FATIGUE OF STUD SHEAR CONNECTORS IN THE NEGATIVE MOMENT REGION OF STEEL GIRDER BRIDGES: A SYNOPSIS OF EXPERIMENTAL RESULTS AND DESIGN RECOMMENDATIONS

FINAL REPORT

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16. Abstract (Limit: 200 words)

In a simply supported composite bridge girder the concrete is in compression over the full length of the bridge, whereas in a continuous multispan bridge some regions of the bridge are subjected to negative bending moment, in which case the concrete goes into tension. With respect to fatigue behavior of stud shear connectors, one key difference between placing studs in the positive and negative moment regions is the stress state of the base metal of the beam flange. The fact that the beam flange is in tension at the base of the studs in the negative moment region raises questions about whether this affects their fatigue endurance. This report summarizes the research behind the fatigue design provisions for shear connectors in the negative moment region in the American Association of State and Highway and Transportation Officials (AASHTO) Load and Resistance Factor Design (LRFD) Specification.

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EXECUTIVE SUMMARY

In a simply supported composite bridge girder the concrete is in compression over the full length of the bridge, whereas in a continuous multispan bridge some regions of the bridge are subjected to negative bending moment, in which case the concrete goes into tension. Neglecting any natural bond between the concrete slab and the steel girders, composite action is only present in the positive moment regions when shear stud connectors are not provided in the negative moment regions. In contrast when shear stud connectors are provided throughout the entire length of the composite bridge, composite action between the concrete slab and the steel section is present in both the positive and negative moment regions. With respect to fatigue behavior of stud shear connectors, one key difference between placing studs in the positive and negative moment regions is the stress state of the base metal of the beam flange. The fact that the beam flange is in tension at the base of the studs in the negative moment region raises questions about whether this affects their fatigue endurance, and how the American Association of State Highway and Transportation Officials (AASHTO) Load and Resistance Factor Design (LRFD) Specification (1994) accounts for this. In fact before much research had been conducted on the behavior of stud shear connectors in continuous bridges, some argued that providing stud shear connectors throughout the full length of the bridge might be harmful to the stud shear connectors due to the presence of any residual stresses at the location of the welded studs and consequently decrease their fatigue endurance. Consequently, this report seeks to determine how the AASHTO LRFD Specification accounts for stud shear connectors in the negative moment region.

The actual behavior of stud shear connectors in composite beams is a complex interaction between axial force, shear force, frictional forces, and bending moment. Consequently, the static strength, fatigue endurance, and ductility of stud shear connector cannot be determined theoretically. Instead, these properties need to be determined experimentally. Common tests used for these experiments are beam tests and push-out tests. The AASHTO LRFD provisions for static and fatigue strength of shear connectors is based largely on push-out tests, while the primary research on the behavior of welded fatigue details such as the base metal adjacent to the shear connectors in the negative moment regions are based largely on beam tests.

Experimental push-out test studies showed the stress range was more important to the cyclic life of the specimens than mean or maximum stress, and the strength of the concrete had a minor effect on the cyclic life. Also, it was determined that the minimum stress was significant to the cyclic life when stress reversal was included. However, because stress reversal increased the cyclic life for the same stress range, it was considered conservative to omit these data for design recommendations (Slutter and Fisher, 1966). Extensive analysis on push-out test data with data from previously conducted beam tests revealed that push-out tests are more conservative. In fact, the mean behavior of push-out tests corresponds to the lower bound of the dispersion for the beam tests, which represent a lower 95 percent confidence limit of these data. Comparison of fatigue push-out test results also showed that behavior of different sizes of stud shear connectors was similar. These observations allowed for the static and fatigue design equations of stud shear connectors that are currently used in the AASHTO LRFD Specifications (AASHTO, 1994, Article 6.10.7.4.4c-1, and Article 6.10.7.4.2, respectively) to be derived from push-out tests.

The concern about the connecting welds for the stud shear connectors in the negative moment regions being potential failure regions in fatigue due to the presence of any stress raisers and decrease their fatigue endurance, has been evaluated by testing a statistically significant number of full-scale beams containing a wide range of all connection details presently used in construction (Keating and Fisher, 1986; Fisher et al., 1993). As a result of these fatigue beam tests the AASHTO LRFD Specification (1994) requires that the base metal of the stud shear connector weld to the top flange be designed for load induced fatigue. In addition, stud shear connectors have been verified to belong to the general category of longitudinally fillet-welded attachments and are classified as a fatigue weld Detail Category C. However, “only details subjected to a net applied tensile stress” need to be checked and in “regions where the unfactored permanent loads produce compression, fatigue shall be considered only if this compressive stress is less than twice the maximum tensile live load stress resulting from the fatigue load combination” (AASHTO 1994 Article 6.6.1.2.1). The fatigue strength and life is affected by the size of the welded attachments. Consequently, the AASHTO LRFD Specification (1994) includes a thickness effect by limiting the overall diameter of stud shear connectors of fatigue Category C to a maximum of 2 in.

1. INTRODUCTION

In a simply supported composite bridge girder the concrete is in compression over the full length of the bridge, whereas in a continuous multispan bridge some regions of the bridge are subjected to negative bending moment, in which case the concrete goes into tension. Neglecting any natural bond between the concrete slab and the steel girders, composite action is only present in the positive moment regions when shear stud connectors are not provided in the negative moment regions. In contrast when shear stud connectors are provided throughout the entire length of the composite bridge, composite action between the concrete slab and the steel section is present in both the positive and negative moment regions. However, since the concrete in the negative moment region can only withstand tensile forces at magnitudes much smaller than the steel section, the concrete often cracks at working loads. As a result, for design purposes, only the reinforcement and the steel girder compose the entire composite action in the negative moment region. Based on these facts, the current AASHTO LRFD Specification (1994) states that for continuous bridges, stud shear connectors in the negative moment region are only required when the longitudinal reinforcement is taken into account as part of the composite section. Otherwise, shear connectors need not be provided in the negative flexure regions. With respect to fatigue behavior of stud shear connectors, one key difference between placing studs in the positive and negative moment regions is the stress state of the base metal of the beam flange. The fact that the beam flange is in tension at the base of the studs in the negative moment region raises questions about whether this affects their fatigue endurance, and how the AASHTO LRFD Specification (1994) accounts for this.

This report outlines the background behind the AASHTO fatigue provisions, focusing on explaining the fatigue behavior and design of stud shear connectors in the negative moment region of composite steel girder bridges. The report begins with a summary of experimental research that forms the basis of the AASHTO provisions for fatigue design of stud shear connectors. Next, the report outlines the development of the AASHTO provisions for fatigue design of the stud shear connectors as well as the base metal at the location of welded attachments (of which studs are considered a subset). The discussion focuses on the specific research conducted relating to welded attachments in the negative moment region, and provides conclusions on the AASHTO provisions for fatigue of stud shear connectors in the negative moment region. For further information and details on analytical and experimental studies on stud shear connectors the reader is referred to Appendices A, B, and C. The appendicies respectively contain a summary of push-out test setup configurations and the relation of results from push-out tests to beam tests, synopses of analytical and experimental studies on stud shear connectors and welded attachments, and tables summarizing experimental data and results.

2. DEVELOPMENT OF FATIGUE PROVISIONS FOR STUD SHEAR CONNECTORS

Before much research had been conducted on the behavior of stud shear connectors in continuous bridges, there was some uncertainty about the fatigue strength and the life of stud shear connectors and whether or not it was beneficial to include them in the negative moment regions. In fact, as early as 1953, Viest and Siess (1953) commented that providing stud shear connectors in the negative moment region of the bridge might be detrimental to the stud shear connectors due to the presence of tensile residual stresses at the location of the welded studs, which would consequently decrease their fatigue endurance.

In addition, based on numerous experimental and theoretical studies at the University of Illinois, Viest and Siess (1953) showed that wheel loads were distributed transversely similarly in a simple span bridge as in a continuous bridge with identical cross section. From both analytical and experimental studies, it had also been found that the transverse distribution of negative moments in continuous bridges at an intermediate support were similar to the distribution of positive moments at the maximum positive moment region (the distributions corresponded better for several point loads than for a single concentrated load acting on the bridge). Thus, they concluded that the presence or absence of shear connectors in the negative moment region had no significant effect on the static behavior of the composite bridge (Viest and Siess, 1953).

Several years later, Viest et al. (1958) reached a similar but more detailed conclusion. By providing shear connectors throughout the entire length of a continuous bridge, vertical separation between the concrete and the steel section is eliminated and flexural conformance throughout the continuous bridge is maintained. However, the length of the negative moment region is relatively short in comparison to the full span length and the concrete, instead of pulling away from the steel section as in the positive moment region, exerts pressure on the girders when loaded. Thus, as long as the concrete slab is continuous throughout the length of the bridge, the slab reinforcement in the negative moment regions interacts substantially with the steel girders regardless of whether shear connectors are provided in this region, but to different degrees. From their experimental and theoretical studies, it had been shown that in bridges with shear connectors throughout the entire length, the reinforcement was fully effective. When shear connectors were only provided in the positive moment region, the slab reinforcement was only partly effective compared to when shear connectors were provided in the negative moment region. However, because the area of the reinforcement may be small in comparison to the steel section, providing shear connectors in the negative moment region may only increase the flexural strengths and stiffness by a small percentage. Therefore, they concluded that adding shear connectors in the negative moment may not always be worthwhile.

2.1 RESEARCH ON AASHTO FATIGUE DESIGN PROVISIONS FOR STUD SHEAR CONNECTORS

While this early work considered the behavior of stud shear connectors in the negative moment region, it did not specifically address the fatigue resistance of studs when the base metal is in tension. However, these early experiments did provide the precursor for subsequent research that led to the development of the current AASHTO LRFD provisions for fatigue strength of stud shear connectors.

The actual behavior of stud shear connectors in composite beams is a complicated interaction between axial force, shear force, frictional forces, and bending moment. Consequently, the static strength, fatigue endurance, and ductility of stud shear connectors are difficult to determine theoretically. Instead, these properties are often determined experimentally. Common tests used for these experiments are beam tests and push-out tests. The AASHTO LRFD provisions for static and fatigue strength of shear connectors are based largely on push-out tests, while the primary research on the behavior of welded fatigue details such as the base metal adjacent to the shear connectors in the negative moment regions are based largely on beam tests.

