On the Fatigue Capacity of Headed Shear Studs in Composite Bridge Girders

Ovuoba, B.¹ and Prinz, G.S.²*

Abstract

Shear connectors are commonly used in steel bridges to join the concrete deck and steel superstructure, providing a mechanism for shear transfer across the steel-concrete interface. The most common type of shear connector is the headed shear stud. In the current AASHTO LRFD bridge specifications on composite design, shear stud fatigue often governs over static strength, and a large number of shear connectors often result. The stud fatigue capacities presented in the AASHTO standard are largely based on a limited sample of composite fatigue tests performed in the 1960s, with limited fatigue test data at lower stress ranges leading to a somewhat arbitrary constant amplitude fatigue limit (CAFL). The somewhat arbitrary CAFL often governs the composite design for bridges with moderate-to-high traffic demands. This paper presents results from an experimental and analytical study into the fatigue behavior of headed shear studs, to address the lack of existing experimental data near the assumed CAFL, and to better characterize the effects of fatigue uncertainty on predicted response. Results from composite push-out specimens tested at stress ranges near the existing AASHTO CAFL suggest an increase of the limit to 44.8MPa (6.5ksi). Recommendations for modification to the existing AASHTO shear stud S-N fatigue capacity curve are proposed.

Keywords: Shear studs, S-N curve, CAFL, bridges, fatigue capacity, composite design

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1. Introduction

Shear connectors are commonly used in steel bridges to join the concrete deck and steel superstructure, providing a mechanism for shear transfer across the steel-concrete interface. Joining the steel and concrete members is advantageous, as the composite steel-concrete section has added strength over the sum of its individual components (the steel girder and concrete deck). This allows for use of lighter steel members and improved economy. The most common type of shear connector is the headed shear stud (see Figure 1(a)).

In the current AASHTO LRFD bridge specifications on composite design, shear studs must satisfy both strength and fatigue requirements [1]. To satisfy strength requirements, the shear connection between the concrete and steel elements must be capable of developing the full plastic capacity of the steel cross-section (create full composite action). To satisfy fatigue requirements, demands at the steel-concrete interface must be lower than the shear stud fatigue capacity as determined from an empirical fatigue capacity curve (called an S-N curve) and anticipated traffic cycles. Fatigue often governs, and a large number of shear connectors often result (see Figure 1(b)). Because traffic cycles are typically fixed from average daily truck traffic extrapolated over a 75-year design life, the S-N curve ultimately determines the required number of shear studs when the design is governed by fatigue.

While many studies have investigated shear stud fatigue [2,3], the stud fatigue requirements in the AASHTO standard are largely based on single-sided push-out tests on 19mm (3/4 in) studs performed by Slutter and Fisher [4]. In the study by Slutter and Fisher, 26 samples containing 19mm (3/4in.) diameter studs were fatigue tested under constant amplitude stress cycles ranging in value from 55MPa (8ksi) to 138MPa (20ksi). To relate the applied stress range to the expected number of cycles for stud fatigue failure (S-N curve equations), a least-squares regression approach was used. Equation 1 presents the stud capacity equation based on the 26 data points from Slutter and Fisher [4], which shows similarity with the current stud fatigue capacity presented in the AASHTO standard [1] (see Equation 2). Note that the least-squares
approach for regression analysis fails to account for any uncertainty distribution in the fatigue response, and therefore prevents the creation of characteristic capacity curves having known confidence levels.

\[
\log(N) = 8.072 - 0.1753\Delta\sigma \\
\log(N) = 8.061 - 0.1834\Delta\sigma
\]

(Eq. 1) Slutter and Fisher [4]  
(Eq. 2) AASHTO [1]

The current AASHTO fatigue requirements assume a lower shear stud fatigue capacity than comparable specifications throughout the world. Figure 2 shows the current AASHTO shear stud S-N curve along with a comparable curve from the European (Eurocode) standard [5]. Other shear stud S-N curves from the Japanese and British standards are similar in form to the Eurocode curve [6]. The AASHTO specification results in a lower estimation of stud fatigue capacity for all traffic demands, and considers a linear-log regression while the Eurocode, Japanese, and British standards consider log-log fatigue behavior. Note in Figure 2 that the AASHTO S-N curve considers a somewhat arbitrary constant amplitude fatigue limit (CAFL) at 24MPa (3.5ksi).

