RESILIENT INFRASTRUCTURE
June 1–4, 2016

FATIGUE OF STUD SHEAR CONNECTORS IN STEEL-PRECAST COMPOSITE BRIDGES

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ABSTRACT

Modular bridge systems consisting of precast concrete deck panels connected to steel girders are becoming increasingly popular due to their rapid construction and optimal material utilization. The shear connection is a critical element of the system, having significant impacts on construction time, economic and environmental cost, structural integrity, and durability. Today welded shear studs are by far the most common type of shear connection. In steel-precast composite bridges, the studs are commonly grouped together so that the precast deck panels can be affixed to the girders by providing full depth "shear pockets" filled with grout. A laboratory beam testing program is underway at the University of Waterloo to investigate the effect of cyclic loading on stud shear connectors in cast-in-place and precast bridge girders. The program consists of twelve beam specimens, uniquely tested using a variable amplitude load history simulating Canadian highway truck traffic. In addition to yielding valuable S-N (stress plotted vs. the number of cycles until fatigue failure) data, initial test results provide evidence of the benefits of redundancy in the structural system and the value of beam tests over push-out tests. Calculating connector stresses in a composite beam is made complicated by interfacial slip and neutral axis migration. The end goal of this research is to provide Canadian bridge designers and erectors with improved design and construction recommendations in order to improve the efficiency and economy of this structural system for rapid bridge replacement projects.

1. INTRODUCTION

The use of precast concrete bridge decks began in the 1960s in North America with regulatory authorities recognizing the benefits of reduced construction time in minimizing traffic disruption and avoiding lost productivity (Fowler, 2008). Although costs to the bridge owner are higher, the overall costs of modular accelerated bridge construction to society is significantly lower when compared with traditional cast-in-place (CIP) construction for many structures, particular those that accommodate high volumes of traffic. In addition to its use in accelerated construction applications, precast concrete construction is sometimes the only option for bridges in remote locations. The original use of precast decks was in non-composite systems, where the deck was used as a road surface and a means to transmit loads transversely to the primary structural elements. However, it was soon realized that a very efficient means of resisting longitudinal moments was through the use of the composite, shear connected system, where the deck works together with the longitudinal beams to carry load.

Besides initial cost, two other inhibitors of using precast decks include lack of knowledge or tools for designers, and questions of durability related to long-term performance. These two issues are the focus of an investigation currently underway at the University of Waterloo. A composite beam fatigue testing program is being carried out in order to study the longitudinal shear connection between precast panels and steel bridge girders. This connection is a critical element to the speed and durability of the modular bridge system (Burak & Seraderian, 2010). Specifically, the study aims to quantify the fatigue performance of stud shear connectors in precast decks compared to CIP decks. A conceptual image of precast deck panels with shear connector "block-outs" is shown in Figure 1.