Beam tests are more difficult to analyze because the connectors are not loaded directly and their strength and fatigue endurance depend on the material stiffness of several components in the composite beam. In addition, residual stresses and the nonlinearity of the concrete and steel make it difficult to accurately determine the actual idealized behavior of stud shear connectors in composite beam tests. Furthermore, beam tests are expensive relative to push-out tests when conducted in full-scale. Small-scale tests have been used. However, they tend not to be as accurate (Keating and Fisher, 1986). For these reasons, the static and fatigue design equations for stud shear connectors that are currently used in the AASHTO LRFD Specification (1994) for composite steel bridge girders are derived predominantly from push-out tests. The results from these tests often vary because of differences in the push-out specimens and their external restraints, as well as the applied loading scheme. Johnson and Oehlers (1981) reported several variables that influence push-out strength including “the thickness of the steel flange; the shank cross section, height, tensile strength, and position of the studs; the properties of the concrete; the amount, location, and anchorage of the transverse reinforcement; and the width of the slab and the constraints imposed on it” (Johnson and Oehlers, 1981).

Push-out specimens exhibit many failure modes (Johnson and Oehlers, 1981). The most common ones include fracture of the shank of the stud shear connector and failure of the weld or base metal. Other failure modes include embedment or pull-out failure, splitting of the slab, and shear failure of the slab. These failure modes can often be prevented by using a sufficient stud height and adequate reinforcement.

Nevertheless, the conventional push-out specimen does not always correctly simulate the actual behavior of the stud shear connector because the boundary conditions of the steel and concrete components in a composite beam may be quite different. Evaluation of stud shear connector’s actual behavior is therefore best performed in a beam test. The type of external constraint imposed on the push-out specimen has a significant effect on the static and fatigue strength of stud shear connectors. Push-out specimens can mainly be categorized into two groups: those with external restraints, which induce a resultant axial compressive force in the stud shear connector, and those with no external restraints, which induce axial tensile force in the stud shear connector (Oehlers, 1990b; Oehlers and Bradford, 1995).

For more detail on the stud shear connectors in push-out tests, the reader is referred to Appendix A and Appendix B, which contain, respectively, a summary of various push-out specimens that have been designed, their relation to composite beam tests, and synopses of all significant push-out tests and beams tests relating to strength and fatigue of stud shear connectors. The next section describes the experimental research leading to the AASHTO fatigue provisions, which are covered in greater detail in Appendix B.

2.2 EXPERIMENTAL RESEARCH LEADING TO AASHTO FATIGUE PROVISIONS

Based on monotonic push-out tests (Viest, 1956), it was found that concrete strength has a substantial impact on the load capacity of stud shear connectors. In fact, at a given amount of slip, an increase in concrete strength yields an increase in load capacity of stud shear connectors that is approximately proportional to the square root of compressive strength of concrete. The stud diameter size was also found to affect the load-carrying capacity of stud shear connectors. In fact, the critical load for stud shear connectors with a diameter smaller than 1 in. was found to be approximately proportional to the square of the diameter of the stud. Furthermore, the critical load for stud shear connectors with a diameter larger then 1 in. was found to be approximately proportional to the diameter of the stud. From these observations, the author was able to determine critical load equations for stud shear connectors of different diameters. Based on these experimental test results, tentative design equations describing the monotonic strength of stud shear connectors were developed. The proposed design equations were semi-empirical and empirical expressions for shear stud connectors having a height of 4 in. Based on previously reported tests on flexible dowels (Viest, 1956; Oehlers and Bradford, 1995), it was recognized that the height of shear stud connectors has little, if any, effect on the load capacity and therefore these design equations may be used for studs 4 in. high or higher. By comparison of the load-slip and residual slip curves, it was discovered that the spacing of studs used in this experiment had no effect on the distribution of the shear forces onto the connectors. Thus the author concluded that uniform spacing of shear stud connectors may be used in the design of composite bridges (Viest, 1956).

From additional experimental push-out test studies (Slutter and Fisher, 1966), it was found that the stress range was more important to the cyclic life of the specimens than maximum stress, and the strength of the concrete had a minor effect on the cyclic life. By comparison of the variance (i.e., the square of the standard deviation) of the test data, it was determined that the minimum stress (or largest reversed stress) was only significant to the cyclic life when stress reversal was included. Because stress reversal increased the cyclic life for the same stress range, it was considered conservative to omit this data for design recommendations (Slutter and Fisher, 1966).

In order to determine the differences in the fatigue strength results between beam tests and push-out tests, analyses were conducted comparing the results from these two tests. Earlier experimental work on beam tests showed that there was a difference in fatigue strength between ½ in. and ¾ in. stud in beam tests (Toprac, 1965). In fact, the fatigue strength of ½ in. studs was higher throughout the cyclic life with a difference of as much as three ksi magnitude in the one to three million cycle range. The fatigue life of beams with ½ in. stud connectors was therefore longer than beams with ¾ in. studs. However, the slopes of the S-N curves for ½ in. and ¾ in. stud connectors were approximately the same (Toprac, 1965; King et al., 1965). Conversely, the analyses of push-out test data determined that the behavior of the $^{7}/_{8}$ –in. stud shear connectors was very similar to the ¾ -in. connectors. This performance implied that the fatigue strengths of the two stud shear connector in push-out test were not significantly different.

In a comparison between the push-out test data and data from previously conducted beam tests, it was determined that push-out tests are more conservative. In fact, the mean behavior of push-out tests corresponded to the lower bound of the dispersion for the beam tests, which represented a lower 95 percent confidence limit of these data. Even though beam tests represent a more accurate illustration of a true composite decking, the push-out test is widely used in tests of shear connectors because it is less expensive (Oehlers and Bradford, 1995). These observations justify that a fatigue design equation can be derived from push-out tests, since it primarily will result in a slightly conservative design equation compared to if the equation were based solely on beam tests.

In addition, this experimental study also showed that uniform spacing of connectors is satisfactory for most loading conditions because the shear load gets redistributed, and placing shear connectors in the negative moment region is beneficial to assist in maintaining flexural conformance throughout the continuous beam (Slutter and Fisher, 1966).

2.3 STATIC AND FATIGUE DESIGN EQUATIONS OF STUD SHEAR CONNECTORS

As seen in the previous section, push-out tests yield conservative fatigue results compared to composite beams tests and therefore allow for fatigue design equations to be derived from push-out test results. Furthermore, it was discussed that fatigue strength and endurance of stud shear connectors in composite beams decrease with increase in stud diameter. However, because the fatigue provisions included tests with a wide range of stud diameters, this variation was accounted for in the design equations. The current design equations for both static and fatigue strength of stud shear connectors have thus been determined from extensive experimental testing on push-out specimens. Ollgaard et al. (1971) determined the static strength (nominal shear resistance) of stud shear connectors embedded in both normal-weight and lightweight concrete slabs. Nominal shear resistance of a stud shear connector, Q_n, is referenced in the AASHTO LRFD Specification (AAHSTO, 1994, Article 6.10.7.4.4c) as:

$$Q_n = 0.5 \cdot A_{sc} \cdot \sqrt{f'_c \cdot E_c} \leq A_{sc} \cdot F_u \tag{2.1}$$

where

A_{sc} = cross-sectional area of stud shear connector, in^2
f'_c = specified 28-day compressive strength of concrete, ksi
E_c = modulus of elasticity of concrete, as specified in Article 5.4.2.4, ksi
F_u = specified minimum tensile strength of a stud shear connector specified in Article 6.4.4, ksi

This equation has an upper bound on the nominal shear strength of stud shear connectors, which is the product of the cross-sectional area of the stud and its ultimate tensile strength. In order to develop full capacity with this equation, the authors determined that the height-to-diameter ratio for stud shear connectors should be kept at approximately 4 or higher.

The equation for fatigue resistance of an individual shear connector, Z_r, was developed by Slutter and Fisher (1966), and is given in the AASHTO LRFD Specification (Article 6.10.7.4.2) as:

$$Z_r = \alpha \cdot d^2 \geq 5.5 \cdot d^2 \tag{2.2}$$

for which:

$$\alpha = 34.5 - 4.28 \cdot \log N \tag{2.3}$$

where

d = diameter of the stud, in

N = number of cycles to failure

The lower bound to this equation establishes the constant amplitude fatigue limit.

3. DEVELOPMENT OF FATIGUE DESIGN PROVISIONS FOR BASE METAL

Section two outlined the experimental research leading to the AASHTO (1994) fatigue design provisions for stud shear connectors, as well as the provisions themselves. As stated in the AASHTO LRFD Specification (1994), shear connectors should be provided in regions of negative moment when the longitudinal reinforcement in the concrete is assumed to be part of the composite section. Otherwise, shear connectors are not required in the negative moment regions. When shear connectors are only provided in the positive moment region in a continuous bridge the slab still offers some degree of restraint to the girders in the negative moment region, since it is anchored to the girders in the regions of positive moment (Viest and Siess, 1953). However, by providing shear connectors in the regions of negative flexure, the sudden transition from a composite to a non-composite section when they are not included is prevented and reduces the danger of fatigue failure in connectors in this transition region (Slutter and Fisher, 1966). However, fatigue design of attachments to the tension flange in the negative moment remains a concern. This chapter outlines the research and development specific to fatigue design of the base metal of welded attachments (of which stud shear connectors are considered as subset). The following sections cover the influence of residual stresses on the fatigue behavior of stud shear connectors, the results from experimental beam tests on welded attachments in the negative moment region, and the fatigue design equations of base metal adjacent to stud shear connectors.

3.1 INFLUENCE OF RESIDUAL STRESSES ON THE FATIGUE BEHAVIOR OF STUD SHEAR CONNECTORS

Residual stresses are introduced into the base of the stud and base metal as a result of the welding process. The residual stresses are dependent on several factors, but primarily on the increase in temperature during welding and the subsequent cooling rate. The cooling process develops tensile residual stresses in the surrounding metal of the stud shear connector, primarily in the base metal and the weld. Equilibrium will be met by developing local areas of compressive residual stresses around the tensile stress region, again primarily in the base metal and weld (Fisher et al., 1997).

The fatigue strength and life, which is related to the actual distribution and magnitude of the residual stress pattern, has been shown from previous experimental work to be primarily dependent upon the strength of the steel and the weld metal, the geometry of the parts, the size of the weld relative to the connected parts, and by the stress conditions due to the applied load (Fisher et al., 1970). One important fact that raises concerns about the tensile residual stresses is that magnitude of these stresses can reach the yield strength of the material, which consequently decreases the fatigue life of the component (Fisher et al., 1997).