Limited fatigue test data exist at these lower stress ranges to justify this CAFL location, which often governs the stud fatigue design for bridges with moderate-to-high traffic demands (ADTT greater than approximately 960 vehicles). Comparing the required number of studs in a rural short span steel bridge design (having a span of 17.3m (57ft)) at various levels of truck traffic, Lee et al. [6] found that bridges designed to the US requirements needed nearly twice as many shear studs than the corresponding European, British, and Japanese designs. In [6], stud capacities for the US designs were always governed by fatigue requirements.

This paper presents an experimental and numerical study into the behavior of headed shear studs, to address the lack of existing experimental data near the assumed CAFL, and to better characterize the effects of fatigue uncertainty on predicted response. In this study, composite push-out specimens are fatigue tested at stress ranges near the existing AASHTO CAFL and a probabilistic approach is applied to both new and existing fatigue data to capture uncertainty and variation in the fatigue response. The paper begins by describing the experimental study, including the specimen geometry, test setup, instrumentation, and
loading. Following, the experimental fatigue results are presented and a probabilistic method for fatigue data evaluation is described. Findings from the experimental study are combined with existing data from previous studies to provide a comprehensive data set for re-evaluation of shear stud fatigue capacity. A characteristic S-N curve for estimating shear stud fatigue capacity is proposed and applied to five prototype bridge designs to provide comparison.

2. Experimental Program

The primary objectives of the experimental program are to 1) characterize stud fatigue capacity at low applied stress ranges, 2) re-evaluate the existing CAFL considering both run-out and failure test results, and 3) investigate stud crack formation during low-stress high-cycle fatigue loading.

2.1. Test Specimen Geometry and Fabrication

Figure 3 shows the experimental push-out specimen geometry, consisting of a rolled W10×54 wide-flange section having 4 headed shear studs and a 6 in. cast-in-place concrete slab on each flange. The chosen geometry for the specimens (called herein double-sided push-out specimens) is based on guidelines for shear-stud testing prescribed in the Eurocode [7]. Double-sided push-out specimens are advantageous over single-sided push-out specimens (having a slab on only one side) as they help reduce loading eccentricities and multi-axial stress states within the stud (combined tension and shear). An applied multi-axial stress state in the stud can provide an overly-conservative estimation of fatigue capacity [3,6,8]. In this study, a total of 6 double-sided push-out fatigue tests are performed at four different applied stress levels ranging in value from 30MPa to 60MPa (4.4ksi to 8.7ksi). Due to the significant time associated with high-cycle fatigue testing, only two replicate stress-ranges are considered in the test matrix (replicates at 40MPa (5.8ksi) and 60MPa (8.7ksi)).

Concrete slabs of the test specimen are designed to represent typical composite bridge conditions. All concrete sections consider normal weight concrete from a standard highway bridge deck mix design [9], and each concrete section is cast with the beam in a horizontal position (see Figure 4). Note that while mean specimen concrete strengths of between 40Mpa (5.8ksi) and 57MPa (8.2ksi) are at the upper end of
those tested by Slutter and Fisher [4], concrete strength has been shown to have little influence on stud
fatigue life [4]. To ensure material consistency across the four different stress levels tested, four push-out
specimens are simultaneously cast from the same concrete batch. Prior to each fatigue test, adequate
crushed concrete compressive strength (at least 80% $f'_{c}$) is checked from concrete cylinders formed during casting.
To prevent adhesion between the concrete and steel, which can contribute to load transfer across the steel-
concrete interface, each steel flange was coated in grease prior to concrete casting [7].

