Title: Reliability Analysis of Welded Stud Shear Connectors on Simply-Supported Bridge Girders

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Abstract:

This paper examines the reliability of welded stud shear connectors for steel-concrete composite bridge girders. A finite element model of a simply-supported bridge was created featuring link connector elements representing the shear studs between beam and shell elements, representing a steel girder and concrete deck, respectively. The model is programmed using a program interface to build a model including studs with random strengths. Using this approach, many analyses can be run in succession, with connectors failing between each analysis. This study considers the probabilistic characteristics of the welded studs and truck loading and recognizes the interaction between ULS and FLS. The example bridge employed in this study was designed according to the CSA S6-2014 code provisions. Based on the presented reliability analysis, an increase in the CSA S6-2014 24 MPa endurance limit of at least 1.45 times is found to be acceptable.

Key words:

composite girders; shear connectors; welded studs; fatigue; reliability

Introduction

The most common type of shear connector in use in steel-concrete composite bridge construction today is the welded shear stud, primarily due to its combination of strength, ductility, and ease of installation. However, the welded detail is fatigue critical when used in bridges. Figure 1 shows a typical composite girder with welded stud connectors.

Figure 1. Steel-cast in place (CIP) concrete beam with welded stud connectors.

The design procedure for welded shear studs in Canada is the same as other fatigue-sensitive details, e.g. in the 2014 Edition of the Canadian Highway Bridge Design Code, CSA S6-2014 studs were classified as 'Category D' (CSA 2014a). Each stud is designed for a stress range calculated from elastic theory and compared to the stress range indicated by the design *S-N* curve (stress vs. number of cycles, plotted on a log-log scale). However, studs are fundamentally different than most other fatigue-sensitive details sharing the same design process. Each stud is uninspectable and part of a redundant set of many studs, which together resist force and slip along the shear interface. It follows that, because of their differences, shear studs should be treated differently in design to ensure a compatible level of reliability between them and other details. Evidence of the uniqueness of the stud detail can also be found in the process through which it was assigned a fatigue category in the first place. According to CSA S6.1-2014 (CSA 2014b), fatigue-sensitive details are assigned a detail category from representative tests, which reveal a "characteristic" fatigue curve representing the 97.7% survival probability rate of the detail for conservatism. Shear studs, however, have been assigned a detail category based on the mean curve (50% survival probability rate) from tests called "push tests", which are said to conservatively approximate the actual behaviour of studs in beams.

The current study aims to quantify the reliability of a shear stud connection using CSA S6-2014 (CSA 2014a) design procedures, as well as to propose recommendations in support of a movement towards a different design procedure for welded studs. This movement has already been initiated to some degree, with changes based on expert judgement and independent study by others appearing in the 2019 Edition of CSA S6 (CSA 2019). To accomplish this task, an example bridge is employed. The example bridge illustrates that the fatigue limit state (FLS) can govern design by requiring almost three times as many connectors as required for ultimate limit state (ULS) design. With the close spacing of studs required under the CSA S6-2014 (CSA 2014a) fatigue provisions, bridge constructors would have had difficulty fitting in transverse rebar, and material and labour costs for stud welding would be high.

The example bridge is next modelled using the finite element (FE) analysis software SAP2000. The model is programmed using a Visual Basic application program interface so that individual shear studs can receive probabilistic properties and moving load analyses can be conducted to study the overall structural consequences of successive stud failures in a reliability analysis. After the reliability of the stud shear connection of the example bridge is determined, loads are increased in order to reduce the reliability of the system towards the code target value. To begin, a brief literature review of the reliability of fatigue components and an overview of the existing fatigue data for shear studs is presented, including test results obtained at the University of Waterloo (UW) as part of a recent beam testing program.

Reliability of Fatigue Components

For the design of new bridges, CSA S6-2014 (CSA 2014a) specified a lifetime target value of β of 3.5 (or a 1 in 4,300 probability of failure, according to CSA S6.1-2014) for the reliability index for those components that will not fail suddenly or will retain post-failure capacity. Section 14 of CSA S6-2014 (CSA 2014a) is for the evaluation of structures and offers some further insight concerning target reliability levels. It states that reliability should depending on element and system behaviour, as well as inspection level. Components in a single load path structure, which fail in a sudden manner and are uninspectable, should have a reliability index of $\beta = 4.0$ ($P_f = 1$ in 31,500) (Figure C14.1 in CSA S6.1-2014). However, as the consequence of the failure of a component becomes less severe, or the degree of inspectability is higher, a reduced reliability level is permitted, to as low as $\beta = 2.5$ ($P_f = 1$ in 160). Similarly, the corresponding Eurocode recommends that the target reliability index for fatigue should fall between $\beta = 1.12$ and 3.73 depending on the inspectability, repairability, and damage tolerance, calculated for a 75-year bridge design life (CEN 2002).

The history of how fatigue critical components came to be designed for two standard deviations removed from the mean, or 97.7% survival probability, is unclear. The 2.3% probability of failure points to a reliability index less than 2.5, without consideration of load as a random variable (which would further reduce reliability). There has likely been some influence of performance-based design in the historical development of provisions for fatigue design. In a 1985 report entitled "Calibration of Bridge Fatigue Design Model", Nyman and Moses point out that their recommended reliability levels were derived from the average performance of bridges in the USA and Ontario, with the presupposition that the average performance prior to that time was deemed acceptable and economical (1985).

Nyman and Moses' comprehensive guide for the consideration of the reliability of fatigue-sensitive details incorporated uncertainties in vehicular loading, analysis methods, and fatigue life, in order to make recommendations with the goal of having more uniform safety levels and fatigue lives for steel bridges (1985). Key variables included in their uncertainty model were the accuracy of the fatigue damage model (Miner's rule was chosen and assigned a coefficient of variation, or COV, of 15%), the stress range (mean and COV values were obtained for design categories of redundant and nonredundant components, as they were separated at the time), the girder distribution factor (COV of 13%), the impact factor (commonly considered as the "Dynamic Load Allowance", assigned a COV of 11%), and the truck volume ("Average Daily Truck Traffic", or ADTT, assigned a COV = 10%). Other variables were also considered in their model, although statistical data was not available for all variables. Target safety index values were back-calculated by finding the safety index values from bridges deemed safe and economical at that time. Nyman and Moses found targets of $\beta = 3.90$ for redundant spans, and 5.28 for nonredundant spans, although they acknowledged that further study could show these values as being too high (1985).

In a subsequent NCHRP report, Moses, Schilling, and Raju made recommendations for evaluation procedures (1987). They reported that Miner's rule should be taken as accurate and justifiable as a means to convert a variable amplitude stress range into an equivalent constant amplitude range for design and evaluation, and they noted that some methods used an effective fatigue truck to represent typical traffic (something that is commonly done now). It was suggested that lower target reliability levels of 1.0 to 3.0 should be satisfactory for the fatigue evaluation of redundant and nonredundant members, respectively, since failure is less critical in fatigue than in ultimate loading considerations (Moses, Schilling, & Raju 1987). In referencing the use of a variable amplitude fatigue limit, they suggested conservatively ignoring the limit in reliability analysis (probabilistically, a guarantee cannot be made that any load value will not be surpassed), and they suggested that a slope of $m = 3$ should be used for *S-N* curves rather than the dual slope model suggested by the European Convention for Constructional Steelwork (with $m =$ 3 and 5). By this time, it was generally accepted that a practical CAFL exists, but an NCHRP study on variable amplitude loading found that "if any of the stress cycles in a stress spectrum exceed the CAFL, the fatigue life could be predicted by Miner's rule considering all cycles contributed to the damage" (Moses, Schilling, & Raju 1987). This is something that is still believed to hold true today, as discussed in a review by Baptista, Reis, and Nussbaumer (2017).