Locations with large tensile residual stresses are often sources of fatigue crack growth and have a significant effect on the fatigue life of the structure because the local tensile stresses, due to applied loading, are augmented and thus decrease the fatigue endurance of that structural component. Through extensive beam tests with welded attachments, it has been shown that residual stresses are beneficial when they decrease the magnitude of the tensile stress due to applied loading, such as in regions of compressive residual stresses. In fact, examination of beam tests has shown that compressive residual stresses in welded connections can effectively stop the propagation of large cracks (Braithwaite and Gurney, 1967). When tensile residual stresses are present, the values of the minimum stress due to applied load, even if compressive (i.e., under service loads), seems not to significantly affect the relationship between the applied nominal stress range and the growth of the crack, if the crack never closes during application of load (Fisher et al., 1970). Minimum stresses only have an effect when it causes the crack to close. In fact, values of minimum stresses that cause the crack to close would decrease the effective stress range that contributes to the crack growth and subsequently increase the fatigue life.

This past research on the influence of residual stresses of fatigue crack initiation and growth is critical to the fatigue behavior of the base metal of welded attachments in the negative moment region. The next section summarizes the results of experiments of beams with welded attachments, which led to the AASHTO fatigue provisions for the base metal adjacent to the stud shear connectors.

3.2 RESULTS FROM EXPERIMENTAL BEAM TESTS ON WELDED ATTACHMENTS IN THE NEGATIVE MOMENT REGION

Throughout the years, extensive research has been conducted to more accurately determine the fatigue behavior of attachments of steel components and to address the overall effects of reversed loading on fatigue endurance. This research was primarily done through extensive fatigue testing programs with multiple full-scale tests on wide flange I-beams (Fisher et al., 1970; Fisher et al., 1974; Keating and Fisher, 1986; Fisher et al., 1993). In order to obtain more information about the fatigue endurance of each welded detail, many of the studies welded several attachments to each beam specimen. The weld details included the most common types of welds used in practice. In some studies, the specimens were repaired after cracking had occurred, such that each specimen could be forced to yield more than one crack. From these tests, mean S-N curves and the lower 95 percent confidence limit, (i.e., the 97.5 percent one-sided confidence limit) S-N curves were determined for each individual weld detail attachment.

For design purposes, these studies showed that the fatigue strength of a given welded detail is independent of the strength of steel and that the stress range is the dominant stress parameter for all steel types, beam types, and weld details tested. In addition stress variables such as minimum stress, mean stress, and maximum stress were shown to be negligible for the fatigue life of welded attachments (Fisher et al., 1970; Fisher et al., 1974; Keating and Fisher, 1986).

The final relationships relating nominal stress range to the fatigue life for design purposes (i.e., S-N curves) were derived from the lower 95 percent confidence limits for 95 percent survival. This was performed with a regression analysis of the fatigue test data. By basing the S-N curves on the 95 percent confidence limit, variations in the test results were accounted for, as well as providing a uniform estimate of the survival. Only stress ranges that caused tensile or reversal stresses were included.

In these experimental tests of single span I-beams, it was noted that failure mainly occurred in the tension flange of all beams with flange attachments. These failures usually originated in regions of high residual tensile stresses such as the weld toe terminations connecting the attachment to the flange. Cracks were also observed in the compression flange again in regions of high residual tension stress such as at the weld toe terminations during both pure compression-compression as well as stress reversal fatigue tests. In the former tests, cracks usually grew more slowly after they had grown out of the residual tension region and were then arrested. None of these beams failed at a welding detail subjected to pure compression-compression stress cycles (Fisher et al., 1970; Fisher et al., 1974). However, when these beams that had developed cracks in the compression flange were subjected to stress reversal, thus putting the compression flange into tension, several of them failed by fracture of the flange now in tension. These failures of the original compression flange sometimes occurred within the life of the original tension flange. In the latter tests, cracks were also formed in regions of high residual stresses. However, many of these stress reversal specimens exhibited longer fatigue life than their counterparts tested in pure tensile fatigue tests. The reason for the prolonged life during stress reversal is because only half of the applied loading cycle (i.e., the tension part of the applied loading cycle) contributes to the growing of the cracks. Conversely, the compression part of the applied loading prevents the growing of the crack by closing the gap, which subsequently prolongs the fatigue life of welded attachments during stress reversal compared to tensile fatigue tests.

In order to more closely analyze and determine the differences between the tensile loading and the stress reversal fatigue tests, a statistical analysis was conducted on the distribution of occurrences of cracking of longitudinal fillet welds for stress ranges as a function of number of cycles at cracking failure (Fisher et al., 1993). Three separate analyses were compared. The first study investigated only tests with positive load ratios, where the top flange was put in tension. The second study included only tests with negative load ratios, where the top flange sometimes was put in compression (i.e., reversal). The third study investigated the overall effect of fatigue cracking of longitudinal fillet welds by including all specimens that failed by fracture.

Several histograms were created and both a Gaussian and a Weibull distribution were fit to the cycles-to-failure data in order to make a probabilistic analysis of all the fatigue test data and the fatigue effect of welded attachments in the negative moment region. By a comparison of statistical distribution of occurrence of cracking of longitudinal fillet welds for all stress ranges, for negative load ratio only (i.e., reversal), and positive load ratio only (i.e., tension), it was discovered that the mean and the standard deviation of the stress reversal data was greater than the tension only data. These results verified the earlier conclusion that the stress reversal tests in fact exhibited longer fatigue life than tensile fatigue tests. On the other hand, inspection of the lower 95 percent confidence limit (i.e., mean stress minus two standard deviations) showed that there was no significant difference between the stress reversal tests and tension fatigue tests. This observation is important for design purposes because attachments in the negative moment region can therefore be designed similar to attachments in the positive moment region.

As a result of this testing, it was verified that stud shear connectors belong to the general category of longitudinally fillet-welded attachments and should be classified as Detail Category C (Keating and Fisher, 1986). In addition, AASHTO LRFD Specification (1994) requires that the base metal adjacent to a stud shear connector weld be designed for load induced fatigue. However, "only details subjected to a net applied tensile stress" need to be checked, and in "regions where the unfactored permanent loads produce compression, fatigue shall be considered only if this compressive stress is less than twice the maximum tensile live load stress resulting from the fatigue load combination" (AASHTO 1994 Article 6.6.1.2.1). AASHTO provisions thus do appropriately categorize the base metal adjacent to stud shear connectors in the negative moment region as being Detail Category C. The next section outlines these provisions.

3.3 FATIGUE DESIGN EQUATIONS OF BASE METAL ADJACENT TO STUD SHEAR CONNECTORS

In the fatigue design of welded steel structures, the welding components are typically separated into weld detail categories. The AASHTO LRFD Specification (1994) lists eight categories that range from A to E' (i.e., A, B, B', C, C', D, E, and E') in order of decreasing fatigue strength. Note that in the previous AASHTO specification (1989), a ninth category, Detail Category F, was present for fillet welds, plug welds, and slot welds in shear in any direction. However, because there have not been many failures related to this shear category, it has been eliminated in the current specification (Fisher et al., 1993). This category was not as well defined and has been conservatively replaced by Detail Category E. For the base metal at the base of stud shear connectors, Detail Category C is used. The fatigue strength and life is affected by the size of the welded attachments. However, AASHTO LRFD Specification (1994) reflects a thickness effect by limiting the overall diameter of stud shear connectors of Detail Category C to a maximum of 2 in.

The fatigue welding categories were obtained from testing a statistically significant number of full-scale beam specimens containing a resemblance of all connection details presently used in construction. These tests were conducted to simulate the actual behavior of each detail before selecting its appropriate fatigue-welding category. Included in the database were fatigue data from the NCHRP test program, which was researched in the previous section. Japanese, Canadian, and European test data were also included, as was data from other sources.

The intent was to make this database as comprehensive as possible by including all main fatigue test data performed on beam specimens with welded attachments. This database mainly contains fatigue test data from large-scale test specimens. The small-scale specimens were minimized to cases where there was no alternative fatigue data (Keating and Fisher, 1986). The final database was broadened to include a wider range of detail types and sizes of welds. This relatively large data base of laboratory results was taken from tests of different steel grades produced in several countries, but has been justified as being independent of steel grade.

As determined from previous experiments, the stress range is the main stress variable that determines the fatigue resistance of a steel components. From this fact, a stress range-cycle life relationship was developed. Studies of test results have shown that the relationship between the stress range and cycle life is log-log in nature with approximately constant sloped between each detail. The S-N curves are defined in log form by (Fisher et al., 1993):

$$N = C \cdot S^{-m} \tag{3.1}$$

The fatigue data is usually plotted with the logarithm of the nominal stress range as a function of the logarithm of the number of cycles to failure so as to obtain a linear relationship of the form:

$$\log N = \log C - m \cdot \log S \tag{3.2}$$

where

N = number of cycles to failure

C = constant dependant on detail category (1994, Table 6.6.1.2.5-1)

S = applied constant amplitude stress range

m = the inverse of the slope of the S-N curve

Through extensive testing, it was found that the slope of each category detail is approximately the same. Therefore the slope, *m*, was made parallel and normalized to a constant slope of 3.0 for consistency within U.S. specifications and with many other international design rules, such as Eurocode (Keating and Fisher, 1986; Fisher et al., 1993). For Detail Category C, the value of the detail category constant is 44.0×10^8.

From full-scale test data, the constant-amplitude fatigue limit (CAFL) for each category was also determined (Keating and Fisher, 1986). Because most structures are designed for long life, the CAFL is perhaps the most important property of the S-N curve. In particular, significant cracking should not occur when constant-amplitude tests are conducted at stress ranges below the CAFL. For Detail Category C, the CAFL value is 10 ksi at approximately 4.4 million cycles.

4. SUMMARY AND CONCLUSIONS

The actual behavior of stud shear connectors in composite beams is a complex combination of axial force, shear force, frictional forces, and bending moment. Consequently, the static strength, fatigue endurance, and ductility of stud shear connectors cannot be determined theoretically. Instead, these properties need to be determined experimentally. Common tests used for these experiments are beam tests and push-out tests.

Methods of estimating the static and fatigue endurance of stud shear connectors in composite bridge beams have been derived primarily from the analysis of an extensive collection of push-out tests. The fatigue endurance of stud shear connectors in composite beams is usually derived from push-out specimens instead of beam tests because they are easier to analyze (Oehlers and Bradford, 1995). In addition, the residual stresses and the nonlinearity of the concrete and steel make it difficult to accurately determine the actual idealized behavior of stud shear connectors in composite beam tests. Moreover, beam tests are expensive relative to push-out tests conducted in full-scale (Keating and Fisher, 1986; Oehlers and Bradford, 1995).