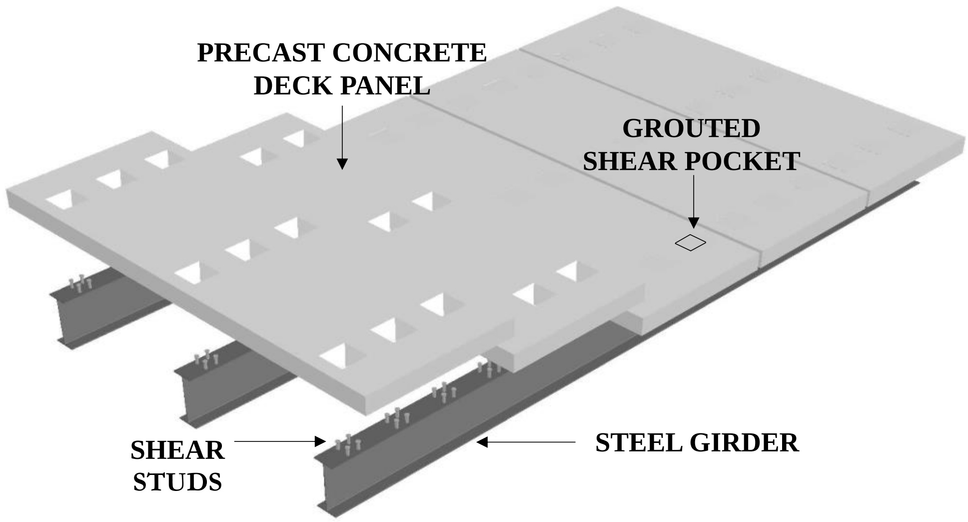

Figure 1: Precast Deck Panels Shear Connected to Steel Girders

2. LITERATURE REVIEW

2.1 Push-Out Tests by Slutter and Fisher

Code provisions regarding the fatigue of welded shear studs all have their roots in the seminal work of Slutter and Fisher, performed at Lehigh University in Pennsylvania around 1965. Prior to that time, fatigue provisions were based on over-conservative approximations from static test results. With the goal of overhauling the shear connector design procedure, over 40 fatigue tests were performed on push-out specimens with stud and channel connectors (Slutter & Fisher, 1966). A push-out specimen is a relatively inexpensive test used to study shear across an interface, often meant to replicate the longitudinal shear in a beam. A typical push-out test specimen is shown in Figure 2.

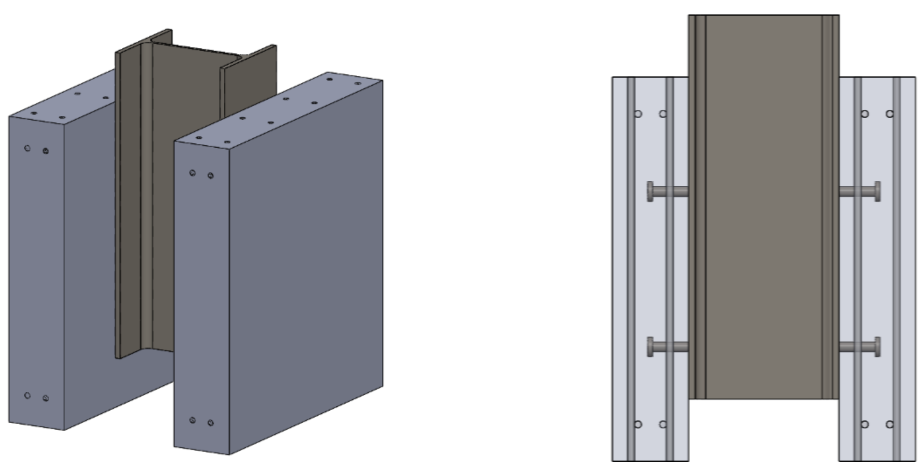

Figure 2: Push-out Test Specimen

Slutter and Fisher found that the stress range during cycling is the most important variable in the fatigue life of a stud, which is in agreement with general fatigue principles. In addition, they found that they could relate stress range to the number of fatigue cycles until failure for a stud using Equation 1.

[1] $\log N = A - B(\Delta\tau)$

In Equation 1, N is the number of cycles experienced by a shear connector until failure, $\Delta\tau$ is the range of shear stress, and A and B are constants (obtained by regression analysis). The relationship they used to relate stress range and fatigue life was semi-log in nature. The justification of using push-out tests instead of beam tests was based on cost, convenience, and their assertion that push-out test results represent a lower bound for shear connector failure. In reality, this assertion has not been confirmed, although it is generally accepted that push-out tests are more conservative. Other notable researchers since Slutter and Fisher in the field of shear connector fatigue include Oehlers (Oehlers and Foley, 1985), Johnson (2000), and Issa (Issa et al., 2003), to name a few.

2.1 Code Provisions

For many years, the Canadian Highway Bridge Design Code (CHBDC) and the American Association of State Highway Transportation Officials (AASHTO) Load and Resistance Factor Design (LRFD) Specification directly used Slutter and Fisher's results to form the requirement for fatigue resistance of stud shear connectors. The semi-log fatigue curve used for design was not in agreement with other fatigue details, which all used the same log-log curve, shifted up or down depending on the severity of the detail. Treating the fatigue of stud connectors differently than other details has logical reasons, since embedded studs have characteristics of a "black box"; the damaged element is not able to be inspected or monitored, and it is difficult to determine the stress state at the critical fatigue crack location because of the welding process as well as interfacial slip between the steel and concrete.