It is necessary to know how many cycles a fatigue component will be subjected to when performing a reliability analysis. The ADTT was taken as 2,500 in most studies in the 1980s (Nyman & Moses 1985); (Moses, Schilling, & Raju 1987), and it was estimated that some bridges may see well over 100 million stress cycles, but that most bridges see well below this number (closer to about two million). CSA S6-2014 (CSA 2014a) specifies that new highway bridges be designed to Class A standards, which see an ADTT of 4,000, translating to about 100 million cycles over a 75-year design life. However, the CSA S6-2014 Commentary presents a disclaimer that "forecasts for specific sites may show that actual truck traffic is considerably less than the design value" (CSA 2014b). For most cases, the passage of a truck may be taken as a single stress range cycle for a fatigue component such as a shear stud. However, short, continuous, and cantilevered spans contain regions where this rule does not hold true (as the individual axles cause significant fatigue cycles).

<u>Fatigue Performance of Welded Shear Studs</u>

The fatigue performance of welded shear studs has been the focus of many investigations in the past. From the first testing in 1966, push tests were used to approximate the shear interface between the steel and concrete in a composite member (Slutter & Fisher 1966). These tests were thought to represent a conservative estimate because of some early testing, and speculation of the role of friction and its absence in the push tests compared to beam tests. Figure 2 shows a push test schematic (lower left), and results from several studies (Sjaarda, West, & Walbridge, 2018). Originally, the Canadian and American codes (CSA 2014a); (AASHTO 2017), both adopted a log-linear fatigue curve recommended by Slutter and Fisher in their original, seminal work on the fatigue of stud connectors (Slutter & Fisher 1966).

The American AASHTO curve remains unchanged at the present time, however the CSA S6-2014 curve was modified to follow the ‘Category D’ log-log curve (AASHTO 2017); (CSA 2014a). Both of these are shown in Figure 2, where it can be noted that the CSA S6-2014 curve passes through the mean of the push test data, and is not shifted to give a 97.7% survival probability. The reasoning given for this in the CSA S6-2014 Commentary (CSA 2014b) is the aforementioned conservatism of push tests. However, there is no basis for assuming that the level of conservatism in push tests corresponds to the 97.7% survival probability. In fact, the assumption of conservatism seems dubious when comparing the small database of beam test results to the design criteria and associated push tests (see Figure 2).

Figure 2. Fatigue results from push and beam tests, showing CSA S6-2014 Category D.

Figure 3. University of Waterloo fatigue results from beam tests (Sjaarda et al. 2017).

A beam testing program at UW was conducted in 2016 to investigate shear connector failures and their consequences, and to support the current reliability analysis. Twelve beam tests were conducted with welded shear studs, and 63 stud failures were observed (Sjaarda et al. 2017).

These results are shown graphically in Figure 3. Since multiple stud failures were observed on each beam, stresses have been adjusted to account for stud force redistribution throughout the tests. Readers may refer to (Sjaarda et al. 2017) for an account of this procedure, as well as a detailed discussion of failure detection and failure definition. The design curves shown (bottom) were calculated using the procedure recommended by the International Institute of Welding (IIW), to give the 97.7% survival probability associated with a two-sided 75% confidence level of the mean. The CSA S6-2014 (CSA 2014a) design curve, with a slope of $m = 3$ and an endurance limit of 24 MPa, is shown as a dashed line in Figure 3, coinciding partially with the CSA S6-2019 (CSA 2019) design curve, which features a dual slope finite life region along with a new endurance limit of 35 MPa. In the lower plots of Figure 3, the mean (μ) and design (μ-2σ) curves are shown for the UW beam tests, with a slope of $m = 3$ (right) and 5 (left). Note that the mean beam test curve lies above the CSA S6-2014 (CSA 2014a) curve, but the design curve falls below. Since the CSA S6-2014 (CSA 2014a) curve has been constructed using the 50% survival probability of existing push test results (μ_{push}), the UW beam test results reveal that push tests are indeed conservative relative to beam tests ($\mu_{beam} > \mu_{push}$), but not as conservative as historically assumed ($\mu\text{-}2\sigma_{beam} < \mu_{push}$).

The choice of slope for the stud *S-N* curve is important and is difficult to infer without long life tests (the UW beam tests did not appreciably add to the dearth of long life tests). Studs are most often designed in the high cycle range (50 to 100 million cycles), where the curve is known to flatten out, but tests are most often conducted to a lower numbers of cycles (0 to 5 million cycles). The practical difference between the slopes is that a slope of $m = 5$ is favourable for high cycle fatigue (leading to less studs necessary in design) and would be unfavourable to low cycle fatigue (which does not often occur on bridges). Although $m = 3$ could be called conservative assumption, it would likely be overly conservative; the Eurocode (CEN 2004) uses a slope of $m = 8$ for the entirety of the design curve, and a comprehensive review of tests by Johnson has indicated a slope of $m = 5.5$ (2000). A log-log analysis of Slutter and Fisher's original experiments gives a slope of $m = 5.26$ (1966). It seems that $m = 3$ has been used in North America to this point simply to match other steel fatigue-sensitive details.

Example Bridge Description and Design

The bridge used to assess the reliability of shear stud design procedures for the current study consists of a concrete deck on steel girders. This bridge type is very common in North American, particularly for medium-spans. The bridge is simply-supported with a span of 30 metres and has a 225 mm thick 30 MPa concrete deck atop 6 steel girders, which have a spacing of 2.5 metres on centre. There are four design lanes on the bridge supporting a Class A highway in Canada. The "code truck" used for the analysis was the CL-625-ONT Truck (see Clause A3.4.1 of CSA S6-2014). The geometry of an interior girder is shown in Figure 4, and features a constant cross-section along the bridge length for simplicity. For the shear connection, 22.2 mm diameter (7/8 inch) headed studs were used in rows of two.

All new highway bridges in Canada are required to be designed as Class A unless otherwise approved (CSA 2014a). These bridges have an average daily truck traffic (ADTT) of 4000 vehicles over a design life of 75 years, resulting in about 100 million trucks ($N_c \approx 100 \cdot 10^6$). For simply-supported bridges, each truck corresponds to one cycle of loading on each stud on the bridge. Each stud is designed as if it were located on a girder underneath the truck lane, but if multiple lanes are available for trucks, a reduction factor is applied. For the CSA S6-2014 (CSA 2014a) design curve, a stud experiencing over 52 million cycles is designed at the endurance limit. It follows that the importance of the stud fatigue design curve is concentrated in the endurance limit value, where most studs are actually designed.

The use of the 0.52 fatigue code truck factor is based on fatigue damage equivalence calculations done on the Canadian code truck model with a slope of $m = 3$. This factor is no longer valid when using a slope of $m = 5$, or a dual slope *S-N* curve, and should be adjusted to be slightly higher (Coughlin & Walbridge 2011). However, this issue is not addressed in this analysis for the sake of simplicity.

Figure 4. Interior girder cross-section.

Table 1: Reference table for example bridge design values

Ultimate Limit State (ULS)

The number of shear studs required at ULS can be determined considering the amount of horizontal shear that must be transferred into the slab of a girder to develop the full flexural capacity of the cross-section (considering steel to be "elastic perfectly-plastic" and using the equivalent stress block method for concrete). In Table 1 this value is given as 10,978 kN. The static strength of an individual stud, for the 22.2 mm stud diameter given, is calculated in Equation 1. It is taken as the lesser of the maximum force limited by the stud steel ultimate capacity, and the crushing of the deck concrete:

Equation 1
$$q_r = 0.5\phi_{sc} A_{sc} \sqrt{f'_c E_c} \leq \phi_{sc} F_u A_{sc} = 136.5 \text{ MPa}$$

In Equation 2, the left side of the inequality represents concrete crushing and the right side represents the steel strength of the stud. The total number of studs required on the girder (two shear spans) is:

Equation 2
$$\#Studs\ (ULS) = \#ShearSpans\left[{C_c}/{q_r}\right] = 2\left[{10{,}978\text{ kN}}/{136.5\text{ kN}}\right] = 154\text{ studs}$$

This is in accordance with Cl. 10.11.8 of CSA S6-2014 (CSA 2014a), which also states that shear connectors may be spaced uniformly, or according to the variation in the interface shear. The assumption is made that each connector has sufficient ductility to reach maximum strength at the same time, regardless of spacing. Although the total horizontal shear force necessary to develop the capacity of the cross section is required by code to be able to be transferred, a partial shear connection is sufficient to carry factored loads in most cases. In CSA S16-2014 "Design of steel structures", the minimum partial shear connection permitted is 40% (CSA 2014c). Figure 5 illustrates the loss of ultimate strength capacity with partial shear connection levels ranging from a full connection (154 studs) to the non-composite case (no studs). The sharp decline below a 10% connection is due to the slenderness of the Class 3 web in the example bridge plate girder. Note that studs provided in excess of 154 cannot theoretically contribute to the strength of the composite girder; they may be required never-the-less for fatigue considerations, and their number indicates the reserve between the FLS design requirements and the ULS requirements.