From experimental push-out test studies, it was found that the stress range was more important to the cyclic life of the specimens than mean or maximum stress, and that the strength of the concrete had a minor effect on the cyclic life. By comparison of the variance of the test data, it was determined that the minimum stress was significant to the cyclic life when stress reversal was included. However, because stress reversal increased the cyclic life for the same stress range, it was considered conservative to omit this data for design recommendations (Slutter and Fisher, 1966).

Extensive analysis on push-out test data with data from previously conducted beam tests revealed that push-out tests are more conservative. In fact, the mean behavior of push-out tests corresponds to the lower bound of the dispersion for the beam tests, which represent a lower 95 percent confidence limit of these data. Comparison of fatigue push-out test results also showed that behavior of the 7/8 –in. stud shear connectors was very similar to the ¾ -in. connectors.

These observations allowed design equations of stud shear connectors to be derived from experimental push-out test results (Slutter and Fisher, 1966; Ollgaard et al., 1971).

From thorough analysis of push-out test results, Ollgaard et al. (1971) determined the static strength (nominal shear resistance) of stud shear connectors, Equation 2.1. This equation is used in the current AASHTO LRFD Specification (AASHTO, 1994, Article 6.10.7.4.4c-1). The equations for fatigue resistance of an individual shear connector, Equations 2.2 and 2.3, were developed by Slutter and Fisher (1966), and is given in the current AASHTO LRFD Specification (AASHTO, 1994, Article 6.10.7.4.2).

The fatigue welding detail categories for the metal at the base of the stud shear connectors were obtained from testing a statistically significant number of full-scale beams containing a wide range of all connection details presently used in construction. Tests were conducted to simulate the actual behavior of each detail before selecting its appropriate fatigue-welding category. Included in the fatigue database were data from the NCHRP test programs, Japanese, Canadian, and European test data, as well as other sources (Keating and Fisher, 1986). The intent was to make this database as comprehensive as possible by including all main fatigue test data performed on beam specimens with welded attachments.

The concern about the connecting welds for the stud shear connectors in the negative moment regions to be potential failure regions in fatigue due to the presence of any stress raisers and decrease its fatigue endurance has been evaluated by the NCHRP test program and many others (Keating and Fisher, 1986; Fisher et al., 1993). From extensive experimental fatigue studies on beams with fillet welded attachments (of which stud shear connectors were not included but were implicitly assumed to be a subset) on both the tension and compression flanges in the negative moment region, it was found that Detail Category C was appropriate for evaluating the fatigue behavior of metal at the base of stud shear connectors in the negative moment region (Keating and Fisher, 1986).

As a result of these beam tests the AASHTO LRFD Specification (1994) requires that the base metal of the stud shear connector weld to the top flange be designed for load induced fatigue. In addition, stud shear connectors have been verified to belong to the general category of longitudinally fillet-welded attachments and are classified as a fatigue weld Detail Category C. However, "only details subjected to a net applied tensile stress" need to be checked, and in "regions where the unfactored permanent loads produce compression, fatigue shall be considered only if this compressive stress is less than twice the maximum tensile live load stress resulting from the fatigue load combination" (AASHTO 1994 Article 6.6.1.2.1). The fatigue strength and life is affected by the size of the welded attachments. Consequently, the AASHTO LRFD Specification (1994) includes a thickness effect by limiting the overall diameter of stud shear connectors of fatigue Category C to a maximum of 2 in.

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REFERENCES

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APPENDIX A

SUMMARY OF PUSH-OUT SPECIMENS AND THEIR RELATION TO BEAM TESTS

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SUMMARY OF PUSH-OUT SPECIMENS AND THEIR RELATION TO BEAM TESTS

This appendix includes a summary of push-out specimens and their relation to beam tests. First, dowel action of stud shear connectors in a composite beam is described. Second, conventional push-out tests are described. Finally, research using a modified push-out specimen, which was developed to give the stud shear connectors similar behavior as in a composite beam, is summarized.

A.1 DOWEL ACTION OF STUD SHEAR CONNECTORS IN A COMPOSITE BEAM

In a composite beam member, there is relative slip movement between the concrete and the steel section, which is resisted by the dowel action of the stud shear connector (manifesting itself through shear, flexure, and axial force in the stud shear connector), as shown in the figure {Fig. A.1 [after (Oehlers and Bradford, 1995)]}. In the figure the shear connector is moving to the right and the concrete on the right has to withstand a compressive force, *F*, in the bearing zone. This force is in horizontal equilibrium with the shear force in the steel element. Because the two horizontal forces are separated by a distance, *e*, a counterclockwise rotational moment is induced. In order to equilibrate the stud shear connector, a clockwise moment at the base results.

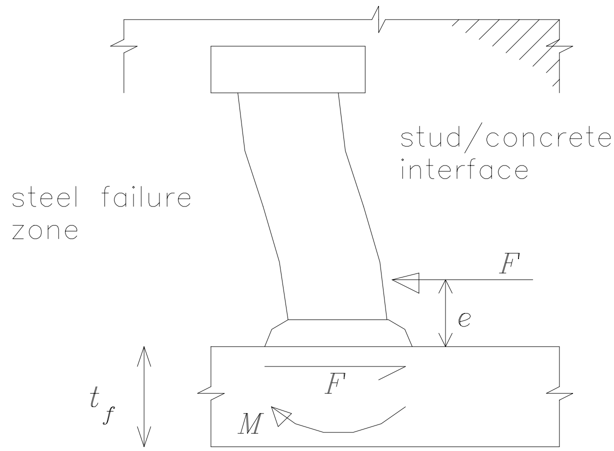

Figure A.1: Stud Shear Connector in a Composite Member [after (Oehlers and Bradford, 1995)]

In a bridge beam, the eccentricity at which the compressive force acts is dependent on the relative material stiffness of the concrete compared to the stiffness of the stud shear connector. As the material stiffness of the concrete decreases, the eccentricity will increase, and therefore induce a larger flexural stress at the base of the stud shear connector, which results in failure at a lower shear load. Consequently, by increasing the strength and material stiffness of the concrete, the shear strength of the stud shear connector is increased (Oehlers, 1990; Oehlers and Bradford, 1995).

A.2 CONVENTIONAL PUSH-OUT TEST

A conventional push-out test typically consists of two reinforced concrete slabs attached, by stud shear connectors, to either flange of an I-shaped steel section {Fig. A.2 [after (Oehlers and Bradford, 1995)]}. During testing, load, *2P*, is usually applied vertically at the top of the steel section. This load is transmitted to the concrete slabs through the stud shear connectors as a force *P* at an eccentricity, *e*, similar to a composite beam. Because this eccentricity is small in comparison to the concrete thickness, it is often assumed that the force *P* acts at the interface of the concrete and the steel at a distance $h_2/2$ from the mid-thickness of the slab.

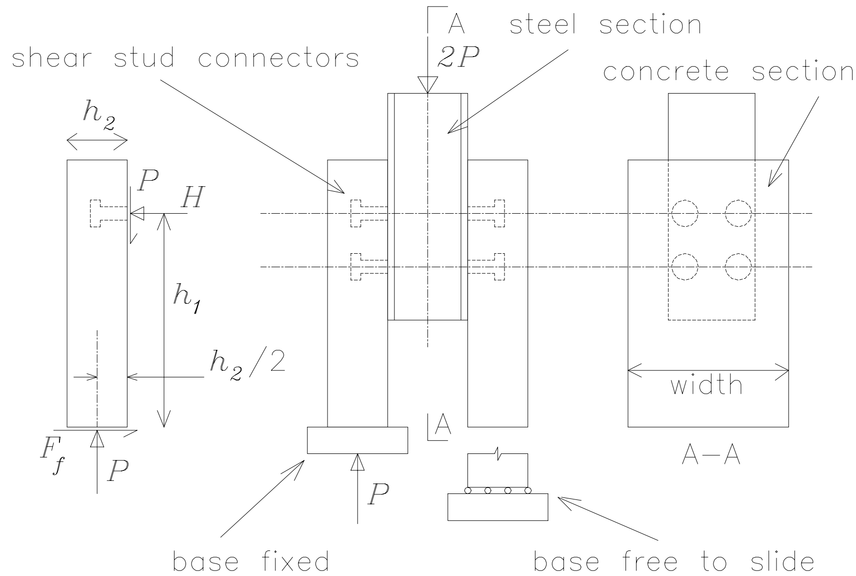

Figure A.2: Conventional Push-Out Specimen [after (Oehlers and Bradford, 1995)]

The external reaction at the base of each concrete slab, P, creates a couple rotating towards the steel section. When the base of the push-out specimens are fully restrained (i.e., not free to slide transversely), there will be an induced frictional force, F_f, below each concrete slab that will yield a couple, $F_f h_1$, and counteract the couple created by the external reaction. In order to equilibrate the horizontal forces of each slab, there will be compressive axial forces, H, introduced in the stud shear connectors of equal magnitude to the frictional force, F_f.

In a push-out specimen with an unrestrained base, there are no frictional forces induced that can equilibrate the couple caused by the external reaction at the base. In this situation, the external reactions at the supports tend to splay the concrete slabs outwards and tensile forces are induced in the studs instead of compressive forces as in the case when the base is restrained.

The main difference between push-out tests and composite beams is the normal force across the steel flange and the concrete slab interface, because bridge beams are not subjected to the same external restraints as those in push-out tests. In the negative moment region there is in fact a compressive normal force across the interface between the concrete and the steel. However, the shear due to friction between the concrete and the steel transferred through this force in a bridge beam has been shown to be small in comparison with the shear strength of the shear stud connectors. For design purposes, the normal force present between the concrete and steel interface in bridge beams can therefore conservatively be assumed to not be present (Oehlers and Bradford, 1995).

The external restraints have a significant effect on the fatigue life on the stud shear connectors, since they determine the direction of the normal force across the concrete and steel interface (Oehlers, 1990; Oehlers and Bradford, 1995). When the axial force in a push-out test is compressive, the static strength as well as the endurance of the stud shear connector is increased compared to when there are no axial forces present. Conversely, tensile axial force in a push-out test will reduce the static strength and decreases the endurance and the fatigue life of the stud shear connector. The results of such a test will be conservative and therefore more appropriate for development of design equations than if the axial forces were compressive.

Throughout the years, there has been extensive research performed on push-out specimens in order to determine the static and fatigue strength of stud shear connectors. In several of these studies, the push-out specimens have been altered to give more accurate results. The main goals of these alterations have been either to minimize the tensile force in the stud shear connector, make the axial forces compressive, or best of all to eliminate the axial forces. Slutter and Fisher (1966) used a push-out specimen with only one concrete slab connected to an I-shaped steel beam. The load was applied to the concrete, which resulted in tensile axial forces in the stud shear connectors and therefore gave conservative results.