2.2. Test Configuration, Instrumentation, and Loading

The experimental setup, shown in Figure 5(a), is designed to apply rapid shear stress cycles to studs
within the push-out specimens. As shown in Figure 5(a), the double-sided push-out specimens are loaded
with the beam oriented vertically, and the axial loads applied to the end of the steel wide-flange section.
All specimens are subjected to unidirectional loading (specimens are loaded in one direction and then
unloaded), resulting in a non-zero mean stress and providing a conservative fatigue loading condition as
compared to reversed cycle loading [4]. To prevent separation between the specimen and testing machine
at unloading, a pre-load of 1kN (0.22kip) is maintained (somewhat shifting the applied mean stress). To
ensure uniform contact between the concrete slabs and testing machine base, each specimen was leveled
using a gypsum grout mixture.

Linear variable differential transducers (LVDTs) and unidirectional strain gauges are used to
provide local measurements during testing. A total of eight LVDTs oriented parallel and perpendicular to
the beam axis are included on each test specimen, to measure relative slip and separation between the
concrete and steel sections. Unidirectional strain gauges are applied on two specimens to measure shear
stresses transferred through the studs. Figure 5(b) shows the specimen instrumentation, including LVDT
placement and strain gauge configurations.

Table 1 presents the experimental test matrix, including the specimen concrete strength, applied stress
range, loading rate, and the resulting fatigue capacity. In Table 1, the applied stress ranges vary between
30MPa (4.4ksi) and 60MPa (8.7ksi) with specimen loading rates applied at between 10Hz and 20Hz. These
high frequency loading rates are possible due to the high stiffness of the loading frame and test specimens. Note that measurements from several pseudo-static loading cycles applied at 1 Hz were used to verify negligible inertial effects at the higher frequency loading (see [10]). Fatigue results provided in Table 1 will be discussed in the following Results section.

3. Experimental Results

3.1. Observations and Measured Fatigue Life

All specimens tested demonstrated higher fatigue capacity than the existing AASHTO limit (see again Equation 2), with the only complete fatigue failures occurring in Specimen 1 having an applied stress range of 60MPa (8.7ksi). Failure in Specimen 1, evidenced by a complete fracture of the four embedded studs, occurred after 12.8 million cycles. In Specimen 1, fractures originated at the base of the stud weld (see Figure 6(c)) and propagated into the beam flange leaving crater-like indentations in the flange as shown in Figure 6(a). This failure mode is similar those observed in other push-out tests [4,11,12] and resulted in little-to-no damage to the concrete surrounding the stud. Specimen 2 (loaded at a stress range of 30 MPa (4.4ksi)) and Specimen 5 (loaded at a stress range of 50 MPa (7.3ksi)) survived more than 30 million cycles prior to being declared runouts. Specimens 3 and 4 loaded at 40 MPa (5.8ksi) were also declared runouts after 12.25 million and 20 million cycles respectively. The resulting fatigue capacities for all eight double-sided push-out specimens are provided in Table 1.

Slip between the concrete slab and steel beam was observed for all test specimens; however, for specimens loaded at stress ranges at or below 50MPa (7.3ksi) this slip was minor over then entire cycle history. Figure 7 shows the average slip for each slab of Specimens 5 and 1 (loaded at 50MPa and 60MPa respectively). Slip measurements for other specimens having lower applied stress ranges were similar to Specimen 5. The slip values presented in Figure 7 are computed by averaging the two LVDTs on each beam flange, providing a single slip value for each slab. In Figure 7 a noticeable slip in slab 1 of Specimen 1 occurs near 3 million cycles, followed by an increase in the slip-per-cycle rate up to failure of the studs at 12.8 million cycles. Slip between the concrete slab and steel beam is an indication of stiffness loss and
possible stud damage. Specimen 5, subjected to a lower applied stress range, experienced minimal slip (suggesting little stud damage) over the entire 30 million cycle loading. While slip measurements are helpful in estimating damage within the embedded studs over time, more detailed investigations are required to determine whether fatigue cracks actually exist.