Figure 5 also shows M_f, the factored design moment at ULS. The ratio of M_r with a 100% shear connection to M_f is of interest when considering the reliability of the shear connection in fatigue. The closer the ratio to a value of 1.0, the less tolerance the structure has for the degradation of the shear connection past $M_{r(100\%)}$. However, since the curve in Figure 5 is relatively flat on the right-hand side, even a ratio of 1.08, which is present in the current example, is enough to allow a shear connection degradation to the 50% level (approximately 78 studs), with the structure still having sufficient ULS capacity, despite violating the code requirement of 100% shear connection. Fatigue design limits govern shear stud design for any bridge with a meaningful number of cycles expected, and the number or studs required will exceed the 100% shear connection amount often by more than a factor of two, allowing significant degradation before ULS concerns arise.

Figure 5. Ultimate capacity of the bridge girder with varying shear connection.

Fatigue Limit State (FLS)

The procedure to calculate number of studs required at the FLS under the CSA S6-2014 (CSA 2014a) code provisions involves ensuring that the calculated stud stress range, τ_{rs}, is below the allowable stress range. While the allowable stress range is a constant value, the calculated stud stress range varies with the design longitudinal shear force range (V_{sr}) and the stud spacing as given in Equation 3.

Equation 3 $$\tau_{rs} = \frac{V_{sr}Q}{A_{sc}I_t}\left[\frac{s}{n}\right]$$

In Equation 3, n is the number of stud connectors per row on the girder top flange and s is the row spacing. Other variables in Equation 3 are given in Table 1. The design shear force can be determined for each stud location from the moving load envelope of shear from the passing of the design truck, including the dynamic load factor (*DLF*). Note that the design longitudinal shear force for fatigue is the fatigue code truck factor (Table 1) multiplied by the total range of shear force per girder as the design truck passes over the bridge, V_g. The latter can be computed based on the shear value per design lane, V_T, using the transverse amplification factor (girder spacing, S, of 2.5 m, divided by the truck load distribution width, F, of 3.7 m).

Equation 4 $$V_{sr} = 0.52V_g = 0.52\left[\frac{S}{F}\right]V_T = 0.351V_T$$

The design shear force range (V_{sr}) is directly proportional to the stud stress range provided the bridge cross section and stud row spacing remain constant. Multiple stud row spacings are normally chosen to obtain material and labour savings where the design shear forces are lower (spacing may follow variations in interface shear, see Cl. 10.11.8 of CSA S6-2014). In Figure 6, an economical spacing plan for the example bridge is shown with rows of 2 studs spaced at 125 mm for approximately 10,000 mm on either end of the bridge, and 187.5 mm for the middle portion, for a total of 426 studs. The maximum allowable stress range of 24 MPa, the elastically calculated stress range commonly used in design (from Equation 3), and the actual stud stress values from the FE model used in this study are also shown in Figure 6.

Figure 6. Allowable and design stud shear stresses.

It can be seen that the actual stress ranges from the FE model used in this study are 15-30% lower than those expected by calculation. The reason for this is that the elastic stress calculation assumes 100% composite interaction. Studs, like any other mechanical shear connector, require deformation to resist shear force; this very fact results in less interfacial shear force transferred due to compatibility (Sjaarda, West, & Walbridge 2018). However, although this may seem to indicate that fatigue provisions are conservative due to the difference in actual stress and elastically calculated stress, this is not the case since the *S-N* results and design curves are already using the elastically calculated stress, and not the lower, real stud stress (see Figure 2 and

Figure 3). This statement doesn't hold true for push test *S-N* results or design curves derived from them, however, since push tests do not feature the same compatibility requirements.

The allowable stress range is 24 MPa, which is the product of the endurance limit for a 'Category D' detail, and 0.52. The 0.52 "fatigue code truck factor" in Table 1, represents the proportional weight of an average truck compared to the 625-tonne code truck. Any bridge designed in Canada that will experience more than 52 million cycles over its design life requires studs to be designed at this 24 MPa endurance limit to ensure that the heaviest trucks do not exceed the limit. The example bridge is expected to experience 87.6 million cycles as a Class A bridge; if Class B were used, the number of design cycles would change to 21.9 million and the allowable stress would increase 33% to 32.1 MPa.

The two-spacing plan shown in Figure 6 of 125 mm and 187.5 mm uses 2 connectors per row (n = 2) and results in a total number of 426 shear connectors for the example bridge. This is almost three times as many connectors as required for code compliance of a 100% shear connection (154), and more than five times than that required for strength purposes (78). It is essential that a reliability analysis be conducted, to discover if the great number of studs required under the CSA S6-2014 (CSA 2014a) design rules are all necessary. The first objective of the investigation is therefore to estimate the reliability of shear connections designed under the CSA S6-2014 (CSA 2014a) rules, and the secondary objective will be to propose modified design rules to bring that level of reliability into better agreement with code targets.

Shear Connection Reliability Definitions

In order to perform a fatigue reliability analysis, the fatigue resistance variable R can be taken as the total number of load cycles a particular shear connection can withstand prior to failure, and the fatigue demand variable Q is the total number of load cycles applied to the connection for the lifetime of the bridge. The safety margin, g, represents the number of cycles in reserve for a given shear connection ($g = R – Q$).

Shear connection failure happens when the shear connection ceases to perform its function of allowing adequate composite action through the transfer of longitudinal shear forces. Shear connections are comprised of many studs; therefore, the connection may still perform adequately after the fatigue failure of one or more studs. It has just been shown that 64% of the studs in the example bridge may fail before ULS design rules are violated, and another 18% many fail before the strength is insufficient to carry factored loads. The total resistance of the connection, R, is therefore the sum of the number of cycles required to fail the first connector, and the number of cycles required to fail each subsequent connector until safety becomes a concern. For simplicity, failure in this study is defined as the point at which a 100% shear connection remains, or in other words, the point at which the reserve between the FLS design requirements and the ULS design requirements is removed due to stud failures.

The number of cycles a stud shear can resist, R, is affected by random variables (RVs), including:

- the fatigue resistance of each stud (stud fatigue curve intercept, or vertical shift),
- the weight of the trucks crossing the bridge (fatigue code truck weight),
- the dynamic load factor (taking into account the dynamic load allowance), and
- the girder distribution factor (or transverse amplification factor).

Normally, only the first variable in the list would be classified as a resistance variable, since the latter three have more to do with the load effect. However, for the formulation of this fatigue reliability problem, it is convenient to group these together into the random variable R.

The Monte Carlo Simulation (MCS) method is necessary to determine the resistance, R, due to the great number of random variables involved in the stud shear connection problem and the geometrical complexity encountered in solving for connector stresses during the fatigue life of a bridge. Although it may appear that there are only four RVs in the current analysis, a closer inspection reveals that *each stud on the bridge* represents a random variable on its own (stud strengths are independent and identically distributed variables, or IIDs), because each stud on the bridge is in a unique location.