A.3 MODIFIED PUSH-OUT TEST

Oehlers and Johnson (1987) developed a new push-out specimen that would better simulate the actual behavior of the stud shear connector. The main difference of this specimen with others developed for experimental testing was that it could be altered such that the direction of the axial forces could be changed or even eliminated. The variation of the external restraint is the main parameter that determines the induced axial forces in the push-out specimen. The specimen was placed on rollers, eliminating any frictional force {Fig. A.3 [after (Oehlers and Bradford, 1995)]}. Adjustable knife-edge supports were used to concentrate the external reaction at a predetermined location.

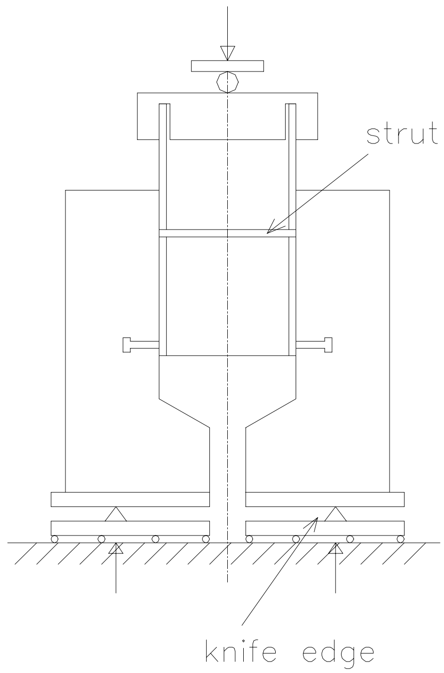

Figure A.3: Modified Push-Out Specimen [after (Oehlers and Bradford, 1995)]

By placing the knife-edge supports to the sides of the steel base, the concrete slabs would tend to splay inwards and subject the stud shear connectors to tensile axial forces. Conversely, moving the supports inwards would develop compressive forces in the stud shear connectors. With this push-out specimen, it was possible to create a setup that would better represent the behavior of stud shear connectors in a composite bridge beam, where predominantly low axial forces are present.

Even though it is possible with this modified push-out specimen to study stud shear connectors under similar conditions as in a composite beam section, it has not been utilized in the development of the static and fatigue provisions in the AASHTO LRFD Specification. On the other hand, it has primarily been used in the development of Australian codes.

Summaries of analytical and experimental studies on push-out and beam tests are included in the Appendix B. The push-out specimens used in these studies vary greatly in setup, construction, and testing. Some of the push-out specimens that are used are conventional push-out specimens, push-out specimens with only one concrete slab, and finally the modified push-out specimen.

APPENDIX B

SUMMARIES OF ANALYTICAL AND EXPERIMENTAL STUDIES ON PUSH-OUT AND BEAM TESTS

APPENDIX B

SUMMARIES OF ANALYTICAL AND EXPERIMENTAL STUDIES ON PUSH-OUT AND BEAM TESTS

This appendix includes a synopsis of all significant push-out tests and beams tests relating to strength and fatigue endurance of stud shear connectors. Symbols used for each reference include:

TYPE OF SPECIMEN TESTED

p = push-out

b = beam

TYPE OF LOADING

c = cyclic

m = monotonic

r = stress reversal

OTHER SYMBOL

a = analytical

Viest, I. M., and Siess, C. P., 1953 (a)

Composite Construction for I-Beam Bridges

Introduction

This paper presents a thorough review of composite construction for I-girder bridges, based primarily on analytical and experimental studies performed at the University of Illinois in cooperation with the Illinois Division of Highways and the Bureau of Public Roads. Much attention is given to the differences between composite and noncomposite construction, effects and benefits of shoring during construction, purpose and different types of shear connectors, and differences between simple span and continuous bridges.

Theoretical Discussion

A detailed theoretical discussion is included about bond connection between the concrete slab and the steel girders. It is noted that this bond is unreliable and should therefore not be considered in design. From analysis, it has been discovered that even shrinkage and warping of the slab may destroy this bond connection. Any connection accounted for in design of composite bridges should therefore be made by mechanical connectors.

In the discussion of continuous bridges, the authors have concluded, based on numerous experimental and theoretical studies at the University of Illinois, that wheel loads are distributed transversely similarly in a simple span bridge as in a continuous bridge with identical cross section. In addition from the studies it has been found that the transverse distribution of negative moments in continuous bridges at an intermediate support are similar to the distribution at the maximum positive moment region. The distributions correspond better for several point loads than for a single concentrated load acting on the bridge. Therefore the presence or absence of shear connectors in the negative moment region has no significant effect on the static behavior of the composite bridge. However, when considering fatigue behavior of composite bridges, it may not be desirable to include the shear connectors in the negative moment region. This conclusion is based on the fact that in the negative moment region, the upper flange of the I-girder is in tension, which causes the connecting welds for the shear connectors to be potential failure regions in fatigue.

Viest, I. M., 1956 (p; m)

Investigation of Stud Shear Connectors for Composite Concrete and Steel T-Beams

Introduction

The main purpose of this paper was to investigate the effect concrete strength, stud spacing, and stud diameter has on the critical load of stud shear connectors. The paper presents the results of a small series of push-out tests and the effects of diameter size of the stud shear connectors, stud spacing, and concrete strength in a composite section. Preliminary design equations for the capacity of stud shear connectors, along with a brief discussion of the comparison between stud and channel connectors, are also presented.

Experimental Study

A series of push-out tests were performed on specimens with varying dimensions and spacing of stud shear connectors. The specimens consisted of two reinforced concrete slabs placed on each side of a standard wide-flange steel section with one row of two stud shear connectors on each side. All the push-out tests were conducted by applying a load concentrically to the steel section in increments until failure occurred. It was assumed that the load was transmitted from the steel sections to the slabs only by the shear connectors and that the load was distributed evenly between every stud. Any friction between the concrete slabs and the steel section was ignored. From these assumptions the load carried by each individual stud was determined by dividing the applied load by the number of studs.

Results and Discussion

By comparison of these test results, it was evident that concrete strength has a substantial impact on the load capacity of stud shear connectors. An increase in concrete strength was seen to cause an increase in load capacity of stud shear connectors that is approximately proportional the square root of the compressive strength of concrete. The stud diameter size was also found to be affecting load-carrying capacity of stud shear connectors. For studs with a diameter larger than 1 in., the critical load was approximately proportional to the diameter of the stud and for studs with diameters smaller than 1 in., the critical load was approximately proportional to the square of the diameter of the stud. From these observations, the author was able to determine the following critical load equations for stud shear connectors with different diameters.

For studs having a diameter d < 1 in.,

$$Q_{cr} = 5.25 \cdot d^2 \cdot f'_c \cdot \sqrt{\frac{4,000}{f'_c}} \tag{B.1}$$

For studs having a diameter d ≥ 1 in.,

$$Q_{cr} = 5 \cdot d \cdot f'_c \cdot \sqrt{\frac{4,000}{f'_c}} \tag{B.2}$$

where

Q_{cr} = critical load, lb

d = diameter of stud shear connector, in.

f'_c = compressive strength of concrete, psi

Design capacity of a stud shear connector can be computed by dividing the critical load with an appropriate factor of safety:

$$Q_{des} = \frac{Q_{cr}}{F.S.} \tag{B.3}$$

where

Q_{des} = design capacity

Q_{cr} = critical load, lb

F.S. = factor of safety

The proposed design equations are semi-empirical and empirical expressions based on experimental tests with stud shear connectors having a height of 4 in. Based on previously reported tests on flexible dowels, it was recognized that the height of stud shear connectors have little, if any, effect on the load capacity, and therefore these design equations may be used for studs 4 in. high or higher. By comparison of the load-slip and residual slip curves, it was discovered that the spacing of studs used in these experiments had no effect on the distribution of the shear forces onto the connectors. The authors concluded that uniform spacing of stud shear connectors may therefore be used in the design of composite bridges.

Fountain, R. S., and Sinclair, G. M., 1956 (p; c, r)

Discussion of Investigation of Stud Shear Connectors for Composite Concrete and Steel T-Beams

Introduction

This paper reconfirms the validity of Viest's (1956) design capacity equation of a stud shear connector. In addition to Viest's investigation of the stud shear connectors, experimental tests were performed at the University of Illinois to investigate the fatigue behavior of 14 welded studs under completely reversed repeated loading.

Experimental Study, Results, and Discussion

The specimens were tested in a setup in which the studs were welded to a base plate. To visually observe the behavior of stud shear connectors during the entire test, this experiment did not include composite action in the test setup. Load was applied at the top of a plate that was welded onto the stud, in order to produce the proper moment arm for the testing machine. An S-N diagram was obtained, along with the failure location of the specimens. It was discovered that not all specimens failed in a similar manner, although they were all tested similarly. Most of the studs failed in the heat-affected zone of the stud. Only one failed ½ -in. from the heat affected zone, and only one failed in the heat-affected zone of the base plate. In addition, some of the specimens had not failed after 12 million cycles, and the tests were ended.

Viest, I. M., Fountain, R. S., and Siess, C. P., 1958 (a)

Development of the New AASHO Specifications for Composite Steel and Concrete Bridges

Introduction

The authors present the new AASHO specifications for composite steel and concrete bridges. A thorough description of background information, analysis and new results from experimental tests is covered, along with information gained from monitoring existing composite bridges upon which the new provisions are based. They also include a comparison of the 1944 and the new 1955 specification, in the areas of design of shear connectors, and present the new design equations. In addition, a detailed discussion of the design of negative moment sections and the effects of deformational stresses, such as creep, shrinkage, and expansion of concrete, in a composite beam are included.

Results and Discussion

The design equations describing the capacity of stud shear connectors presented in the new AASHO specification are semiempirical and empirical expressions derived from test results along with analysis of actual composite bridges. The capacity of stud shear connectors with a height of 4 in or higher is determined by:

For studs having a diameter d < 1 in.,

$$Q_{uc} = 332 \cdot d^2 \cdot \sqrt{f'_c} \tag{B.4}$$

For studs having a diameter $d \geq 1$ in.,

$$Q_{uc} = 316 \cdot d \cdot \sqrt{f'_c} \tag{B.5}$$

where

Q_{uc} = static capacity of stud shear connectors with height of 4 in, lb

d = diameter of the stud shear connector, in.

f'_c = compressive strength of concrete, psi

The same equations also apply to 3 in high studs, however a 15 percent reduction should be applied to account for the size difference.