In 2010, a supplement to the 2006 publication of the CHBDC (CSA S6S1-10) reworked the fatigue requirement for studs to be consistent with that of other fatigue details according to an investigation by Zhang (2007). In this investigation, a regression analysis was performed on a large collection of push-out tests from many researchers. A log-log relationship was found closely approximating an existing fatigue category (Detail Category D). It should be noted that the endurance limit for the detail did not change, since Category D had the same constant amplitude fatigue limit as the value previously used for studs. Zhang also performed regression analysis on beam tests, but too few had been performed and published in the literature to draw any conclusions. The current code equation form is given below, and can be compared with Equation 1 (its prior form).

[2] $\log N = \log C + m\log(\Delta\tau)$

In Equation 2, N is the number of cycles, $\Delta\tau$ is the range of shear stress, C is a constant (given by the code as $721 \cdot 10^9$ for Detail Category D), and m is the slope of the design curve (given as 3).

2.2 Push-Out Tests vs Beam Tests

Another key difference in the treatment of stud connectors compared to other fatigue details involves the approximation of using push-out tests rather than beam tests to drive design. Due to the apparent conservatism in this, the standard procedure of shifting the design curve two standard deviations from the mean for safety was abandoned. Instead, the design curve simply cuts through the mean of the push-out test data. This assumption is not based upon any critical analysis, but has not concerned anyone. The lack of concern probably has to do with the fact that there have not been any reported fatigue failures of stud shear connectors in the field.

The value of a beam test lies primarily in the boundary conditions, which add several key elements when compared to push-out tests. In a push-out test loading on connectors remains constant relative to one another throughout the test. A beam test features force redistribution; when one stud begins to crack and fail, others pick up the load, and the crack grows slower because the cracked stud attracts less force. Not only is there redistribution in a beam, but there is friction at the interface which lessens the force transferred through the studs. The actual stress state near the potential crack location is made complicated by stud axial forces and the state of stress in the top flange of the beam, something that changes as fatigue progresses and the neutral axis of the section moves down into the steel. These are only some of the complications a beam test introduces relative to a push-out test, and it is easy to see just how different they are.

2.3 Precast Panel Considerations

Three differences of possible relevance arise when considering precast panels in lieu of CIP construction from a stud fatigue perspective. These issues include the absence of stresses in connectors due to concrete shrinkage, the possible absence of friction due to panel placement and levelling, and maximum spacing considerations to avoid deck lift-off. Neither shrinkage nor friction are accounted for anywhere in design, and these issues receive relatively little attention by researchers and code makers. Maximum spacing considerations, on the other hand, are an important consideration because they affect the economy of the precast system. Shear pockets are expensive and time consuming to create and fill with grout, and it is in the contractor's best interest to minimize them. Until recently, Canadian and American bridge codes limited the spacing of stud connectors in precast applications to the same standard as CIP construction at 610 mm or 24 inches. Recognizing the work of researchers including Badie et al. (2010), who showed that significant slab liftoff does not occur at larger spacings, this limit has been almost doubled in Canada.

3. BEAM TESTING PROGRAM

3.1 Beam Specimen Geometry & Instrumentation

The beam testing program currently underway investigates the fatigue behaviour of headed shear stud connectors embedded in steel-concrete composite sections. A total of 12 beams are being tested, 6 of which utilize traditional CIP methods, and 6 that have been constructed using precast slabs. Both the CIP and precast specimens span a length of three meters and use a W250x49 steel section connected to the underside of a 600 mm wide, 125 mm deep, 45 MPa concrete slab as seen in Figure 3. The specified compressive strength of the grout is approximately 60 MPa. The beams are simply-supported and the loading is applied with a spreader beam which provides two point loads at an offset distance of 500 mm and 1000 mm from the west support. These details can be seen in Figure 3. A total of 24 stud connectors were welded to the top flange of the steel section for each specimen. The CIP beams have uniformly spaced stud pairs at 250 mm along the length, while the precast beams utilize a cluster of 4 studs at 6 discrete locations along the beam, spaced at 500 mm. These full depth pockets were filled with grout once the slab was positioned on the steel to complete the connection.