The fatigue endurance limit concept presents a challenge when conducting failure simulations. The CSA S6-2014 (CSA 2014a) fatigue provisions state that any shear stud experiencing a stress range of under 24 MPa with the passage of a fatigue code truck sustains no damage and will not fail regardless of the number of loading cycles. As discussed previously, almost all highway bridges designed in Canada, including the bridge under discussion, are required to have studs designed at this endurance limit due to the high number of loading cycles expected. In the current study, the endurance limit is conservatively ignored, and it is assumed that the *S-N* curve maintains a slope of $m = 5$ past the endurance limit. The degree of overdesign identified with this analysis is, however, ultimately used to recommend a vertical shift in the endurance limit, on the basis that the vertical position (i.e. stress range) at which the endurance limit is set is directly relatable to the vertical position of the sloping portion of the S-N curve in the finite life domain. The UW beam tests are used to define the *S-N* curve, so that the mean (μ) from Figure 3 is adopted directly.

The fatigue demand variable, Q, is the total number of cycles (for most bridges, this is the total number of trucks that will cross the bridge) in its service life (N_c in CSA S6-2014). This is a random variable, but due to limited knowledge, the deterministic code design value will be used for the purposes of this investigation. This value only depends on the number of design lanes on the bridge, the design life of the bridge, and the expected average daily truck traffic (ADTT), as well as the assumed number of cycles per truck passage, N_d. Equation 5 shows that Q is equal to the product of N_d, p (a scaling factor equal to or less than 1 accounting for multiple design lanes), y (the design life of the bridge in years), and ADTT. The value of Q is 87.6 million cycles for a typical simple span composite bridge ($N_d = 1$) with three design lanes ($p = 0.80$), a typical service life of $y = 75$ years, and a design ADTT of 4,000.

Equation 5 $$Q = N_c = N_d p\left(ADTT\right)365y$$

Since Q is assumed to be deterministic, the distribution of g takes the same form as R (g is simply shifted by Q). For the stud shear connection this distribution will be shown to be lognormal.

FE Model Description

SAP2000 Model Description

A SAP2000 model was programmed using the CSi Application Program Interface (API). Figure 7 depicts the model and a typical girder the model approximates, with the CL-625-ONT Truck Loading (CSA 2014a) . The concrete deck is represented by shell elements, and beam elements are used for the steel beam. The two are connected using linear link elements representing shear connectors. The stiffness of the link elements is a key input to the model, because it determines how much force the studs will attract, and affects the resulting degree of composite interaction. For this model, an equation developed by Ollgaard, Slutter, and Fisher (1971) was used to estimate stud stiffness, and it was confirmed through analysis that the studs remained in the linear range of load-slip behaviour (consistent with expectations for the fatigue load level). The model contains simplifications, including elastic material behaviour for the steel and concrete, which allow it to be solved quickly. As a result, validation of the model was necessary to ensure the results were not impacted by these assumptions, as detailed in (Sjaarda et al. 2017). The effects of friction, which would serve to reduce the shear stress on connectors, are not included for conservatism (and since friction is implicitly included in *S-N* results from beam tests, as the flanges were not greased).

Figure 7. FE beam model showing element types.

Results from the FE model are shown in Figure 6, where a moving load analysis was used to generate shear stress envelopes for the studs. For all moving load simulations, the SAP2000 influence-based moving load analysis function was used to establish an envelope of connector stresses. This is necessary because the design of shear connectors depends not on the highest stress produced from the passage of the fatigue code truck, but on the stress range.

Monte Carlo Simulation Procedure

The FE model was used to determine the failure order and stress history of each stud on the bridge as stud fatigue failures occur. This was done for the deterministic case, and it was done probabilistically through Monte Carlo Simulation (MCS), where each variable was assigned a random variate for every failure trial, and many trials were completed in order to estimate the distribution of the safety margin, g, and to fine the overall reliability of the connection, β. As discussed earlier, the passage of a truck over the example bridge causes a single load cycle for a shear connector. During the passage of a truck on a typical simple span composite bridge, the peak and minimum loads are of opposite sign. The magnitude of load during the load cycle is the algebraic difference between the maximum and minimum.

The random variables used in the model and their associated statistical parameters are given in Table 2. For the MCS procedure, a new bridge geometry is given at the start of every trial consisting of studs with random strengths according to the statistical strength distribution observed during the beam testing at UW (Sjaarda et al. 2017). A moving load analysis is performed for a fatigue code truck with a random weight (F_C), fatigue dynamic load factor (DLF), and distribution factor (F_v). The number of cycles until failure of the most vulnerable stud (stud with the lowest number of cycles until failure based on the stud stress range and fatigue strength) is calculated, and then that stud connector is reprogrammed to simulate failure. Damage is calculated for each of the other studs for use in Miner's sum calculations, which are repeated after every analysis for each stud that has not failed. A new analysis then begins with the modified bridge geometry (failed stud), and subsequent stud failures are determined based on cumulative damage calculations. Each trial continues in this way until full shear connection failure.

The stud fatigue curve intercept and COV of 16.76 and 4.7%, respectively, were taken from the UW beam tests (shown in Figure 3). These tests included 63 stud failures and are believed to be a good representation of stud behaviour in beams and girders (although the database is still lacking in very long life results). The fatigue code truck weight is the CL-625-ONT truck scaled by a factor of 0.52 to reflect the average truck causing fatigue damage, and a COV of 5.3% is taken for this variable from the CSA S6-2014 Commentary (CSA 2014b). The fatigue dynamic load factor and COV (1.11 and 11%, respectively) are taken from a fatigue design calibration by Nyman and Moses (1985) and applied to each bridge, not to each truck. This factor has a minimum value of 1.0. Even though a factor of 1.11 for dynamic load consideration is not high, one of the primary variables, pavement quality, is likely to be repaired during the design life of a bridge, mitigating the effect of extreme values of this variable. The girder distribution factor is taken from CSA S6-2014 (CSA 2014a) and has a bias factor of 0.93 applied, as well as a COV of 12%.

Table 2: Probabilistic simulation variables.

A single trial for the simulation procedure includes the following steps:

1. All studs on the example bridge are assigned random strengths (based on experimental data), and the bridge is assigned a dynamic load allowance factor.
2. An effective truck load is sampled and applied to the bridge in a moving load analysis.
3. Stud stress ranges are recorded, and the number of cycles until failure is calculated for each stud, N_i.
4. The lowest calculated N_i value is saved as N_f, and damage is calculated for all studs using this N_f. As a result, the failed stud has a damage of 1.0, and all other studs sustain damage of N_i/N_f.

5. The failed stud is reprogrammed as failed, and Steps 2 through 5 are repeated until the shear connection has failed.

Runtime and Sample Size

Since each trial consists of many stud failures, with every failure requiring an individual moving load analysis, a significant amount of computing time is needed to complete a statistically significant number of trials. However, it was observed that the failure order of the studs depends only on the random strengths of the studs, and not on the random properties of the load (truck weight, dynamic load factor, and girder distribution factor), because the load only serves to scale all stud stresses up or down in the same proportion for a given moving load analysis. As a result, a strategy was employed where a limited number of SAP2000 moving load trials were completed using probabilistic stud strengths but deterministic loads. Afterwards, the stud failure orders and stud stresses from those analyses were used to repeat the analysis in MATLAB with randomized load effect variables. Since the moving load analyses were already completed, a large number of trials could be simulated in MATLAB. In this way, the reliability analysis of the bridge was completed by sampling a small number of random bridges in terms of stud strengths along the span (100 for example), but then subjecting each random bridge to a large number of random loading trials (200 each, for example, for a total of 20,000 trials).

Results

The distribution of the shear connection fatigue life was found to be lognormal, as shown by the square of the Pearson correlation coefficient (99.7%) in the probability paper plot in Figure 8. The reliability of the as-designed shear connection was found to be $\beta = 5.13$ (95% confidence interval ± 0.07) for 100 random bridge geometries subjected to 20,000 complete failure simulations (200 for each bridge). This value goes up to 5.51 when considering only the scatter on the left-hand side of zero in the probability paper plot, to obtain ζ. These results indicate that the CSA S6-2014 (CSA 2014a) design procedure for welded studs is conservative when considering the shear connection as a whole, taking advantage of the high level of redundancy.