3.2. Stud Fatigue-Crack Investigations

Metallographic investigation and micro-hardness testing of stud cross-sections cored from completed tests indicate fatigue crack initiations within runout specimens and a critical fracture location near the stud-to-flange weld heat affected zone (HAZ). Stud samples cored from runout Specimens 2 and 5, were sectioned, polished with abrasive paper and diamond powder of increasing fineness (mirror polished to a surface roughness of 1 \( \mu \)m), and then surface etched with a Nitol solution (5ml HNO3 per 100ml of ethanol). Figure 8 shows the polished stud cross-sections taken from the specimens with the various weld features highlighted, including the weld HAZ, base metals (BM), and fusion zone (FZ). Vickers micro-hardness measurements (shown as contours in Figure 8) highlight material property changes (potential changes in material toughness) within the welded stud-to-flange zone and confirm the location of the HAZ, FZ, and BM. Stud sections taken from Specimen 2 (declared a runout after more than 30 million cycles at 30MPa (4.4ksi)) show no indication of fatigue crack initiation (see Figure 8(a)); however, samples taken from Specimen 5 (declared a runout after more than 30 million cycles at 50MPa (7.3ksi)) indicate fatigue cracks initiating near the weld HAZ at the stud-to-flange interface (see Figure 8(b)). The initiated fracture observed in Specimen 5 closely resembles the initial fracture path shown in Figure 6(c) for failure in Specimen 1. These initiated fatigue cracks were present in all studs cored from Specimen 5.

4. Probabilistic Approach to Shear Stud Fatigue Capacity Evaluation

Scatter in fatigue test results is inevitable, and can provide uncertainty when predicting fatigue performance. When creating S-N curves for fatigue prediction, quantifying this uncertainty and maximizing the likelihood of predicting an experimental outcome is desired. In the regression analysis by Slutter and Fisher [4] (on which the current AASHTO stud capacity limits are based), S-N curves for shear stud fatigue
capacity were created using a simplified least-squares fitting procedure incapable of quantifying the uncertainty in the experimental scatter. In this section, an alternative curve creation approach is proposed, wherein a more robust statistical method called maximum likelihood estimation (MLE) is used to create S-N curves that maximize the joint probability of predicting the observed experimental result. Several studies have successfully used MLE to define curve regressions for large data sets [13,14,15,16]. The following paragraphs describe a random fatigue limit model proposed by Pascual et al. [17] using the MLE method. The newly generated shear stud fatigue data is combined with existing data from the previous studies and analyzed using the random fatigue limit model. A new characteristic shear stud S-N curve considering data uncertainty and having a known confidence level is proposed.

4.1. Overview of MLE

The goal of the MLE approach is to identify a population (probability distribution) at each stress level that is most likely to have generated the experimental data. To achieve this, parameters for the population are chosen that maximize the joint probability of predicting failure at all points (or in other words, to maximize the likelihood of predicting failure at all points). This joint failure probability (or likelihood) is simply the product of every data-point failure probability, written as:

\[ L = \prod_{i=1}^{n_f} f_{N_i} \cdot \prod_{i=1}^{n_r} R_{N_i} \]  

(Eq. 3)

where, \( L, f_{N_i}, R_{N_i}, n_f \) and \( n_r \) are the likelihood, probability of predicting failure at an individual data point \( (i) \), the probability of predicting run-out at an individual data point \( (i) \), the total number of failure data-points, and the total number of run-out points respectively.

In this study, a nonlinear generalized reduced gradient optimization algorithm is used to maximize the likelihood given by the variable parameters in the regression model. In determining the individual failure probabilities required in Equation 3, a power-law relationship is assumed to appropriately represent the fatigue data [18,19]. This power-law relationship is given in Equation 4,

\[ \log_e N = \alpha + \beta \log_e (S - \gamma) \]  

(Eq. 4)
where $N$ is the number of cycles to failure at a given applied stress range, $S$. Parameters $\alpha$ and $\beta$ in Equation 4 are unknown parameters to be determined through MLE and $\gamma'$ is the assumed CAFL, also to be determined through MLE. Note that for a confidence level of 50%, $\gamma'$ in Equation 4 will be equal to the mean, $\mu_{\gamma'}$, of the CAFL distribution. For other curve confidence levels, $\gamma'$ is taken as $\mu_{\gamma'} - z^* \sigma_{\gamma'}$ (shifting the CAFL location $z^*$ standard deviations from the mean). Equation 5 presents the regression relationship that can be used for confidence levels other than the mean, and Figure 9 depicts the MLE based model assuming the above power-law relationship and considering normally distributed data at each stress-range level.