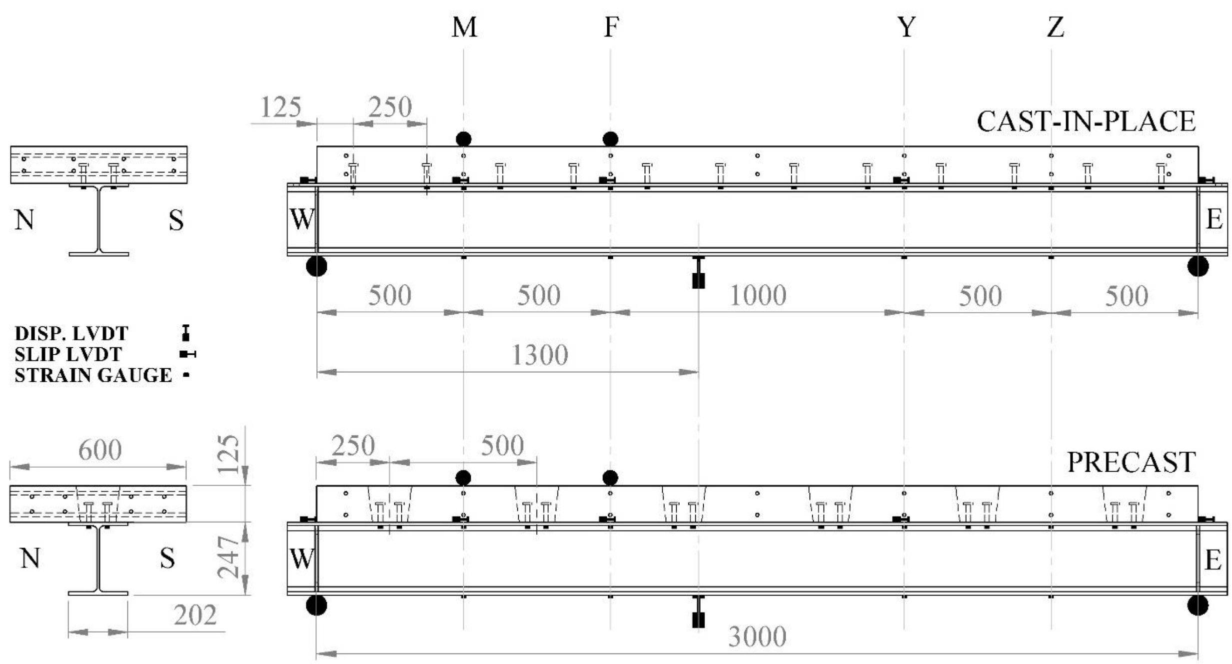

Figure 3: Specimen Geometry and Instrumentation

One challenge that the research team was presented with was developing an instrumentation approach that would both quantitatively and qualitatively track the progression of stud failures during cyclic loading. The goal was to capture this progression of stud failure and to collect data points that could be overlaid on top of existing S-N data (stress plotted with the number of cycles until fatigue failure) and compared to previous tests gathered from the literature. Complications arose from the fact that studs cannot be instrumented directly without being damaged during loading or impeding the shear connection properties themselves. The fact that the stud is also encased in concrete or grout discards any means of visual inspection during the test. Visual inspection can only take place afterwards during a specimen autopsy, during which time the specimen is destroyed. This challenge was overcome several ways: by applying strain gauges to the underside of the top flange directly under a welded stud, capturing a strain profile at locations between stud groups, and measuring interfacial slip between the slab and the steel flange. Figure 3 shows the location and type of instrumentation used.

Capturing the change in local strain under the stud provides a means of determining fatigue crack initiation and its subsequent propagation. The strain measured is induced by the local distortion of the top flange due to the horizontal force resisted by the stud. Since the magnitude of the strain readings is largely affected by the placement of the gauges, the general trend of the data was used as a qualitative descriptor in formulating the failure criteria and was utilized in determining the number of cycles to failure for each of the studs.

Strain profiles are measured between shear pockets locations at sections M, F, Y, and Z as shown in Figure 3. By measuring the strain on the underside of the top and bottom flanges respectively, it is possible to calculate the horizontal axial force in the slab being transferred into the studs. It is expected that as the studs are increasingly damaged, the slab axial force should decrease and approach zero when the studs have completely sheared off. This data provides information on the actual stress experienced by the studs over time and offers insight to the redistribution of stresses over the specimen’s fatigue life. To capture the interfacial slip behaviour of the beams during fatigue testing, a set of five linear voltage displacement transducers (LVDTs) were positioned at each of the beam’s ends (W and E) and at profiles M, F, and Y. An additional LVDT was positioned 1300 mm from the west support on the underside of the bottom flange to measure the change in maximum deflection during the cyclic loading.

