tr>
<tr>
<td>3F1</td>
<td>36</td>
<td>20</td>
<td>51,000</td>
</tr>
<tr>
<td>4F1</td>
<td>48</td>
<td>20</td>
<td>91,000</td>
</tr>
<tr>
<td>1F2</td>
<td>12</td>
<td>16</td>
<td>130,000</td>
</tr>
<tr>
<td>2F2</td>
<td>24</td>
<td>16</td>
<td>260,000</td>
</tr>
<tr>
<td>3F2</td>
<td>36</td>
<td>16</td>
<td>747,000</td>
</tr>
<tr>
<td>4F2</td>
<td>48</td>
<td>16</td>
<td>680,000</td>
</tr>
<tr>
<td>1F3</td>
<td>12</td>
<td>12</td>
<td>yet to test</td>
</tr>
<tr>
<td>2F3</td>
<td>24</td>
<td>12</td>
<td>ongoing</td>
</tr>
<tr>
<td>3F3</td>
<td>36</td>
<td>12</td>
<td>4,048,000</td>
</tr>
<tr>
<td>4F3</td>
<td>48</td>
<td>10</td>
<td>25,000,000&lt;sup&gt;a&lt;/sup&gt;</td>
</tr>
</tbody>
</table>

<sup>a</sup> – Test declared a run out

In the table, tests are placed into three stress range groups: large (20 ksi), medium (16 ksi), and small (10 and 12 ksi). Within each stress range group, a test was conducted at each of the four stud cluster spacings. One test, beam 4F3, was declared a run out since composite action still remained and the shear studs were still intact after 25 million cycles of loading at a stress range of 10 ksi.

Using the data from Table 2, an S-N plot was constructed for the completed tests and is shown in Figure 9. Beams with different stud spacings are shown using different shaped data points. Also included in the figure are the AASHTO LRFD shear stud fatigue design equation (12) and upper- and lower-bound regression curves for the S-N data of the large-scale tests. The mean regression is shown as a solid line, while the +2σ (2 x standard
deviation) and -2σ bounds are shown as dashed lines. These represent the 95% and 5% confidence intervals, respectively.

In some cases on the figure, two data points (1F1 and 2F1) are close enough together that on the plot, they might appear as a single point. The slope of the regression line for the 10 completed tests is -6.7. It is interesting to note that the 5% confidence interval falls nearly on top of the AASHTO (2010) shear stud design equation from approximately 10k-1.0 million cycles. Over this range, the AASHTO stud design equation seems to accurately represent the S-N test data. However, after 1.0 million cycles, due to the semi-log nature of the AASHTO curve, the design provisions underestimate the fatigue life of the test beams, especially at the CAFL of 7.0 ksi. This is especially apparent since the 4F3 test was declared a run out under a 10 ksi stress range after 25.0 million cycles.

Another observation from the S-N plot is that there does not appear to be any strong correlation between the shear stud cluster spacing and its fatigue life. In particular, the extended cluster spacings of 36" and 48" did not seem to negatively affect the shear studs’ fatigue performance due to shear lag. Any difference in fatigue behavior appears only to be scatter, as is common with any fatigue test data.

**FARO Slip Results**

Since the relative slip and uplift between the top flange of the beam and grouted haunch was recorded at predetermined cycle intervals, generally every 100k cycles depending on the expected fatigue failure, fatigue damage could be observed throughout testing. This section will present selected data recorded using the FARO laser system since it provided good resolution of slip and uplift along the entire length of a beam. The LVDT data did provide good agreement with the FARO data, but are not presented for brevity.

Figure 8 shows the FARO slip results for beam 4F2. In the figure, slip values were recorded at the maximum loads of the fatigue loading. Cyclic loading was stopped and the beam was loaded statically to record measurements every 100k cycles. Positive values represent a slip toward each end of the beam, away from midspan. The values are referenced from measurements recorded of the beam under self-weight only. Shaded vertical columns are included to indicate the location of the shear stud clusters.

In general, the slip values confirm the behavior seen in the strain gauge and neutral axis data. It is most easily seen when examining the slip values of the west shear span, which was determined to have failed first. The slip increases at a moderate, relatively consistent rate for each cyclic interval until 300k cycles. From 300k-600k cycles, the slip increases at a much slower rate. Then, between 600k-700k cycles, there is a much larger slip increase; this agrees with the fatigue life of this beam, which was defined to have occurred at 680k cycles.