Figure 8. Probability paper plot for the shear connection as designed.

Since the load and load effect on the simply-supported example bridge are linearly related, the analysis can be extended by introducing a load multiplier, *LM*, which simulates a relaxed spacing of the studs (or a higher permissible endurance limit). In Figure 9, reliability is shown to decrease with increasing values of *LM* between the CSA S6-2014 (CSA 2014a) design allowable stress ($LM = 1$, stud allowable stress of 24 MPa), and twice this limit. It is shown that a 45% increase in the allowable stress (an endurance limit increase from 24 MPa to 35 MPa) will lower the lifetime reliability of the system to between 3.25 and 3.50. This is close to the CSA S6-2014 lifetime target value of $\beta = 3.5$ (CSA 2014a), and at the higher end of the Eurocode recommended range (CEN 2004). This reliability value allows for no inspections for the lifetime of the bridge, which will become important as bridge decks are made to last longer over time. The relationship between β and *LM* is roughly linear. Since failures occur on the "left side" of the median simulation (zero on the probability paper plot in Figure 8), the β to *LM* curve considering only the left side scatter is more relevant. This curve is not negatively affected by simulations producing extremely long fatigue lives, and is taken as the "base analysis" going forward into the sensitivity investigation.

Figure 9. Reliability (β) as a function of the Load Multiplier (*LM*).

Applicability, Conservatism and Sensitivity

The applicability of the presented results may be limited by the use of the example bridge, which may not represent all bridges with stud shear connections. In particular, the length of this bridge was 30 metres, which allowed one cycle to be taken per truck crossing. For shorter bridges, studs will experience more than one cycle per truck crossing; CSA S6-2014 (CSA 2014a) sets this length as 12 metres or less. Steel composite bridges of 12 metres or less are rare, so although this is an important issue, it only affects a small number of bridges. The same issue occurs for regions near interior supports of continuous girders, but shear connectors are not required in these areas. Thus, it is believed that the example bridge is largely representative of steel-concrete composite bridges, and results are applicable to a wide range of cases.

The conservatism in the presented results lies in the decisions to: 1) count all cycles as producing fatigue damage (ignoring the endurance limit altogether), 2) give no consideration to multiple girder redundancy, and 3) assume no post-failure stiffness of the studs. In this analysis, studs were assumed to fail immediately; this was not observed in the UW beam test program. Rather, studs were found to retain significant long-term stiffness, with an average long-term stiffness loss of only 21% after fatigue failure, as can be seen in Figure 10 (Sjaarda et al. 2017). Computed reliability levels considering this phenomenon are vastly higher, as will be shown in the sensitivity investigation. The post-failure stiffness (PFS) observing in testing probably arose primarily due to mechanical interlock between the stud and the flange after failure, owing to the shape of the failure plane, which dug into the flange. For different flange thicknesses, the failure pattern may differ, so it may be prudent to ignore any potential gains from stud PFS in establishing design rules.

Figure 10. PFS of studs in beam test program (Sjaarda, West, and Walbridge 2018).

The main analysis result presented in Figure 9 may be sensitive to a number of factors, including each of the input variables shown in Table 2 (both mean and COV values), the slope of the *S-N* curve used in the analysis, and the expected traffic volume over the life of the bridge, Q. In Figure 11 (top), reliability is shown as a function of *LM* for the base analysis (m = 5), as well as for the case of an *S-N* curve with m = 3, both with 50% stud post-failure stiffness and without. It is shown that the result is very sensitive to the slope of the *S-N* curve, as well as to the including of PFS in the model. Lower *S-N* slope values, such as m = 3, permit higher stresses at low-cycle fatigue, but lead to low fatigue lives for high-cycle fatigue, particularly when ignoring the endurance limit.

Although the stud connector detail in CSA S6-2014 (CSA 2014a) was given a slope of m = 3, CSA S6-2019 (CSA 2019) specifies a slope of m = 5 for high cycle fatigue. As discussed previously, this higher slope is more in line with the Eurocode (m = 8, CEN 2004), a comprehensive study by Johnson (m = 5.5, 2000), and Slutter and Fisher's original work (m = 5.26, 1966). The evidence from the UW beam tests in inconclusive in determining a slope, due to a lack of long life test results. Figure 11 shows that considering no PFS and taking a slope of m = 3 (ignoring the endurance limit) would lead to a reliability level of β = 3.03. Applying these assumptions together would be highly conservative, and unnecessary, given what we know about stud behaviour from recent laboratory testing. Not only is some post failure stiffness a reality, but ignoring the endurance limit without assuming any change to a shallower slope in the high cycle region is quite unrealistic based on standard fatigue knowledge, and the frequent observance of runouts under low stress levels (Baptista, Reis, & Nussbaumer 2017). Thus, the authors feel that the approach of taking $m = 5$ and no PFS is reasonable and defendable. Taking a slope of $m = 5$ and considering PFS may lead to further design economy. However, prior to adopting a more drastic endurance limit increase, additional study is advisable to quantify the true PFS more precisely.

The reliability level is also sensitive to increased loads, from both a traffic and weight perspective. For example, with a 20% increase in expected traffic volume over the lifetime of the bridge, the reliability of the system decreases from 3.42 to 3.23 at the 1.45 *LM* level. This is not a large drop, and is put into perspective when compared to the result of a similar increase in mean fatigue load (truck weight). A 20% increase in the effective fatigue truck weight would lower the reliability from 3.42 to 2.40.

Figure 11. Sensitivity of stud shear connection reliability to *S-N* curve slope & stud PFS (top) and traffic volume, mean fatigue load, & standard deviation of *DLF* (bottom).

Design Recommendations and Conclusions

At present, the design procedure for shear connections is essentially aimed at preventing failure of individual studs and treats the detail like other inspectable details. This manuscript aims to quantify the reliability of shear studs by considering them as a redundant system and asserts that the stud detail is quite different from others and should be treated as such. The reliability of stud shear connections designed according to CSA S6-2014 (CSA 2014a) results in overly conservative designs with reliability indices above $\beta = 5.0$. The redundancy of the shear stud connection allows for many stud fatigue failures to occur prior to ultimate limit state concerns arising. Although the *S-N* fatigue endurance limit was not considered explicitly in the presented analysis, this analysis can be used to infer an increase in the endurance limit on the basis that the vertical positions of the S-N curve in the finite life domain and the endurance limit are proportional. On this basis, the presented analysis indicates that an increase in the CSA S6-2014 (CSA 2014a) endurance limit of approximately 1.45 times, from 24 to 35 MPa, is justifiable. It is worth noting that a similar strength increase could also be applied to the sloping (finite life) portion of the design S-N curve. Whether or not this is done has limited practical relevance, however, given that most new bridges in Canada are currently designed as Class A highway bridges. A further increase may be justified in the future after additional study of the post-failure stiffness of the studs and if more high-cycle variable amplitude (20 – 100 million cycles) *S-N* data were to become available to shed further light on the slope and scatter of the *S-N* curve in the high cycle region. Future work may also include the investigation of other bridge geometries.

Acknowledgements

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Tables:

Table 1: Reference table for example bridge design values

Variable Description	*Symbol*	*Value*	*Units*
Stud ultimate strength	F_U	415	MPa
Concrete modulus of elasticity	E_C	25,588	MPa
Compressive force in slab at midspan	$C_C + C_R$	10,978	kN
Design life cycles	N_C	$87.6 \cdot 10^6$	cycles
Fatigue code truck factor	-	0.52	-
First moment of area of the transformed slab about the elastic neutral axis of the transformed section	Q	$21.244 \cdot 10^6$	mm^3
Transformed moment of inertia	I_T	$30.340 \cdot 10^9$	mm^4
Resistance factor for concrete/steel/shear	$\Phi_c/\Phi_s/\Phi_{sc}$	0.75/0.95/0.85	-

*The plastic neutral axis was calculated to be in the steel top flange at the ultimate limit state

Table 2: Probabilistic simulation variables.