3.2 Experimental Loading & Program

Most fatigue testing on shear connectors conducted to date have been under constant amplitude loading conditions. The effects on fatigue performance of connector slip during infrequent overload cycles (due to very heavy trucks or convoys for example) may be significant. For this reason, a variable amplitude loading history was used for the fatigue testing, simulating typical in-service loading for a bridge subjected to Ontario highway truck traffic. The foundation of the variable amplitude loading history utilized in this experiment was data collected from a study completed by the Ministry of Transportation Ontario (MTO) in 1995. This study contained the weights of 10,198 trucks that were randomly selected while traveling on the road network. The results from this survey are presented in the form of a histogram in Figure 4 (left). This data was randomized and formulated into a variable amplitude load-history. A segment of the first 25 cycles within this history is presented in Figure 4 (right).

The test setup employs two-point loading, offset to create a varying interface shear profile along the beam span, which allows the effects of connector failures on the subsequent shear loading of neighbouring connector groups along the span to be studied. Since the experiment utilizes a variable amplitude loading history, an equivalent stud stress was determined using the Palmgren-Miner’s linear damage accumulation rule. This equivalent stress and variable amplitude loading history was then scaled down appropriately to hit the target equivalent stress levels for each test specimen. The test program matrix is presented in Table 1. Due to the unique off-centred loading arrangement of the spreader, the stud pairs 1 and 2 (west to east) experience the target stress, while stud pairs 3-12 experience exactly one-third of the target stress.

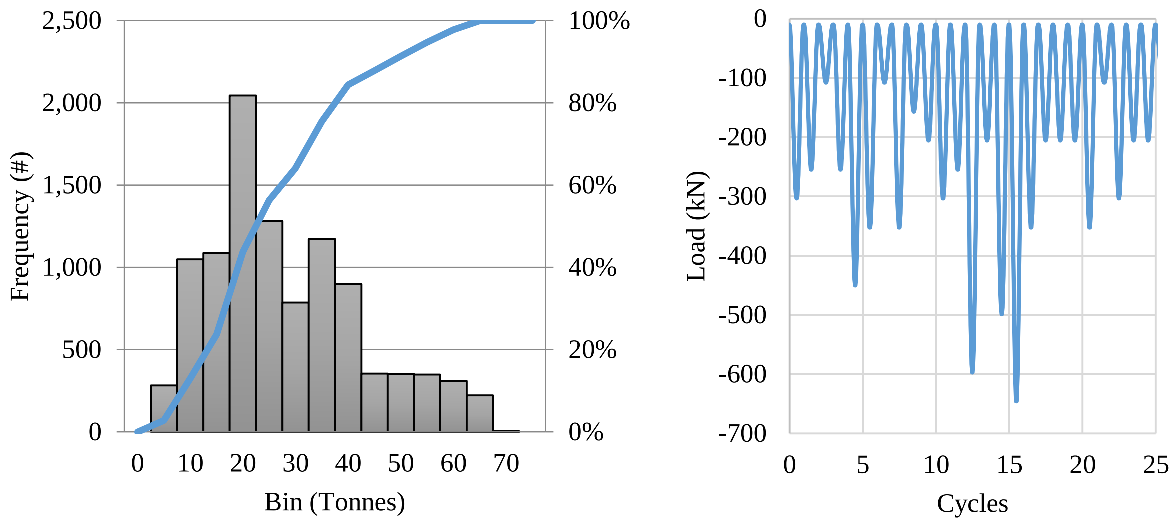

Figure 4: 1995 MTO Truck Survey Data (left), and a Segment of Variable Amplitude Loading History (right)

Table 1: Test Program Matrix

Eq. Stud Shear Stress (MPa)		No. of Specimens	
Studs 1-2	Studs 3-12	Precast	Cast-in-Place
80	27	1	1
100	33	1	1
120	40	1	1
140	47	1	1
200	67	1	1
300	100	1	1

Static loading was completed initially for each specimen (prior to the application of load cycles), and at 100% and 1000% of the code predicted fatigue lives of the critical stud connectors. The initial static test is to break the chemical bond adhering the concrete to the top of the steel flange upon casting (for the CIP specimens), and to collect data before any damage is produced from the subsequent cyclic loading phase. The data collected in the static tests include max deflection, interfacial slip, and strain profiles located at M, F, Y, and Z. A load of 200 kN was selected as a standard load at which the data from all specimens would be compared.