One interesting feature to note in the figure is the shape of the slip plot. It appears almost step-like, with most of the drastic slip increases occurring on
the beam-end side of each of the stud clusters. For example, in the west shear span, the slip increases very sharply at each cyclic interval directly to the west of the stud cluster closest to the west load point. This indicates that there is slip damage in the two stud clusters closest to the west end of the beam since the slip increases after the first cluster. At the second stud cluster, there is another, though smaller, sharp increase in slip on the west side of the cluster, indicating slip damage in the cluster closest to the west end of the beam. From there, the slip remains relatively constant toward the west end of the beam.

This step-like slip behavior was present in the beams with larger cluster spacings (36” and 48”). In the more typical stud spacing beams (12” and 24”), the slip values increased gradually in a more continuous fashion from each load point toward the end of the beam. A more detailed discussion of this behavior can be found in Provines and Ocel (2014) (13).

FARO Uplift Results

Similar to the slip results, the FARO laser system was also used to determine the relative uplift between the top flange of the beam and the haunch at predetermined cycle counts. Figure 10 presents the uplift results for beam 4F2, in which measurements were recorded at the maximum load of the fatigue cycles every 100k cycles. Similar to the slip figure, the stud clusters are shown by the presence of shaded columns and values are referenced from the beam under self weight only. Positive values indicate that the haunch is moving upward relative to the top flange. Note that the “negative” uplift, especially in the west end of the west shear span, is likely due to degradation of the grout during fatigue loading.

The damage progression of the uplift is somewhat different than was shown with the other data presented. According to the plot, there was little uplift damage done within the first 100k cycles. From 100k-300k cycles, there is a moderate amount of damage done in both shear spans, with the east shear span, interestingly, accumulating the maximum value. From 300k-500k cycles, little increase in uplift is present in both shear spans. Then from 500k-700k cycles, a greater increase in uplift is present in the west shear span, while the east shear span uplift remains relatively constant. Although the progression is different, the uplift data still does agree with the result that the west shear span failed first.

The overall uplift behavior is quite different from the slip behavior. There is little to no uplift in the constant moment region and then reaches a maximum at about the mid-shear span before decreasing to approximately zero at the ends of the beam. This behavior demonstrates that the concrete deck panel is arching somewhat over the steel beam along the shear spans. This arching behavior was present in all of the beams tested, though it was more pronounced in some than in others.

The figure also clearly show the presence of peaks, or local maximums, in the uplift at each of the stud clusters. These peaks are occurring since the shear studs are trying to maintain vertical compatibility between the deck and the beam. The uplift tends to decrease between pockets since there are no studs at these locations for damage to occur. These uplift peaks were not present in the 1 ft. spacing beams, but were in other spacings. The peaks were clearly more apparent in the larger cluster spacings (36” and 48”).

Small-Scale Fatigue Tests

Specimens & Test Setup

Small-scale push out tests were also conducted in conjunction with the large-scale testing. These test specimens are similar to historical push out tests, but with some adjustments. Each of the small-scale tests consists of a 24” long W10x60 steel beam and two small precast concrete decks with a pocket cast into

Figure 10. Uplift results taken at peak loads for 100k cycle intervals for beam 4F2
each of them. One shear stud is welded to each flange of the steel beam and the concrete deck panels are placed around the shear stud and grouted into place. Figure 11 shows a drawing of these specimens.

Some of the key differences between push out tests in this study and those used in some previous tests include the presence of shear studs on both sides of the steel beam and the number of studs per side. Some previous push out tests, such as those done by Slutter and Fisher (2), were “one face” push out tests, meaning they only had a slab on one side of the steel beam. An analytical study has shown that these types of tests can induce up to 20% more tensile forces in the studs, which could result in a reduced fatigue life of the studs (14). It is for this reason that the push out tests conducted in this study have slabs on both sides of the steel beam. Doing so allows for a comparison of the tests in this study and the majority of the historical data (15, 16, and 17).