Variable Description	**Symbol**	**Mean Value**	**COV (%)**	**Source**
Stud Fatigue Curve Intercept	LOG(C)	16.76	4.7%	UW Beam Tests, $m = 5$ (Sjaarda et al., 2017)
Fatigue Code Truck Weight	F_C	0.52·[CL-625-ONT]	5.3%	(CSA 2014a) (CSA 2014b)
Fatigue Dynamic Load Factor	*DLF*	1.11	11%	Nyman & Moses (1985)
Girder Distribution Factor	F_v	0.676·0.93	12%	CSA S6.1-2014 Clause C14.11 (CSA 2014b)

Figure Captions:

Figure 1. Cast in place (CIP) welded stud connectors.

Figure 2. Fatigue results from push and beam tests, showing CSA S6-2014 Category D.

Figure 3. University of Waterloo fatigue results from beam tests (Sjaarda et al. 2017).

Figure 4. Interior girder cross-section.

Figure 5. Ultimate capacity of the bridge girder with varying shear connection.

Figure 6. Allowable and design stud shear stresses.

Figure 7. FE beam model showing element types.

Figure 8. Probability paper plot for the shear connection as designed.

Figure 9. Reliability (β) as a function of the Load Multiplier (*LM*).

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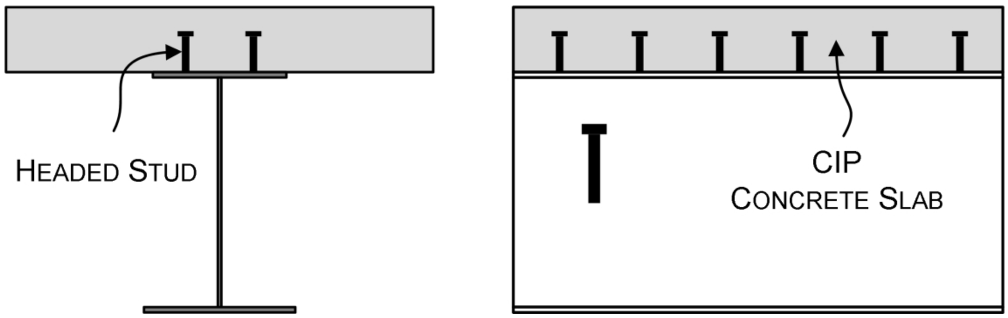

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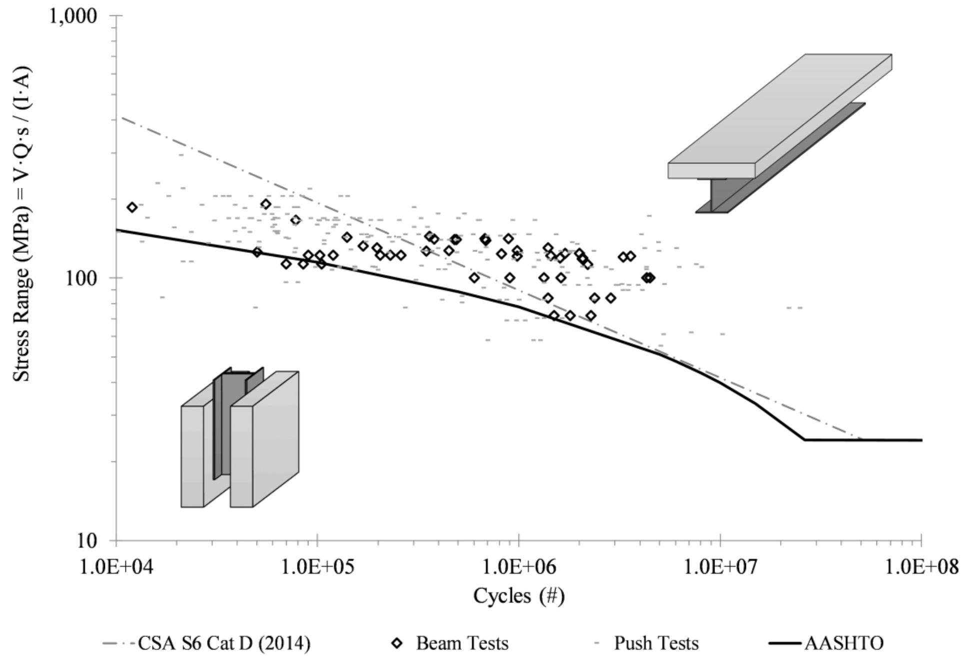

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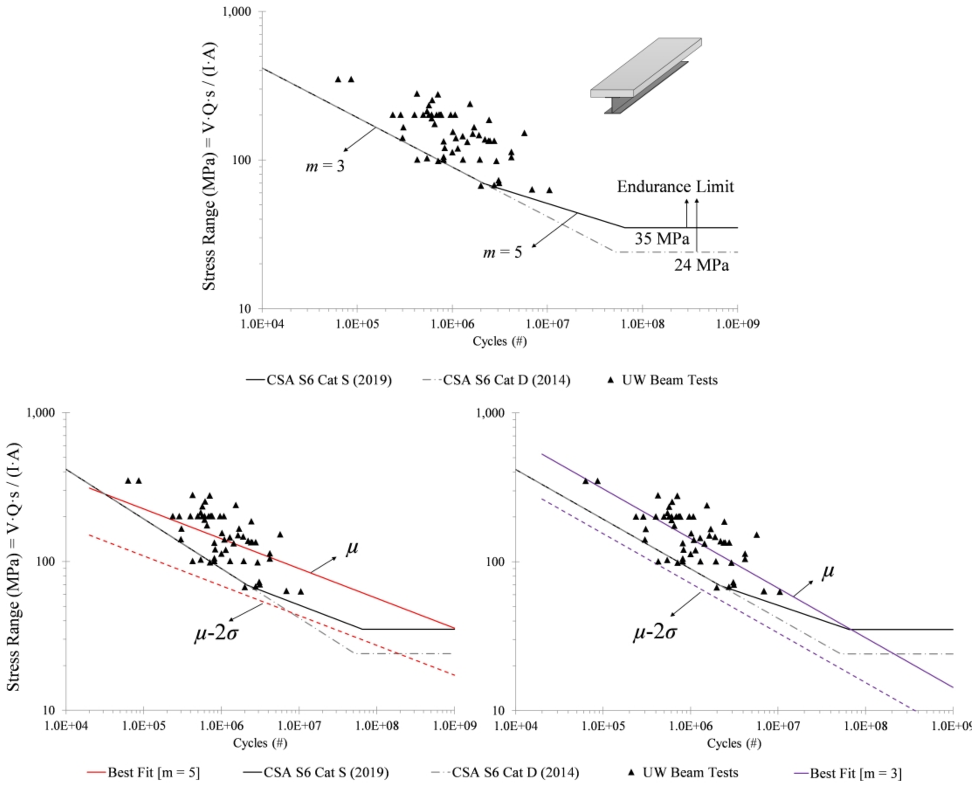

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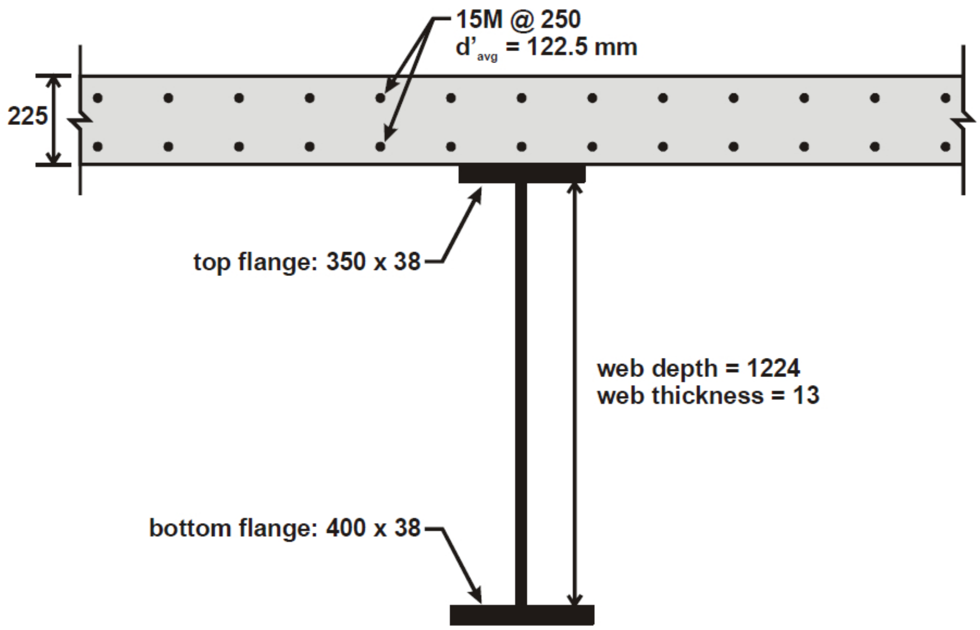