4. EXPERIMENTAL RESULTS

4.1 Specimen Autopsy & Observations

Testing is currently underway on the specimens shown in the test matrix (Table 1). Specimen autopsies and analysis of data harvested through instrumentation are ongoing, but several observations have been made at this stage. The best way to illustrate these observations is to describe them for a single specimen, and for our purposes this will be Specimen S3. This specimen is a precast specimen that was tested with a fatigue loading equivalent to constant amplitude loading at $\Delta\tau$ = 140 MPa on the critical stud group. It is important to note that 140 MPa is a longitudinal shear stress; it is the amount of force the critical stud group is transmitting between the steel beam and the concrete slab, divided by the cross-sectional area of the studs in the group. The concrete in the precast slab for S3 reached a 28-day strength of 48.3 MPa, and the grout used to fill the shear pockets reached a similar 54.1 MPa.

For a precast specimen, one shear pocket on the west side of the beam is loaded at the critical loading (140 MPa for S3), and the rest are loaded at one third of this value. The number of cycles until failure for this stud group, according to the CHBDC provision (Det. Cat. D), was 260,000 cycles. S3 was cycled more than ten times this value to 2.75 million cycles before being taken out of the loading frame for autopsy. The reason for loading so far above the code specified failure cycle count is to ensure failure at the critical group of studs, and to attempt observation of any progressive failure patterns thereafter. During the autopsy, the slab was cut transverse to the beam direction between stud groups, starting at the ends, and then direct lifting of an end slab portion was attempted. This lifting is only possible if the studs were fully severed from the steel top flange at the interface of slip. It was found that direct lifting was not possible for S3; lifting was rarely possible for any specimen as studs are almost always connected to the steel with some area. This is probably due to force redistribution, where neighbouring stud groups prevent the amount of slip necessary to fully severe cracked studs from the top flange.

Since direct slab lifting could not take place for Specimen S3 for the critical stud group, the concrete was fully chipped away around the group. At this point the studs were bent over with a sledgehammer until they fell off of the beam. This is referred to as a “bend test”. A stud passes the bend test when it becomes fully bent over, with its head touching the top flange of the beam, and does not fracture and fall off. This is adapted from CSA W59 (Welded Structural Construction), where the bend test requires a bent angle of 30 degrees. The bend test is a sign of residual ductility and strength, and is a standard procedure for any stud welder to perform during the welding process. Performing the bend test after fatigue loading does not carry the same meaning as the initial welding bend test, as some studs that are cracked can still pass the bend test due to flexibility, but it was found to be a useful classification tool.

Each stud in the critical group failed the bend test for Specimen S3, with little residual strength remaining (the sledgehammer blows did not need to be forceful to fail the studs). Figure 5 shows the group before and after the bend test, and shows the area that remained attached at the time of test termination. It can be seen that the fatigue cracks occurred in the base metal of the steel top flange in the heat affected zone (HAZ) of the weld. The fatigue cracks of these studs were typical, protruding deep into the top flange from the front and back edge of the stud, and meeting at the last attached region. The studs are believed to pivot from this attached area, with enough flexibility to avoid being severed from the base metal altogether. The remaining area of each stud was between 10% and 30% of the original un-cracked area.

Figure 5: Critical Stud Group in Autopsy of Specimen S3

4.2 Stud Failure

An important discussion is warranted on the topic of stud failure definition and detection. The criterion for failure could be the onset of a decrease in stud capacity, the start of a fatigue crack, all the way to the complete severance of the stud with no capacity remaining. This criterion is important, but can also be hard to enforce, as not all these points can be identified in an instrumented test (it would be near impossible to detect failure in the field with no instrumentation). For our research purposes thus far, we have defined failure as the point when the stud begins to carry less load. It is believed from the observation of test results that this occurs after the initial fatigue crack begins to form. The crack may be in the stud shank, the weld, or in the base metal (beam flange).