The other main difference between the push out tests in the current study and in previous testing is that only one shear stud is welded per side of the steel beam. Many other push out tests have either two or four studs per side. In most push out tests, a specimen is considered to have reached failure when all of the shear studs on one side of the beam have cracked such that the slab is detached from the steel beam. In these cases, failure of the test is actually the fatigue failure of all of the shear studs on one side of the steel beam, which is typically either two or four

For the present study, one stud is used per side so that failure of a specimen would be the fatigue failure of a single shear stud. Because of the single stud configuration, there were concerns that the concrete slabs could rotate about the shear stud when loaded. Therefore, the decision was made to load the specimens in tension, rather than compression. Loading the specimens in tension would allow the concrete slabs to essentially be “self-leveling” to prevent any unwanted rotation.

The set up for loading the fatigue push out tests is shown in Figure 12. A previous test specimen is also included in the figure for reference. The fatigue push out tests are loaded using a 55-kip servo-hydraulic actuator. A loading fixture was constructed out of C-shape channels to act as reactions on the tops of the concrete slabs. Steel plates were grouted into plate on top of both concrete slab to ensure the reactions were being applied evenly between the two slabs. The actuator was cycled under a constant load range to produce the desired stress range at the base of both shear studs while still maintaining elastic behavior. The specimens were loaded until one or both of the slabs had completely separated from the steel beam. Typically one of the slabs became separated first because one of the studs failed in fatigue before the other. There were cases, however,
in which both studs failed in fatigue at approximately the same time.

**Instrumentation**

There are a total of four strain gauges and two LVDTs on each push out test. The strain gauges are placed on the top and bottom (in the direction of force) of each shear stud, located close to the base of the stud. Since similar strain gauges were placed on the large-scale tests, they were placed on the small-scale tests as a means of comparing the two results. One LVDT is installed on each deck panel to measure the relative slip between the concrete and the steel beam.

**Test Results & Discussion**

To date, 9 of the 14 anticipated fatigue tests are complete. Initial stress ranges were selected in hopes of reaching a fatigue failure within a reasonable amount of time. Once a few tests had been completed, stress ranges could be selected to essentially fill in the gaps in the data. Because one of primary goals of the push out tests was to evaluate the current AASHTO LRFD shear stud CAFL, most of the stress ranges for the push out tests were selected to produce “long life” fatigue tests.

**S-N Data**

Unlike the large-scale tests, a fatigue failure was much easier to define in the small-scale tests. For these tests, a fatigue failure is defined to have occurred once one or both of the two slabs is completely separated from the steel beam. Table 3 shows the S-N data from the small-scale tests.

<table>
<thead>
<tr>
<th>Test name</th>
<th>Stud stress range (ksi)</th>
<th>Cycles to failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>PO-F1</td>
<td>14</td>
<td>50,000,000(^a)</td>
</tr>
<tr>
<td>PO-F2</td>
<td>20</td>
<td>148,935</td>
</tr>
<tr>
<td>PO-F3</td>
<td>18</td>
<td>817,519</td>
</tr>
<tr>
<td>PO-F4</td>
<td>16</td>
<td>7,472,601</td>
</tr>
<tr>
<td>PO-F5</td>
<td>15</td>
<td>1,792,647</td>
</tr>
<tr>
<td>PO-F6</td>
<td>13</td>
<td>50,000,000(^a)</td>
</tr>
<tr>
<td>PO-F7</td>
<td>15</td>
<td>712,456</td>
</tr>
<tr>
<td>PO-F8</td>
<td>15</td>
<td>12,884,837</td>
</tr>
<tr>
<td>PO-F9</td>
<td>15</td>
<td>2,846,232</td>
</tr>
<tr>
<td>PO-F10</td>
<td>15</td>
<td>ongoing</td>
</tr>
</tbody>
</table>

\(^a\) – Test declared a run out

As seen in the table, two of the tests were stopped after 50 million cycles without ever reaching failure; these tests were considered run out tests. The stress ranges that produced these run out tests were 13 and 14 ksi. All tests cycled at load ranges greater than 14 ksi produced tests with fatigue failures. Data from the table were used to develop an S-N plot shown in Figure 13.