$$\log_e N = \alpha + \beta \log_e (S - (\mu_{\gamma'} - z^* \sigma_{\gamma'})) - z^* \sigma$$  \hspace{1cm} (Eq. 5)

The probability of having failure at each data point $(N_i, S_i)$ in Figure 9, given a specific CAFL value $(\gamma')$ and assuming the data at each stress-range level as normally distributed, is given by the conditional probability density function shown in Equation 6.

$$f_{N_i,\gamma'} = PDF_{N_i,\gamma'} = \left(\frac{1}{\sqrt{2\pi \cdot \sigma_{\gamma'}}}\right) \exp\left(\frac{-1}{2\sigma_{\gamma'}^2} \cdot [N_i - \alpha - \beta \log(S_i - \gamma')]^2\right)$$  \hspace{1cm} (Eq. 6)

Because $f_{N_i,\gamma'}$ assumes a given $\gamma'$, the probability that $\gamma'$ exists ($f_{\gamma'}$) must also be determined (see Equation 7). The resulting probability of predicting failure at $N_i$ is the marginal probability density function representing the joint probability between $f_{N_i,\gamma'}$ and $f_{\gamma'}$ as given in Equation 8.

$$f_{\gamma'} = \left(\frac{1}{\sqrt{2\pi \cdot \sigma_{\gamma'}}}\right) \exp\left(\frac{-1}{2\sigma_{\gamma'}^2} \cdot [\gamma' - \mu_{\gamma'}]^2\right)$$  \hspace{1cm} (Eq. 7)

$$f_{N_i} = PDF_{N_i} = \int_{\gamma'}^{\gamma'} f_{N_i,\gamma'} \cdot f_{\gamma'} \cdot d\gamma'$$  \hspace{1cm} (Eq. 8)

### 4.2. Influence of Run-Outs on CAFL

Many S-N curves often only consider failure test results in identifying regression parameters, neglecting run-out test results and their potential influence on curve features such as the CAFL. At certain low stress levels, such as those considered in this study, the possibility exists for run-out test results to occur. MLE allows these run-out test results to influence the S-N curve through the population cumulative distribution
function, since run-out simply indicates the absence of failure. In the case of run-outs, the probability of predicting run-out given an assumed CAFL ($\gamma'$) is given by:

$$R_{N_i|\gamma'} = 1 - CDF_{N_i,S_i|\gamma'}$$  \hspace{1cm} (Eq. 9)$$

where $CDF_{N_i,S_i|\gamma'}$ is the cumulative density function assuming $\gamma'$. The resulting probability of run-out, $R_{N_i}$, is the marginal probability density function between Equation 9 and Equation 7, given by equation 10.

$$R_{N_i} = \int_{0}^{\infty} (1 - CDF_{N_i,S_i|\gamma'}) \cdot f_{\gamma'} d\gamma'$$  \hspace{1cm} (Eq. 10)$$

**4.3. Shear Stud Fatigue Dataset and Analysis using MLE**

The complete fatigue data set considered in this study consists of the six fatigue results described earlier and 100 fatigue results from existing comparable testing found in the literature (see [10] for fatigue data tables). The 100 fatigue results taken from the literature were from a total of seven shear stud fatigue studies conducted between 1959 and 1988 [4,11,20,21,22,23,24]. All existing fatigue data were selected based on four criteria, including: 1) a stud shank diameter of 19mm (3/4 in.); 2) constant amplitude loading; 3) unidirectional loading (no reversed cycles), and 4) failure occurring in the stud-shank or weld (i.e. no concrete crushing failures). For conservancy, test results from reversed cycle loading were not included, as they typically result in higher fatigue capacities due to the reduced applied mean stress [4]. Fatigue results from both single-sided and double-sided push-out tests were considered. Additional test data for 12.7mm (1/2 in.), 22mm (7/8 in.), and 32mm (1-1/4 in.) studs from other studies are used in comparisons [4,8,12,25,26,27,28].