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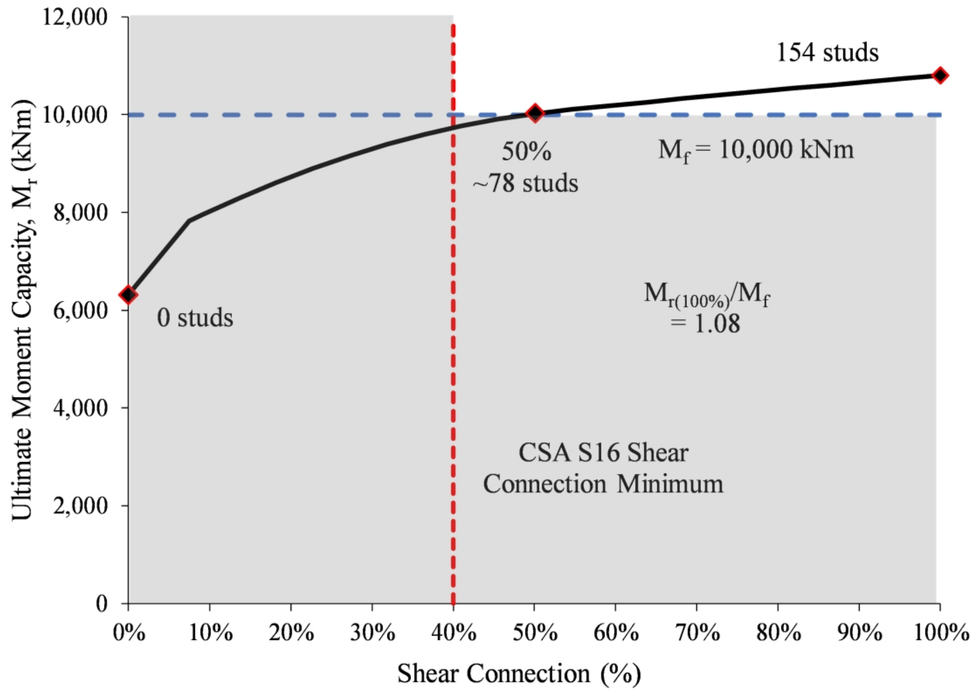

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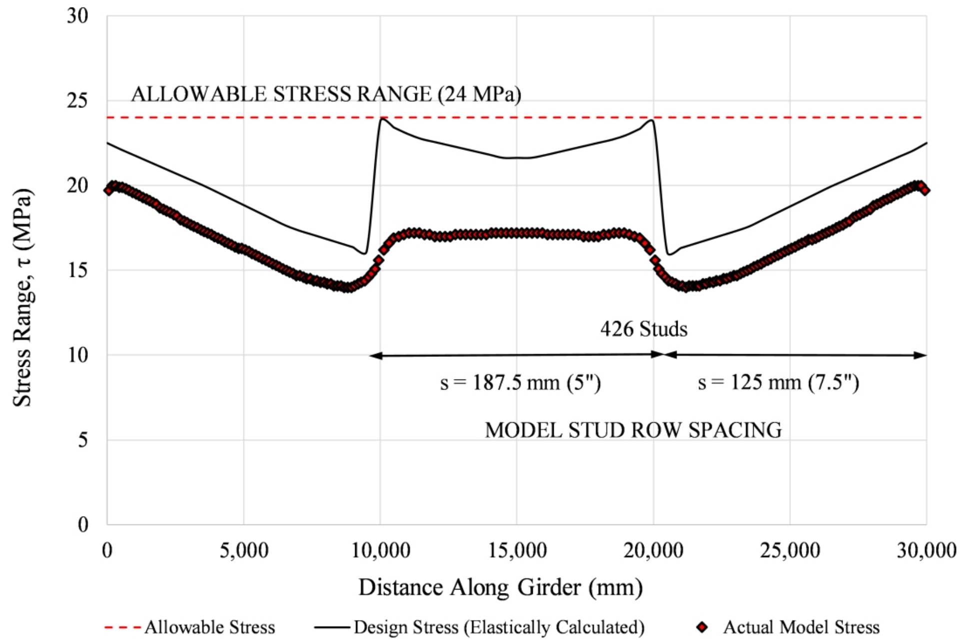

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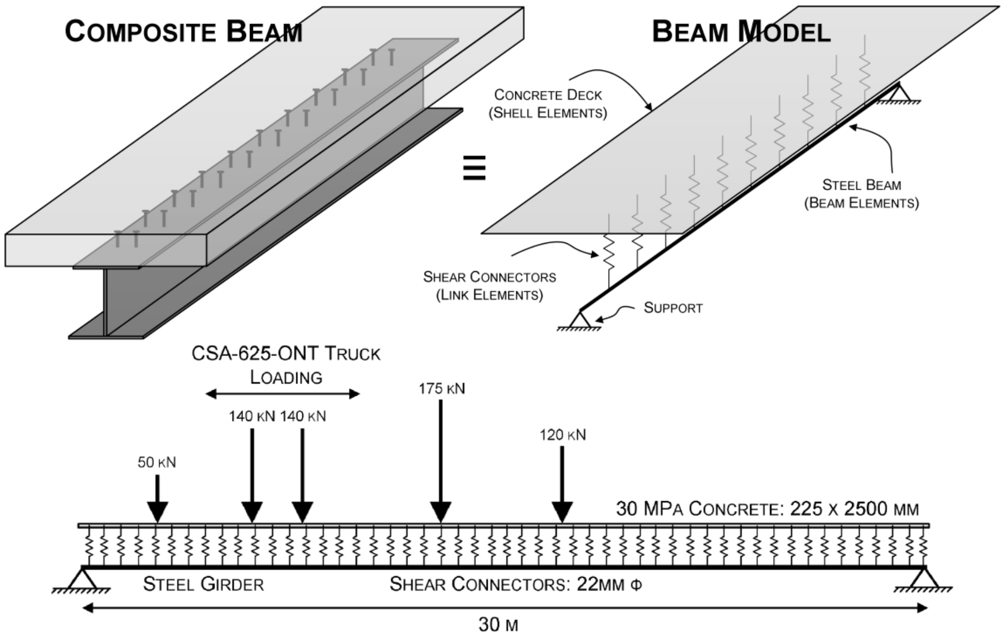

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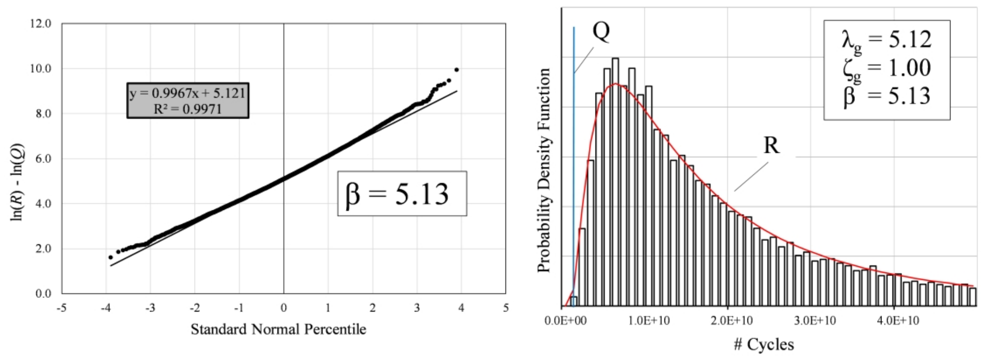