![Figure 13. S-N data for small-scale tests](image)

Similar to the S-N plot for the large-scale tests, Figure 13 includes the S-N test results as well as three regression lines, which include the mean regression and 95% and 5% confidence intervals. The slope of the regression lines is -9.3. The 5% confidence interval bears a close resemblance to the AASHTO stud fatigue design curve at about 1k-100k cycles. After this point, the AASHTO curve decreases more rapidly until reaching the 7.0 ksi CAFL.

**Comparison of Test Results**

One effective means of comparing the large- and small-scale test results is to compare the regression lines developed from each respective data set. This comparison is shown in an S-N plot in Figure 14. Also included in the figure are the shear stud fatigue design equations for AASHTO and the Eurocode.

From the figure, it is easy to see that the large-scale test regression has a steeper slope (-6.7) than the small-scale regression (-9.3), with the two mean regressions crossing at around 10k cycles. The difference in slopes could be due to the difference in loading. In the large-scale tests, the loading on the shear studs, though idealized to be pure shear, is
more likely a combination of shear and tension forces. For the small-scale tests, the loading on the shear studs is much more like pure shear. In the case of fatigue, tensile loading is more severe since it causes a crack to open up and extend when loaded.

This paper describes the results of construction. To date, 10 of the 12 clusters as a means of providing composite action between a steel beam and a precast concrete deck panel. To date, 10 of the 12 anticipated large-scale fatigue tests are complete, with 1 more test ongoing. A failure criterion for the large-scale tests has been defined as a complete loss in composite action, as indicated by the movement of the neutral axis in the steel beam. Using this definition of failure, S-N data for the large-scale tests has been developed. A regression analysis reveals that the slope this data is approximately -6.7 and that the 5% confidence interval appears similar to that AASHTO stud design curve over a fatigue life of 10k-1.0 million cycles. At cycles greater than this, AASHTO appears too conservative compared to the test data due to the semi-log nature of the design curve.

Strain gauge, LVDT, and FARO slip data suggested that fatigue damage occurred at moderate rate at the beginning of a test, then the rate of damage decreased to a relatively steady state, then increased once again before reaching failure. Strain gauge data revealed that composite action was initially lost at a faster rate near the ends of the beams, but that an entire shear span seemed to reach a point of no composite action at approximately the same time. The extended stud cluster spacings of 36” and 48” did not appear to have a negative impact on the fatigue performance of the shear studs, both in terms of the stud spacing greater than the current AASHTO spacing limit of 24” and using shear studs in clusters rather than a uniform longitudinal spacing as is typically done in conventional bridge construction.

At this point, 9 of the 14 anticipated small-scale tests are complete, with 1 more test ongoing. A regression analysis of the S-N data revealed a slope of -9.3, which is slightly shallower than the large-scale test regression. The increased slope is likely due to the pure shear loading of the small-scale tests rather than the combined shear and tension loading of the large-scale tests. The large- and small-scale test regressions equations were relatively similar to the Eurocode stud design equation, providing more evidence that the AASHTO stud fatigue design equation is too conservative, and the semi-log nature of the curve warrants re-examination. The overly conservative nature of the AASHTO provisions are especially problematic for the use of precast concrete deck panels.

In addition to completing the remaining large- and small-scale fatigue tests as described in this paper, a

Summary of Tests & Future Work

This paper describes the results of ongoing testing to investigate the fatigue resistance of using clustered shear studs in clusters as a means of providing composite action between a steel beam and a precast concrete deck panel. To date, 10 of the 12...
partial composite action analysis will be performed as a means for comparison to the large-scale test data. Forensic examination, including non-destructive evaluation, of many shear studs in both tests is being conducted. This includes examining the grout around the shear studs to determine the extent of damage sustained during testing, as well as examining the size and initiation point of the fatigue cracks found in the shear studs. Metallographic analysis of the shear stud welds is also anticipated.

In addition to the fatigue testing described in this paper, static tests have also been conducted as part of this study. Large-scale testing is complete, while small-scale static tests will commence in the near future. The large-scale static tests utilized similar deck panels and shear stud cluster spacings as used in the large-scale fatigue tests as described in this paper. The push out static tests will consist of both cast-in-place and precast concrete deck panels. The static tests will examine the current AASHTO shear stud strength and spacing provisions.

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