Analysis of the fatigue dataset suggests an increase in the CAFL and higher fatigue capacity within the finite-life region for stress-ranges over 117MPa (17ksi). Equation 11 presents the stud fatigue capacity equation resulting from the MLE analysis, with the optimized parameters of $\alpha$, $\beta$, $\mu_p$, $\sigma$, and $\sigma_g$ being 17.26, -2.09, 6.5ksi, 1.45, and 1.21ksi respectively. Note that the stress range parameter in Equation 11 is based on units of ksi. In Equation 11, the mean CAFL value of 44.8MPa (6.5ksi) suggests an 86% increase in the allowable stress range for infinite life as compared to the current AASHTO standard. Note that analysis of
the data considered uniformly distributed data at each stress-range level, and a mean confidence level (50%) based on the inherent conservancies in fatigue data resulting from push-out specimens [4,6,27].

\[ \log_e N = 17.26 - 2.09 \log_e (S - 6.5) \]  
(Eq. 11)

Figure 10 shows the resulting regression from the MLE analysis. For comparison, the current AASHTO shear stud S-N curve is also plotted along with the considered fatigue data-set.

5. Proposed Design S-N Curve for Predicting Shear Stud Fatigue Capacity

Given similarities in form between the MLE S-N curve and the S-N curves for various steel bridge fatigue details provided in the AASHTO standard, a simplification of Equation 11 is proposed to provide consistency in design. In AASHTO [1], the design load-induced fatigue resistance for bridge details (excepting fatigue of the stud) takes the form:

\[ (\Delta F)_a = \left( \frac{A}{N} \right)^{\frac{1}{m}} \geq (\Delta F)_{TH} \]  
(Eq. 12)

where \( m \) and \( A \) are constants representing the slope and intercept of the fatigue S-N curve. In Equation 12, \( (\Delta F)_{TH} \) is the CAFL, and \( (\Delta F)_a \) is the allowable stress range. To adapt Equation 11 to the form provided in Equation 12, a bi-linear design S-N curve is fit to the power-law relationship determined through MLE using the CAFL asymptote and approximate tangent at 103MPa (15ksi). This simplification provides an avenue for consistency between shear stud fatigue capacities and standard fatigue detail capacity forms.

Table 2 presents the proposed detail category description, including the proposed S-N curve constant (\( A \)), slope (\( m \)), threshold value (CAFL or (\( \Delta F \))_{TH}), description of the potential crack initiation point, and an illustrative example of potential damage.

Figure 11(a) shows the proposed design S-N curve along with the MLE regression and fatigue data, and Figure 11(b) compares the proposed bi-linear design S-N curve with the current AASHTO fatigue detail categories. Note in Figure 11(b), that the proposed stud fatigue design S-N curve indicates a lower fatigue capacity than the curve for fracture in the base metal outside the stud weld (Category C), but a higher capacity than the current AASHTO stud fatigue limit. While the proposed design S-N curve is derived
from the MLE analysis on 19mm (3/4 in) stud fatigue tests, data from other fatigue tests on 12.7mm (1/2 in), 22mm (7/8 in), and 32mm (1-1/4 in) studs fit the general trend of the curve and fall within the scatter of the 19mm (3/4 in) results. For comparison, Figure 11(c) is provided to show the proposed design S-N curve with data from 12.7mm (1/2in), 19mm (3/4 in.), 22mm (7/8 in), and 32mm (1-1/4in) stud fatigue test results.

6. **Effect of New Shear Stud Fatigue-Life Equation on Composite Bridge Design**

To compare the effect of the proposed shear stud fatigue life equation on the existing composite design requirements, design calculations investigating the required number of studs are performed. All design examples in this report use existing bridge geometries of varying span lengths and consider both strength and fatigue limit states.

6.1. **Prototype Bridge Geometries & Considered Traffic Levels**

Five multi-girder highway bridge span geometries, two levels of single-lane average daily truck traffic (ADTTSL), and two fatigue criteria (the proposed and existing stud fatigue capacities) are considered for a total of 20 composite designs. The two ADTTSL values considered are 500 and 1000, representing low and high traffic demands for both finite and infinite life stud designs. The five bridge span geometries are taken from existing multi-girder highway bridges located within Arkansas and Oklahoma. Each existing bridge span is simply-supported and originally designed to be composite. Details on span length, concrete deck depth, girder spacing, and girder geometry, for each of the five spans are provided in Table 3. All designs in this study consider 19 × 102mm (3/4 × 4 in.) shear studs.