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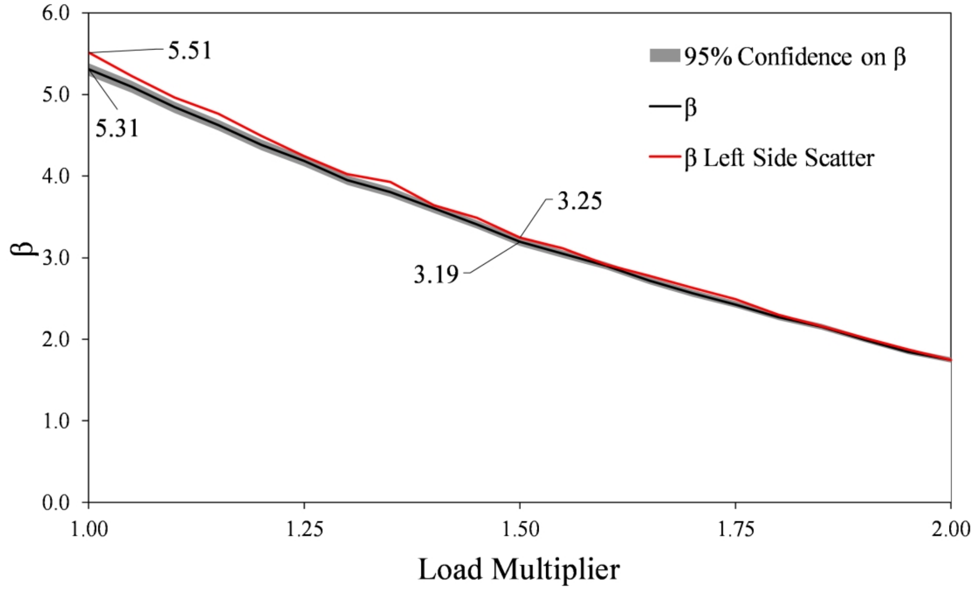

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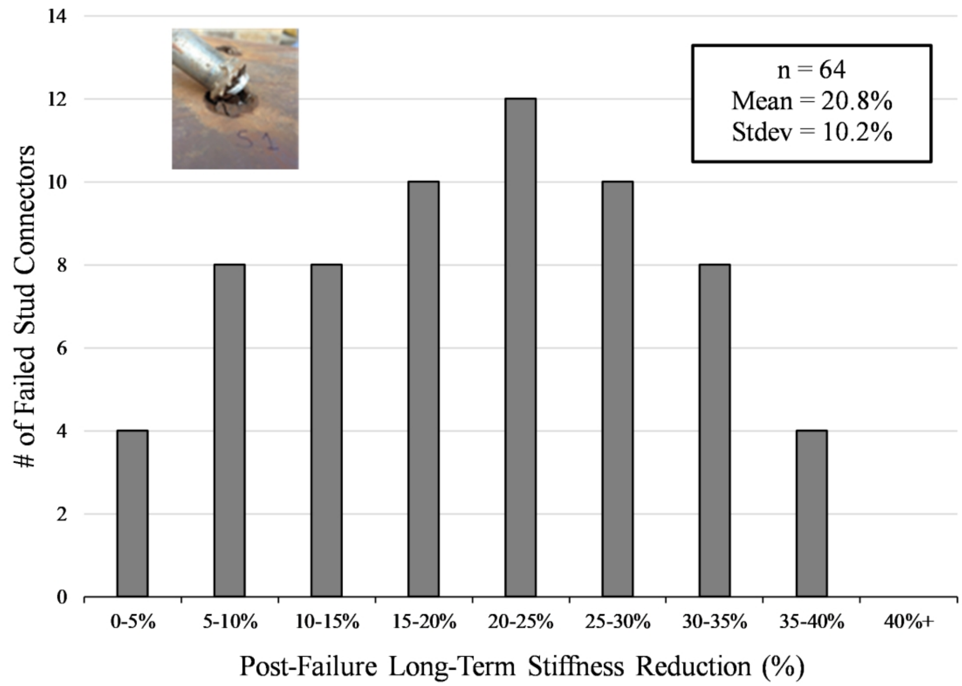

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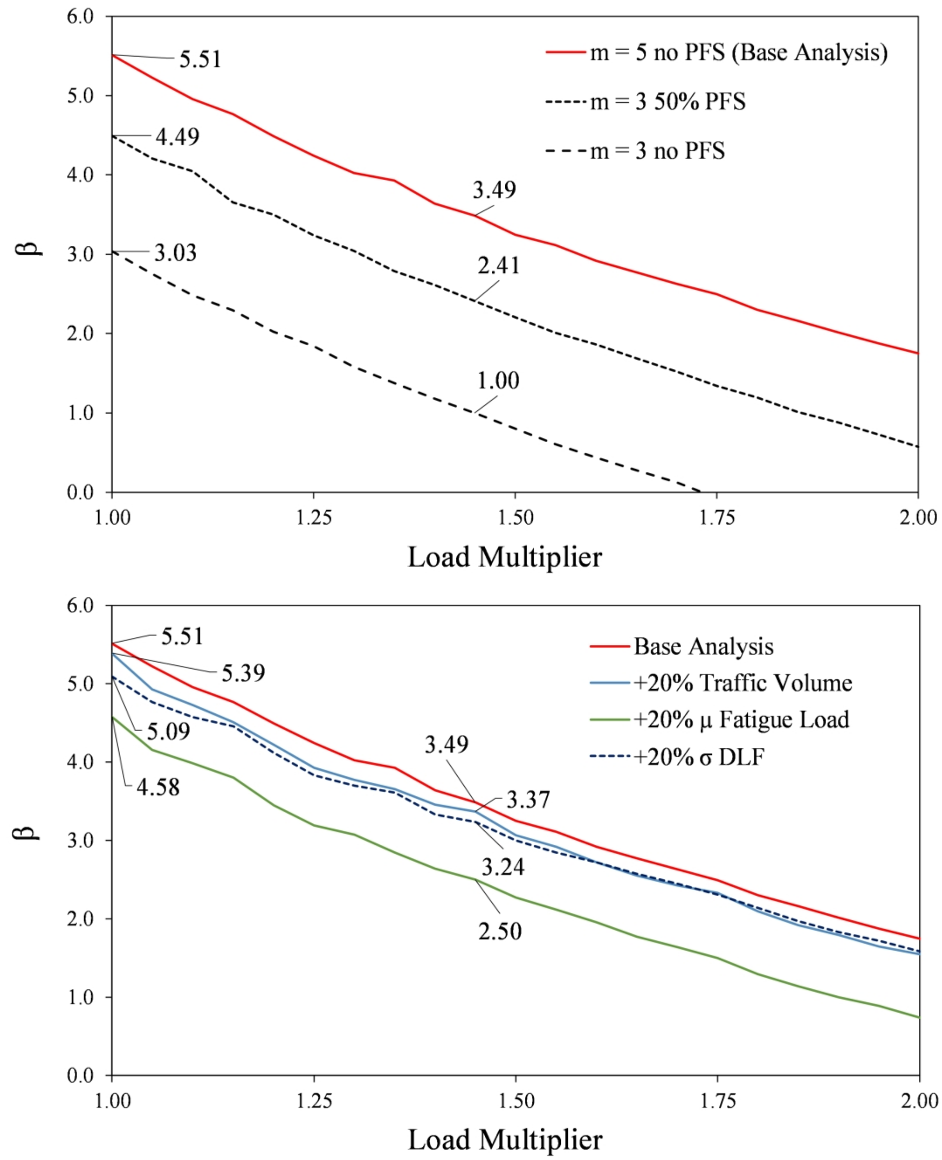

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