A discussion is included that describes the derivation of adequate factors of safety as well as variation of the factor of safety for the design of shear connectors for composite and non-composite bridges. Article 3.9.5 of the new specifications provides equations to calculate the appropriate factor of safety in design of composite bridges or permits the use of a constant value of 4.0.

In regards to design of negative moment sections, the new specifications only require shear connectors where the longitudinal reinforcement is considered to be part of the composite section. In addition, no special provisions are needed in design of continuous composite bridges, and elastic analysis in combination with ordinary load distribution factors is considered applicable.

Thürlimann, B., 1958 (b; m, c)

Composite Beams with Stud Shear Connectors

Introduction

The author performed one composite beam fatigue test, including two types of shear connectors. The shear connectors were ¾ -in. diameter 4 in. tall studs and ½ -in. L-shaped connectors. A detailed analysis and interpretation of the results is covered, including elastic and plastic calculations of the shear stresses in the connectors. Final evaluation of the results yielded tentative design recommendations for both stud shear connectors and L-shaped connectors. Only the tentative design recommendations for stud shear connectors is included in the comparison of results.

Experimental Study

This study consisted of a simply supported composite beam structure that was subjected to fatigue loading, by applying a load at the center of the beam between the supports. The composite beam structure consisted of two 32 ft long 18WF50 steel beams with a 6 in. thick concrete slab 32 ft long and 10 ft 11 in. wide. The two I-shaped steel beams were connected at the ends and in the middle by a 16WF40 cross beam. The north beam had ¾ -in. diameter 4 in. tall studs with upset heads, while the south beam had ½ -in. L-shaped connectors. During testing, constant load was applied sinusoidally at a rate of 250 cycles per minute. Throughout the test program, the loading intervals were increased several times before a destruction test was performed under static loading. In addition, double shear tests were performed separately on the welded shear connectors, from which the strength of the weld was determined to be stronger than the base metal.

Results and Discussion

The beams did not fail in fatigue during the first 1.3 million cycles and were therefore tested in a static test until failure. During these tests, it was observed for the composite beam with the L-shaped connectors that the concrete slab would separate from the beam. The author points out that the upset heads of the ¾ -in. stud prevented this separation. An investigation of the shear induced in the shear connector was also undertaken. The elastic stress was based on the method of transformed sections, whereas at the ultimate load the stress computations were based on inelastic behavior of the steel and concrete. The author also points out that because frictional resistance between the steel beams and concrete slab was neglected, the shear forces in the stud shear connectors were higher.

Strains were not measured by strain gages around the flanges since this tends to be extremely difficult. Even if such measurements were taken, they would probably not be reliable measurements of the actual behavior of the stud shear connectors and would therefore not be appropriate to use for design recommendations. The design recommendations were therefore based on nominal stress computations for the average shearing stress in the studs, and any probable bond and friction were neglected.

Comparison of Results

By analysis and interpretation of the test results, the author presents a static shear capacity design equation for ½ -in. L-shaped connectors. In order to validate this tentative design equation, the author modified it for the ¾ -in. headed stud shear connectors before comparing it to the current AASHO specified design equation. The following formula represents the authors modified design equation for static shear capacity of the ¾ -in. headed stud shear connector:

$$Q_{uc} = 11{,}000 \cdot \sqrt{\frac{f'_c}{3{,}600}} \qquad \text{(B.6)}$$

where

Q_{uc} = static capacity of ¾ -in. stud shear connectors, lb

f'_c = compressive strength of concrete, psi

By comparison to the current AASHO Specification (1956, Article 3.9.5 – Shear), which is only intended for 4–in. tall straight studs with upset heads, the correspondence was very good. The recommended design equation by AASHO (1956) is the following:

$$Q_{uc} = 332 \cdot d^2 \cdot \sqrt{f'_c} \qquad \text{(B.7)}$$

where d is the diameter of the stud connectors in in.

The comparison of the new proposed design equation describing the static capacity for L-shaped connectors showed that the current AASHO design equation for straight studs with upset heads yielded a very conservative design when applied to L-shaped connectors. This experimentally and theoretically based investigation justified the AASHO design equation for stud shear connectors, since the results from the two design equations agreed well.

<u>Thürlimann, B., 1959</u> (p; m, c)

<u>Fatigue and Static Strength of Stud Shear Connectors</u>

Introduction

This paper follows on the work of Thürlimann (1958). In this paper, the author expands the research to include the fatigue behavior of stud shear connectors. The main goal of this paper was to include fatigue design along with his previously presented static design recommendations for L-shaped shear connectors. The experimental study included both straight headed stud shear and L-shaped connectors. The author's static design recommendations for L-shaped shear connectors are not included in the results and discussion.

Experimental Study, Results and Discussion

Push-out tests were conducted using specimens consisting of a 8 WF 40 beam with 6 in thick reinforced concrete slabs on each side. Two rows of 4 stud shear connectors with a height of 4 in were embedded in each slab.

In the static tests, the load was applied in increments while readings of the load and slip were taken. In both the static and fatigue tests, the load was applied to the center of the steel beam. In order to obtain a uniform distribution of the reactions from the floor, ½ in. thick plywood panels were placed below the concrete slabs.

In the fatigue tests, an axial load was applied sinusoidally to the wide flange beam. The load was applied at a frequency of 500 cycles per minute between the minimum and maximum load levels of 2 and 22 ksi, respectively until failure. In most of the fatigue tests, fatigue cracks initiated at the reinforcement of the stud weld and penetrated into the beam flange, causing a concave depression into the beam flange.

Throughout all the tests, measurements of the relative slip between the concrete slabs and metal flanges were taken. Strain measurements were not taken during these tests, because such measurements were difficult to make, and would probably not give reliable information about the behavior of the shear connectors. Even if strain gages would be used to instrument the shear connectors, it was recognized that local stress peaks around the connectors make it difficult to determine the capacity of connectors.

From these tests, several plots describing number of cycles vs. slip and average shear stress vs. slip were prepared to analyze the behavior and to compare the differences between L-shaped and headed stud shear connectors.

King, D. C., Slutter, R. G., and Driscoll, G. C. Jr., 1965 (b; m, c)

Fatigue Strength of ½-Inch Diameter Stud Shear Connectors

Introduction

To determine the fatigue strength of ½ in. headed stud shear connectors, twelve composite beams were constructed and tested under cyclic fatigue loading. A comparison is also made with previous tests on push-out specimens with ¾ in. stud shear connectors. A detailed description of how the connectors failed is also included.

Experimental Study

In order to develop a successful testing procedure, four identical composite beam specimens with a span length of 10 ft were initially created. These specimens were tested under both static and fatigue loading. In order to determine when a stud shear connector failed, strain gages were attached to the bottom of both the top and bottom flange at midspan. Additional gages were later added in the following test to better determine the initial failure of a stud shear connector. Dial gages located at midspan and at the ends of the steel section were used to measure deflection and slip.

For the primary test, eight identical composite beams were constructed. They consisted of a four foot wide by four inch thick concrete slab on a 12 WF 27 I-shaped steel section. The connection throughout the beam was made of two rows of ½ inch headed stud shear connectors.

The primary beam tests were all conducted similarly as simply supported members with a span length of 15 ft. The load was applied 9 in. on each side of the centerline at a frequency of 250 cycles per minute. The maximum load was determined from results of previous experiments. The minimum load was approximately 10 percent of the maximum load.

At regular intervals each fatigue test was stopped in order to perform static tests. These tests were performed by applying the load in increments until maximum load was reached. Readings from strain and dial gages were taken throughout all tests to determine stud shear connector failure. Visual inspection of the stud shear connectors was done at the end of each specimen. This was conducted by removing the concrete slab from the steel beam. This inspection was used as a final verification of which connectors actually had failed.

The horizontal shear stress on the connectors was calculated by the following elastic formula:

$$f_S = \frac{V \cdot Q \cdot S}{I_{tr} \cdot A_S} \tag{B.8}$$

where

V = applied shear force at the cross-section, kips

Q = first moment of the transformed concrete slab area, in^3

S = spacing of studs, in

I_{tr} = moment of inertia of the transformed composite section, in^4

A_s = cross-sectional area of stud shear connectors, in^2

Results and Discussion

From the information, obtained an S-N curve was derived for ½ in. headed stud shear connectors. The curve was created by a regression analysis of the data, where the following mathematical model was used:

$$\log N = A + B \cdot (S_{\max} - S_{\min}) \qquad \text{(B.9)}$$

where

S_{max} = maximum shear stress

S_{min} = minimum shear stress

A, B = empirical constants

N = number of cycles to failure

From this S-N curve, it was determined that the average shear stress for ½ inch stud shear connectors at 1,000,000 cycles of loading was 18.2 ksi. Failure of the composite members usually occurred in the base metal instead of in the shank of the stud shear connector.

A comparison between the results obtained from this testing program with previous push-out test data showed that the beam tests resulted in higher mean strength compared to the push-out tests. In addition, the push-out test data showed no significant difference between the results of ½ and ¾ in. stud shear connectors. In other words, the ½ and ¾ in stud shear connectors appeared to result in similar S-N curves.

Toprac, A. A., 1965 (b; c)

Fatigue Strength of ¾ Inch Stud Shear Connectors

Introduction

The main purpose of this paper was to investigate the overall effects of fatigue loading on composite beams, to what degree the number of stud shear connectors affects the fatigue strength of composite beams, and the effect defective stud welds have on the fatigue strength of beams. A comparison was also made with results of previous experiments at Lehigh University to determine the fatigue strength of ½ in. and ¾ in. stud shear connectors.

Experimental Study

This experiment consisted of specimens divided into group 1, consisting of four beams, and group 2, consisting of three beams. All the beams in group 1 were equal except for a different number of studs in each beam. The beams in group 2 were made as duplicates of group 1 with the exception of one beam. Group 2 beams were intentionally made to have faulty stud welds, and therefore were not satisfactory and acceptable for highway construction. Stud shear connectors were welded onto the beam flange in two rows throughout the entire beam in order to obtain a better distribution of the shear forces. The stud connectors were embedded into a 6-in. thick, 6-ft. wide reinforced concrete slab.

All of the test specimens were tested as simply supported beams with a span of 36 ft. They were loaded with two equal hydraulic jacks 14 ft from each support. The load was then applied dynamically and cycled between the minimum and maximum test loads, at a frequency of 180 cycles per minute. The average stud shear connector stress was kept between 1.4 and 20.3 ksi.