6.2. **Design Approach**

Excepting the designs considering the proposed shear stud fatigue capacity, all shear studs in the prototype bridge geometries are designed in accordance with AASHTO Article 6.10.10. Following typical design practices, shear demands at the steel-concrete interface are calculated at every 10th point along the bridge span allowing the stud pitch to change along the span. The following sections provide a brief background on Article 6.10.10 in the AASHTO standard.
6.2.1. Brief Background on Determining the Required Stud Pitch for Fatigue

Under fatigue loading, the required pitch (or spacing) of shear studs in a composite bridge girder is dependent on a capacity-to-demand ratio between the individual stud capacity \( Z_r \) and the applied shear demand at the steel-concrete interface \( V_{sr} \). Equation 13 presents the capacity-to-demand ratio used by [1], with \( n \) being the number of shear studs across the flange width. While the stud fatigue capacity \( Z_r \) is calculated as described earlier (shown again in Equation 14), the demand at the steel-concrete interface is calculated as a shear flow resulting from the girder shear \( V_f \) caused by the passage of a design-level fatigue truck (see Equation 15).

Two fatigue load combinations are considered when calculating shear demands from the design-level truck: Fatigue Load Combination I for higher traffic demands (ADTT_{SL} greater than 960) and Fatigue Load Combination II for lower traffic demands (ADTT_{SL} lower than 960). The loading factors for Fatigue Load Combinations I and II are 0.75 and 1.5 respectively [1].

\[
p \leq \frac{n \cdot Z_r}{V_{sr}} \quad \text{(EQ. 13)}
\]

\[
Z_r = \begin{cases} 
(34.5 - 4.28 \log N) \cdot d^2 & \text{for ADTT < 960} \\
5.5 \cdot d^2 & \text{for ADTT > 960}
\end{cases} \quad \text{(U.S. Cust. units)} \quad \text{(EQ. 14)}
\]

\[
V_{sr} = \frac{V_f \cdot Q}{l} \quad \text{(EQ. 15)}
\]

6.3. Comparison of Composite Bridge Designs using the AASHTO and Proposed Stud Fatigue Capacities

The proposed stud fatigue S-N curve results in fewer required studs than the existing AASHTO S-N curve. Table 4 shows the number of shear studs required to satisfy the fatigue limit state in the five prototype bridge geometries, considering both the AASHTO and proposed stud S-N curves. Note in Table 4 that stud values using the proposed S-N curve remain unchanged between the ADTT 1000 and ADTT 500 levels as the CAFL is shifted relative to the AASHTO curve. For the higher traffic level (ADTT of 1000), the number of required shear studs was reduced by 40-45% using the proposed S-N curve. For the
lower traffic level (ADTT of 500) under the lower traffic level, the number of required studs was reduced by between 17-48%.

That the required number of studs is somewhat higher than that required by the capacity-to-demand ratio provided in EQ 13, due to the maximum stud spacing requirement of 610mm (24 in.) [1]. If the maximum stud spacing requirement were increased, further reductions in the number of required shear studs would be achieved.

7. Summary and Conclusions

In this study, six composite push-out specimens were fatigue tested under repeated cyclic loads at stress ranges varying between 30MPa (4.4ksi) and 60MPa (8.7ksi). These composite push-out specimens represent a conservative estimation of stud fatigue damage as the adhesion and friction at the steel-concrete interface were inhibited by greasing of the steel flanges prior to concrete casting. Measured fatigue life from the eight specimens were combined with existing shear stud fatigue data sets in the literature, and analyzed using a probabilistic method called maximum likelihood estimation. Results from the eight fatigue tests and analysis of the new and existing fatigue data provide the following conclusions:

1) The current AASHTO CAFL for headed shear studs provides an overly-conservative estimation of fatigue capacity. Analysis of existing data along with the additional high-cycle fatigue test results suggests an increase in the CAFL from 24MPa (3.5ksi) to 44.8MPa (6.5ksi).