Figure 6 is instrumental to the explanation and interpretation of stud failure. For proper understanding of the data, one must understand where it is coming from. This data, termed “local distortion” data, is harvested from a strain gauge placed on the underside of the top flange of the steel beam underneath the location of a welded shear stud. The strains it captures are due to local bending of the top flange, as shown in Figure 6 on the right hand side. This data has proven to be the most reliable means of capturing changes in the load carrying of individual studs over time. It can be seen from Figure 6 that as testing progresses (along the x-axis), the local distortion increases to some maximum value and then begins to decrease. In an idealized setting these local distortions would decrease to zero when the stud is severed from the top flange. In reality, these distortions would not go to zero, even if the stud was fully severed, because of the shape of the fatigue cracks; mechanical interaction results due to the interlocking shapes of the stud and the flange at the failure surface, resulting in residual capacity to transmit shear, even after fatigue failure.

An important step in validating this definition of failure was to stop some of the tests as we noticed local distortions rising, but not yet peaking. The bend test was then performed on these studs to test the strength and area remaining. It was found that in all cases before local distortions peaked, the stud in question passed the bend test, exhibiting strength and ductility. This strength and ductility existed despite visible crack growth in most cases. It is tougher to see in Figure 6 because of the load level applied to Specimen S3, but the start of rising local distortion strains seems to correspond to the onset of cracking in the HAZ in other specimens.

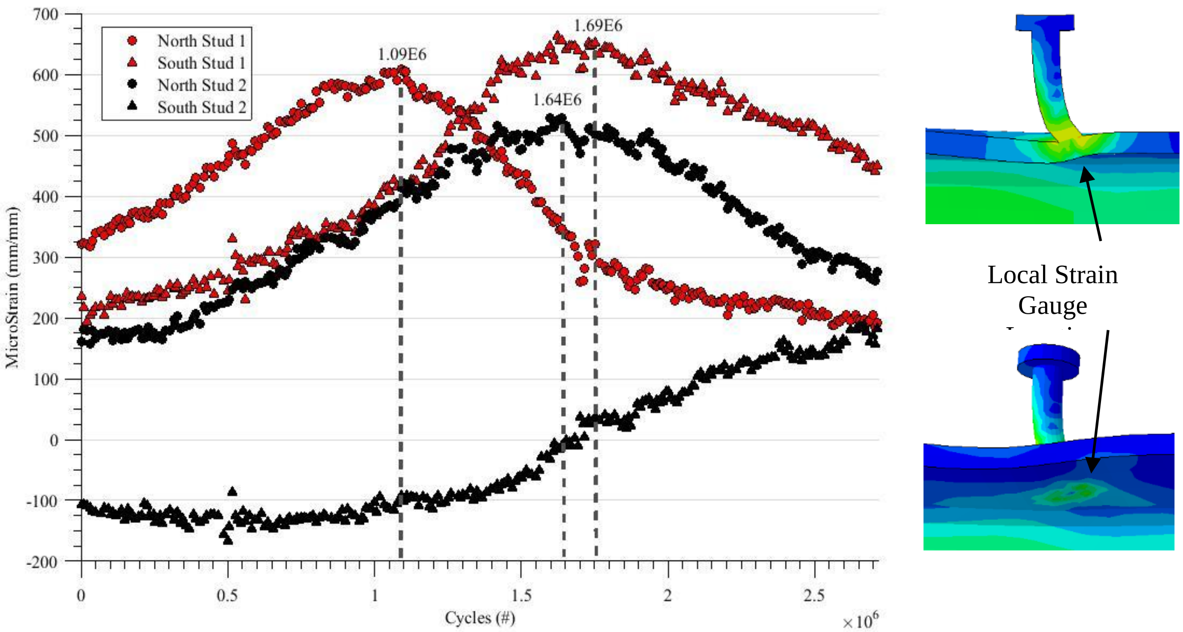

Figure 6: Local Strain Gauge Data for the Four Studs in Pocket 1 of Specimen S3

4.3 S-N Plot

The peak of each local distortion data set was used to indicate the cycle count corresponding to failure. The steady decrease in the stress range as the test continued indicated the connector's ability to distort the top flange as the crack propagated. The result from the first test is displayed on the S-N plot provided in Figure 7.