In order to determine when an individual stud failed, strain gages were attached on the underside of the top flange directly below the studs. The author pointed out that these strain readings only give qualitative results and may only be used to determine the effectiveness of each stud throughout the fatigue test and not for design purposes.

Shear forces induced on the stud shear connectors were calculated based on elastic analysis and the method of transformed sections. A total horizontal shear force was computed in each shear span and divided by the total area of the studs to give the average stud shear stress.

Results and Discussion

In order to investigate the relationship between stress range and cyclic life, S-N curves were created from the test results obtained in the beam tests. Comparisons were also made with results of previous experiments at Lehigh University to investigate whether ½ in. and ¾ in. stud connectors yield the same fatigue strength.

The comparison showed that there is a difference in fatigue strength between ½ in. and ¾ in. stud. The fatigue strength of ½ in. studs was higher throughout the cyclic life and was as much as 3 ksi higher in magnitude in the 1 to 3 million cycle range compared to the ¾ in. stud shear connectors. The fatigue life of beams with ½ in. stud connectors was therefore longer than beams with ¾ in. studs. The slopes of the S-N curves for ½ in. and ¾ in. stud connectors were approximately the same, and for ¾ in. stud connectors the stress varies from 20 to 13 ksi between 10,000 and 10 million cycles.

From this study it has been found that the American Association of State Highway Officials (AASHO) specifications were underestimating their design, and by reducing the factor of safety, the allowable stress for stud shear connectors could be increased.

From the investigation of defective stud welds, it was shown that two of the three defective specimens tested developed fatigue strengths approximately equal to the satisfactory specimens.

Slutter, R. G., and Fisher, J. W., 1966 (p; m, c, r)

Fatigue Strength of Shear Connectors

Introduction

The main purpose of this paper was to determine the relationship of stud shear connectors under cyclic load. Both fatigue and full stress reversal tests were conducted on push-out specimens. These specimens included both ¾ in, and 7/8 in. diameter 4-in. tall headed stud shear connectors and 4 in. tall 5.4 lb channel connectors. A detailed comparison of fatigue push-out test data with earlier beam test data is included. Finally, the authors present a tentative design equation for stud shear connectors in composite bridge beams subjected to cyclic load. Only the stud shear connectors are included in the summary.

Experimental Study

This experimental study involved a series of fatigue tests that were conducted on push-out specimens. Two types of connectors were used, namely ¾ -in. and 7/8 –in. stud shear connectors. The tests were conducted similarly on specimens with both types of shear connectors. The specimens chosen for this experiment differed from the conventional push-out specimen. Instead of using two reinforced concrete slabs attached to each side of an I-shaped steel beam, only one reinforced concrete slab was used. A sinusoidally varying load was applied concentrically through the reinforced concrete slab during the fatigue tests. Full stress reversal tests were also performed, where load was applied alternately to both edges of the concrete slab. Five different levels of maximum stress and three levels of minimum stress ranging from 10 to 26 ksi and –6 to 10 ksi, respectively, were chosen in the experiment. The stress levels were chosen based on previous beam experiments to be representative of the true conditions in a composite bridge structure. The average shear stresses were determined by dividing the applied load by the total nominal cross-sectional area of the studs.

The most common failure mode that occurred was failure initiated at the stud weld, which penetrated into the beam flange. In a few cases, when the weld penetration was incomplete, fracture through the weld occurred.

Results and Discussion

From this experimental study, it was found that the stress range was more important to the cyclic life of the specimens than maximum stress, and that the strength of the concrete had a minor effect on the cyclic life.

By comparison of the variance of the test data, it was determined that the minimum stress, -6 ksi, was significant to the cyclic life when stress reversal was included. Because stress reversal increased the cyclic life for the same stress range, it was considered conservative to omit this data for design recommendation.

The information from these fatigue tests yielded the following equation expressing the logarithm of the fatigue life as a linear function of stress range:

$$\log N = 8.072 - 0.1753 \cdot S_r \qquad \text{(B.10)}$$

where

N = number of cycles to failure

S_r = range of shear stress in ksi, $S_{max} - S_{min}$

This equation is conservative for stress reversal tests. However, because most connectors will only be subjected to shear loading in one direction, this is not considered critical.

By analysis of the push-out test data, it was determined that the behavior of the 7/8 –in. stud shear connectors was very similar to the ¾ -in. connectors. This implies that the fatigue strengths of the two studs are not significantly different, which allows for the same fatigue life equation to be used.

From a comparison between the push-out test data with data from previously conducted beam tests, it was determined that push-out tests are more conservative. In fact, the mean behavior of push-out tests corresponds to the lower bound of the dispersion for the beam tests, which represent a lower 95 percent confidence limit of these data. Even though beam tests represent a more accurate illustration of a true composite decking, the push-out test is widely used in tests of shear connectors because it is less expensive. On the basis of establishing the design strength of the shear connectors, push-out tests are suitable, because they provide an appropriate margin of safety.

Based on the test data, a tentative design formula for the allowable range of load was obtained from the equation describing the S-N curve, with the following result:

$$Z_r = \alpha \cdot d_s^2 \qquad \text{(B.11)}$$

where

Z_r = allowable range of shear force per stud, lb

d_s = diameter of the stud in inches

α = 13,800 for 100,000 cycles

10,600 for 500,000 cycles

7,850 for 2,000,000 cycles

This experimental study also showed that uniform spacing of connectors is satisfactory for most loading conditions since the shear load gets redistributed. Furthermore, placing shear connectors in the negative moment region should assist in maintaining flexural conformance throughout the continuous beam. Finally, it was observed that concrete strength did not significantly affect the fatigue strength of the stud shear connectors.

Mainstone, R. J., and Menzies, J. B., 1967 (p; m, c, r)

Shear connectors in steel-concrete composite beams for bridges

Part 1: Static and fatigue tests on push-out specimens

Introduction

The main objective of this paper was to investigate the behavior of shear connectors and to establish relationships between fatigue strengths of connectors subjected to different load ratios. Static and fatigue tests on push-out specimens with different types of shear connectors were performed. These included ¾ -in. diameter by 4 in. tall headed stud shear, channel, and bar connectors. Only the stud shear connectors are included in the summary.

Experimental Study

The push-out specimens consisted of having two reinforced 9 in. thick concrete slabs connected by one pair of shear connectors to each side of an I-shaped steel beam. CP 117 (British Standards Institution, 1967) recommends a reinforced concrete slab thickness of 6 in., but to avoid horizontal cracking of the slab during reversed testing, which might yield lower fatigue strength of the connectors, a thicker concrete slab was chosen.

A series of static tests was conducted on specimens with ¾ -in. stud shear connectors. The load was applied at the upper end of the steel beam in increments of 4 tons until cracking of the concrete occurred. Thereafter, the load was applied in increments of 2 tons until severe cracking of the slabs or fracture of connectors occurred. Dial gages where used to measure the vertical slip and the horizontal separation of the slabs relative to the steel beams.

The specimens tested with fatigue loading had the load applied dynamically either by jacks actuated by a pulsator or by a slow cycling machine, working at frequencies of 250 and 15 cycles per minute, respectively.

The specimens tested under reversed loading had to be slightly modified to account for the upward force that was obtained by using eight 2-ton springs. A thick steel plate was welded at the base of the steel beam section and a bearing plate was attached to the top of each concrete specimen. The springs were attached between the steel base plate and external steel frame that was connected to the bearing plates at the top of the specimens. Typical failure modes that occurred were either shear fracture through the weld or fracture of the base metal, causing a concave depression into the beam

Results and Discussion

The results from the static tests were plotted against the mean crushing strengths of concrete and compared to the recommendations tabulated in CP 117. This analysis showed that the static ultimate strength of stud shear connectors is approximately 10 percent lower than recommended. More experimental investigation was proposed to determine which results better describe the static ultimate strength of stud shear connectors.

The results from the fatigue tests were used to establish relationships between the maximum thrust on a connector and its life at different ratios of minimum to maximum thrust (i.e., load ratio). These results show that the slope of the lines are steeper the lower the load ratio gets, which results in shorter life cycles.

<u>Mainstone, R. J., and Menzies, J. B., 1967</u> (b; m, c, r)

<u>Shear connectors in steel-concrete composite beams for bridges</u>

<u>Part 2: Fatigue tests on beams</u>

Introduction

In a previously reported article the authors performed static and fatigue tests on push-out specimens to investigate the behavior of stud shear connectors and to establish relationships between fatigue strengths of stud connectors subjected to different load ratios. In order to validate these results obtained from the push-out tests, nine beam tests with the same types of connectors were conducted. The connectors used in these beams were ¾ in. diameter by 4 in. headed stud, channel, and bar connectors. Only the stud shear connectors are reported in the summary.

Experimental Study

Six beams were tested with ¾ in. diameter 4 in. headed stud shear connectors embedded in a 6-in. thick reinforced concrete slab. The beams were simply supported with a span of 16 ft.

The fatigue tests concentrated on three ratios of minimum to maximum applied shear, -1.0, 0.5, and 0.1. Each beam was loaded by two jacks placed at equal distance from the supports and operating in phase with one another at a frequency of 250 cycles per minute.

To account for reversed loading, the loads were applied by alternating between the jacks, which resulted in the intervening connectors being reversed completely. By altering the positioning of the two jacks and their cyclic ratio it made it possible to apply a wide range of shear ratios to the stud shear connectors. Cyclic loading continued until substantial failure had occurred in the connectors.

Strain gages and dial gages were attached to the specimens to measure the strain, slip between the steel flanges and the concrete slab, and deflection.

Results and Discussion

In order to determine how well push-out tests resemble beam tests, a comparison was done by plotting the maximum nominal shear stress on the stud shank or weld throat versus life cycle for different cyclic load ratios. By comparison, it was shown that the push-out tests agreed well with the beam tests and there was no evidence showing that push-out tests would underestimate the results.

Fisher, J. W., Frank, K. H., Hirt, M. A., and McNamee, B. M., 1970 (b; c, r)

Effect of Weldments on the Fatigue Strength of Steel Beams

Introduction

The principal objective of this research project was to derive design equations that would determine in general terms the fatigue resistance of rolled and welded beams, rolled and welded beams with cover plates, and welded beams with flange splices under fatigue loading.

This objective was achieved through an extensive review of existing fatigue data from previous experimental studies on beams with several different weld details and any previously derived mathematical relationships that defined the fatigue resistance of weld details. In addition, several experiments were developed and performed on steel beams to provide more information for this development.

Experimental Study

Before this research was conducted, only limited experimental data was available on the fatigue strength of steel beams with welded attachments. The general design relationship in the early 1970's AASHO Standard Specification for Highway Bridges was considered to be very approximate and needed to be improved to more accurately predict the life expectance of welded highway girder bridges.