2) The current AASHTO S-N curve for finite life of the shear stud underestimates fatigue capacity and is not representative of the larger considered fatigue dataset. An alternative design S-N curve of similar form to the existing AASHTO detail categories (log-log form) is proposed. The proposed curve of the form

\[(\Delta F)^n = \left(\frac{A}{N}\right)^m \geq (\Delta F)^{n}\]

has an \(m=4\) and \(A= 150 \times 10^8\) and provides a known level of confidence in the estimated fatigue capacity (based on the MLE analysis with a confidence
level of 50%) while providing a unification in the fatigue design procedure. Note that stress range capacities provided in the proposed equation were derived using imperial units of ksi.

3) For bridge designs subjected to high traffic levels, the number of required shear studs may be reasonably reduced by 40-45% using the proposed design S-N curve. These stud savings will vary with girder geometry due to the maximum stud spacing requirement of 610mm (24in.). If the maximum stud spacing requirement were increased, further reductions in the number of required shear studs could be achieved.
8. References


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<tr>
<th>Specimen Number</th>
<th>Average Concrete Compressive Strength [MPa]</th>
<th>Applied Stress Range [MPa]</th>
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## Table 2

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<tr>
<th>Description</th>
<th>Category</th>
<th>Constant A (ksi^4)</th>
<th>Threshold (ΔF)/n ksi</th>
<th>Potential Crack Initiation Point</th>
<th>Illustrative Examples</th>
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<tbody>
<tr>
<td>Connection weld or shank of stud-type shear connector attached by fillet or automatic stud welding subjected to shear loading</td>
<td>D' [m = 4]</td>
<td>150 ×10^4</td>
<td>6.5</td>
<td>Toe of stud-to-flange welds, propagating through the stud shank or into the flange base metal</td>
<td><img src="image" alt="Illustrative Examples" /></td>
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<tr>
<td>Location</td>
<td>Span Length (ft)</td>
<td>Concrete Deck Depth (in)</td>
<td>Girder Spacing (ft)</td>
<td>Girder Section</td>
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\textsuperscript{a} See calculation examples provided in Appendix D for site-specific bridge location information
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<td>149</td>
<td>153</td>
</tr>
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</table>

Note: Values presented are adjusted to satisfy maximum stud spacing requirements.
Concrete cast with beam in horizontal position

Formwork

Flange greased to limit adhesion
Figure 6

a) Steel Flange Surface

b) Concrete Surface

c) Removed Shear Stud
Figure 7: Graph showing average slip in millimeters against cycles for two slabs labeled Slab 1 and Slab 2. The graphs are for Specimen 1 (Δσ = 60 MPa) and Specimen 5 (Δσ = 50 MPa). Failure is indicated at 12.8 x 10^6 cycles.
Figure 8

Variation in material microstructure within weld region

a) Specimen 2

Stud removal using coring machine

Loading Direction

Crack penetrating into flange base material

Fatigue crack initiating near weld HAZ & geometry defect

b) Specimen 5

Stud Shank

Flange

Vickers

Hardness

0

100

200

300

Click here to download Figure 8.pdf
\[ \mu = \alpha + \beta \cdot \log(S-\gamma) = \log(N) \]

Assumed normal distribution of fatigue data

\[(N_i, \theta_i)\]

Fatigue Limit

\[ \mu_f \]

Fatigue Life, N [Cycles]
Stress Range, $S$ [ksi] vs. $N_f$

- CAFL distribution from failure and runout test data
- $\mu_f = 6.5$ ksi
- AASHTO = 3.5 ksi

- 50% Conf.
  - X Failures
  - ● Run-out

Figure 10: CAFL distribution from failure and runout test data.
Stress Range, $S$ [ksi]

- Proposed Design: $\frac{150 \times 10^6}{N} \geq \Delta F$ [ksi] = 6.5 ksi

<table>
<thead>
<tr>
<th>Detail Category</th>
<th>$m$</th>
<th>$\Delta_f$</th>
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<tr>
<td>C</td>
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Figure 11

(a) Proposed Design Curve

(b) 7/8" Stud data

(c) Proposed: D'