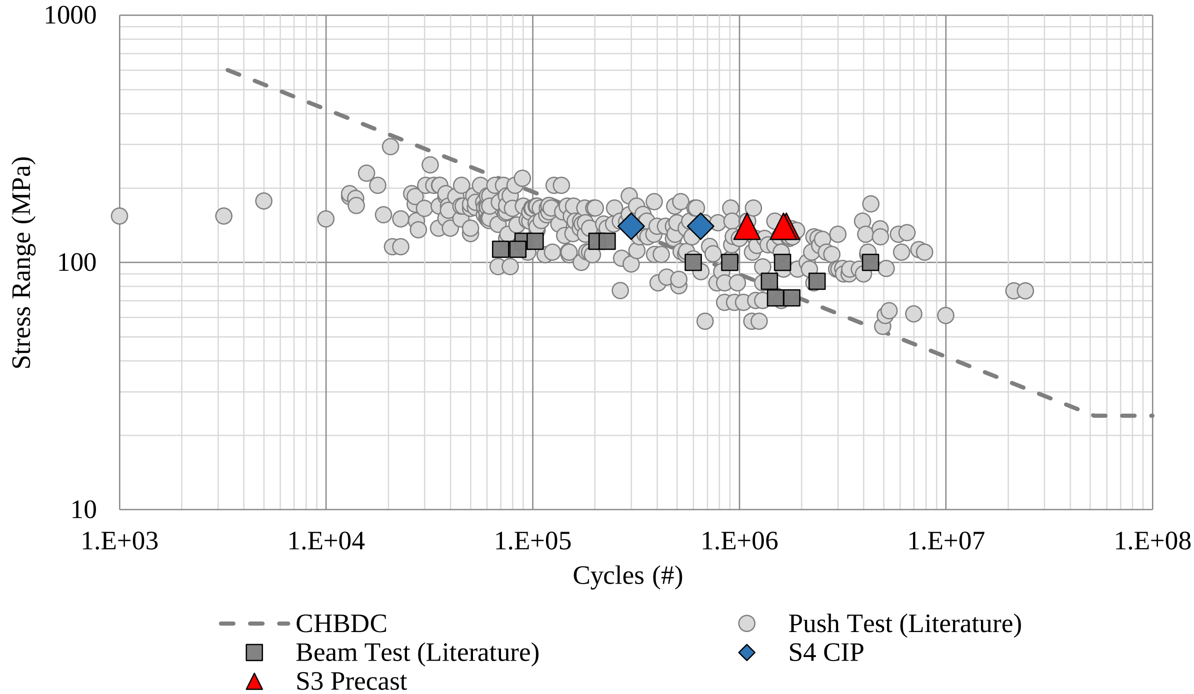

Figure 7: Preliminary S-N Results from a Precast (red) and CIP (blue) Specimen

Figure 7 also includes the results from a CIP specimen (S4), as well as previous fatigue results from push-out tests and beam tests found in the literature, and the design curve provided by the CHBDC (CSA S6) for stud shear connectors. For Specimen S4 with a connector stress of $\Delta\tau$ = 140 MPa, the studs failed at approximately 400,000 cycles on average. This constitutes to a 54% increase compared to the code fatigue design life of 260,000 cycles. For Specimen S3 however, the average failure was closer to 1.4 million cycles or a more than 450% increase compared to the code predicted fatigue life.

5. CONCLUSIONS

The need for beam tests to determine shear connector fatigue failure has been highlighted by researchers and code makers alike, and until now the costs and time required have prevented a full data set. By studying the differences in shear connection fatigue behaviour between conventional CIP construction and state-of-the-art precast panel construction, this ongoing testing will both add to the knowledge base of conventional stud failure mechanisms, and provide insights into whether or not precast panels can be designed using the same rules, with no long term durability drawbacks. To this point, redistribution of forces has been observed in beam tests, delaying full fatigue failure and providing structural redundancy. Additionally, it has been observed that local strain gauges can be used to predict, with some level of accuracy, the behaviour and stress state of stud shear connectors while they are embedded in concrete in an “un-inspectable” state. Judgements comparing the current fatigue results with each other and with push-out tests from the literature should be withheld until more data is available, knowing the tendency of fatigue data to be prone to large statistical scatter. Nevertheless, the path forward promises to provide insights for designers into the long term durability of steel-precast composite bridges.

ACKNOWLEDGEMENTS

The authors would like to thank the Steel Structures Education Foundation (SSEF), the Canadian Institute of Steel Construction (CISC), David Lai and the Ministry of Transportation of Ontario (MTO), and NSERC for providing research funding and support.

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