In order to develop suitable mathematical relationships relating the fatigue behavior of the beams to design details, applied stresses, and type of steels, a series of experiments were developed. To provide a wide range of yield points from 36 to 100 ksi, three types of steel were incorporated in this study. These were A36, A441, and A514. This experimental study included fabrication and testing of 374 steel I-beams. Of these, 204 were constructed with cover plates at both flanges. Each cover plate was welded to each flange with longitudinal fillet welds. In addition, one transverse weld was placed at one end of each cover plate.

To determine the fatigue strength of the basic structural members without the cover-plate and flange splice details, 86 out of the total were plain rolled and plain welded beams.
This series of specimens had mainly two purposes. It expanded the existing test data for plain rolled and plain welded beams, and was used for comparison with the previous test data. The remaining 84 beams were constructed with butt welded flange splices.

All specimens were tested as simply supported beams with a span of 10 ft. Two point loads were applied at the center of the span with a distance between the loading points of 2 ft and 3 ft 6 in. During the fatigue tests, the loads were applied sinusoidally to every specimen at an operating speed between 260 and 800 cycles per minute. Even though varying operating speeds of the applied load were used, it had been shown by previous studies not to significantly affect the result of the fatigue behavior. This experimental study, however, only focused on constant magnitude cyclic loading.

In the fatigue tests with no stress reversal, the load was applied to the top of the I-beam. In order to get a constant moment region through the weld details, a spreader beam was used. In the stress reversal tests, when the minimum stress in the tension flange (i.e., bottom flange) was compression, the beam was loaded by a pair of jacks directly to the bottom flange. These jacks applied a constant load to the beam throughout the entire stress reversal tests.

Results and Discussion

For design purposes, this study showed that the fatigue strength of a given welded detail is independent of the strength of steel and that the stress range was the dominant stress parameter for all steels, beam types, and weld details tested. In addition, stress variables such as minimum stress, mean stress, and maximum stress were shown to be negligible for the fatigue life of welded attachments.

Previous experimental studies have shown that the strength and type of steel have an effect of the fatigue resistance for a particular detail. However, during this study it was determined that the variation of steel type was found to be negligible for A36 and A441 steel, since they both exhibited similar fatigue lives. Furthermore, the A514 welded beams did produce longer life than did their rolled counterpart. The slightly longer life of the welded beams was believed to be caused by the residual stresses that are introduced during the fabrication. The rolled beams were basically stress-relieved by the rotarizing process performed during fabrication. The welded beams had large compressive residual stresses, located at the edge of the cover plates, which slowed down the crack propagation and therefore yielded longer fatigue life. The variation in fatigue life due to the type of steel was considered to be negligible for design purposes of structures for fatigue.

An empirical exponential model relating stress range to cycle life was observed to provide the best fit to the test data for all beam series. This resulted in a log-log linear relationship between the stress range and the cycle life for each detail. This study determined the fatigue behavior of rolled and welded beams between 50,000 and 10 million cycles of loading.

During the testing it was shown that failures occurred mainly in the tension flange of all cover-plated beams initiated by cracks in regions of high tensile residual stresses. However it was discovered that many cover-plated specimens were observed to have cracks in the compression flanges. These cracks that formed in the compression flange of many cover-plated, plain welded, and flange splice beams were initiated in regions of high residual tension stress. These cracks usually grew more slowly after they had grown out of the residual tension region. However when these beams that had developed cracks in the compression region were subjected to tensile load, during stress reversal, several of them failed by fracture of the compression flange. These failures of the original compression flange sometimes occurred within the life of the original tension flange.

Menzies, J. B., 1971 (p; m, c)

CP 117 and Shear Connectors in Steel Concrete Composite Beams Made With Normal Density or Lightweight Concrete

Introduction

The main scope of this paper was to investigate the influence that concrete strength and density has on static and fatigue strength of connectors. Comparisons are made to previously reported results from push-out tests and from 49 new push-out tests. Detailed plots of maximum load per stud versus concrete cube strength and load-slip relationships describe the effects of varying each parameter. The connectors included in this investigation were ¾ in. headed stud shear, channel, and bar connectors. Only test results from specimens with stud shear connectors are included in the summary.

Experimental Study

All push-out specimens were constructed the same with the exception of the type mechanical shear connectors. The specimens had one 6 in. thick concrete slab on each side of a standard I-shaped steel section. One row of 2 stud shear connectors were embedded in each concrete slab.

A series of 15 specimens were tested in static push-out tests with varying concrete aggregates, such as normal-density, Lytag, and Leca concrete. The load was applied in increments until failure of the connectors. The maximum load range from approximately 5 to 12.70 tonf/connector

In addition, 10 specimens were tested in fatigue with the same concrete aggregates as in the static tests. The load was applied dynamically at a frequency of 4.17 cycles per second with a minimum and maximum fatigue load of approximately 0.5 and 5.0 tonf/ connector, respectively. Most of the failures that occurred were fracture of the welds.

Results and Discussion

The results showed that the static strength of stud shear connectors in normal density concrete are overestimated in the British Codes of Practice CP 117 specifications, and more reliable representations are given in the form of regression lines in the paper. Regression lines representing the static strength of stud shear connectors in lightweight concrete are also given in the paper.

The static and fatigue strength of stud shear connectors embedded in Leca concrete was approximately 60% compared to normal-density concrete. Due to much scatter in the test results for studs embedded in Leca concrete, the information was insufficient to base any design recommendations.

The static and fatigue strength of stud shear connectors embedded in Lytag concrete are also given by regression lines and are within 15 percent of the strength of studs embedded in normal-density concrete.

<u>Ollgaard, J. G., Slutter, R. G., and Fisher, J. W., 1971</u> (p; m)

<u>Shear Strength of Stud Connectors in Lightweight and Normal-Weight Concrete</u>

Introduction

The main scope of this paper was to determine the relationship of the shear strength of stud shear connectors embedded in normal and lightweight concrete. Two types of normal weight concrete and three types of lightweight concrete were used to get a wide variety of test results from 48 push-out tests.

Experimental Study

The specimens consisted of two reinforced 6-in. thick concrete slabs on each side of an I-shaped steel beam. Most of the specimens had two rows of two stud connectors, but a few specimens had a single row of two studs. Both 5/8 and ¾ in. stud shear connectors were used in the experiment.

In the static push-out tests, load was applied to the steel beam in increments up to the ultimate load, while slip readings between the steel flanges and the concrete slab were taken. The other tests were loaded to approximately the working load of the stud shear connectors and then unloaded, before being reloaded to their ultimate load capacity.

The two dominant failure modes that occurred during testing were failure in the shank of the stud connectors and failure in the beam flange below the connector, causing a concave depression in the beam flange. All the specimens with a single row of connectors failed by shearing off the studs.

Results and Discussion

The variables considered in the tests, so as to influence the shear strength of stud shear connectors, were the basic material properties of concrete and connectors, cross-sectional area of the stud, and the number of stud connectors:

$$Q_u = f\left(A, f'_c, f'_{sv}, E_c, w\right) \qquad \text{(B.12)}$$

where

Q_u = shear strength of stud shear connectors, kips

A = cross-sectional area of stud shear connector, in^2

f'_c = compressive strength of concrete, ksi

f'_{sv} = split tensile strength of concrete, ksi

E_c = modulus of elasticity of concrete, ksi

w = density of concrete, pcf

By comparison and analysis of the collected test results, multiple regression lines were constructed that represented the data for all different concrete types. From this analysis the following empirical equation representing the shear strength of stud shear connectors was derived:

$$Q_u = 1/2 \cdot A \cdot \sqrt{f'_c \cdot E_c} \qquad \text{(B.13)}$$

This equation describes the shear strength of stud shear connectors embedded in both normal-weight and lightweight concrete slabs well. In order to develop full capacity with this equation, they determined that the height-to-diameter ratio for stud connectors should be kept around 4 or higher.

Since shear strength of stud shear connectors in push-out tests tends to represent a lower bound to shear strength in beam tests, they indicated that there is already a factor of safety built into this equation. Based on experimental and theoretical studies, the authors recommend that this safety factor be about 2.

An empirical equation describing the load-slip relationship of stud shear connectors was also derived, based on the test results from the push-out specimens:

$$Q = Q_u \cdot \left(1 - e^{-18 \cdot \Delta}\right)^{2/5} \tag{B.14}$$

where

Q = applied load on shear connector, kips

Δ = average slip, in

The load-slip relationship for the reloading condition, which was determined in earlier experimental investigations, can be determined by the following equation:

$$Q = Q_u \cdot \left(\frac{80 \cdot \Delta}{1 + 80 \cdot \Delta}\right) \tag{B.15}$$

By comparison to the test data, this relationship gave very good approximations to the real load-slip behavior during reloading.

Fisher, J. W., Albrecht, P. A., Yen, B. T., Klingerman, D. J., and McNamee, B. M., 1974

(b; c, r)

Fatigue Strength of Steel Beams with Welded Stiffeners and Attachments

Introduction

The overall objective of this study was to derive design relationships to determine the fatigue strength of steel beams. For purpose of design, these equations considered the fatigue behavior of beams and design details, applied stresses, and types of steels.

This report, Phase II, is a continuation of Fisher et al. (1970), which constituted Phase I. In order to make the investigations of Phase II more comprehensive, this study was broadened to include more details that were not included in Phase I. The design relationship was intended to determine in general terms the fatigue strength of stiffeners and of lateral and transverse connections under constant-amplitude fatigue loading.

Experimental Study

In order to develop these mathematical relationships relating the fatigue resistance of stiffeners and attachments, a series of experiments were developed. Much information was gathered in a review of previous fatigue tests on rolled beams, welded beams, and girders with transverse stiffeners, and with lateral and transverse connections. This experimental study included fabrication and testing of 157 steel I-beams and girders. In the previous experiments reported in Phase I, it was shown that the type of steel did not significantly affect the fatigue life. Nevertheless both A441 and A514 were used to make direct comparison to the previous test results of Phase I.

To determine the fatigue resistance of high-strength steel members, 29 of the total 157 beams were plain rolled A514 steel beams. These also served as a basis of comparison to the previous studies of Phase I.

In order to make more comparison with the test results of Phase I, 106 plain welded beams similar in size to the beams of the previous study were constructed. These specimens accommodated several different locations of transverse stiffeners such as attached to the web alone or to the web and flanges, as well as including other flange attachment.

Finally, 22 larger welded girder specimens with transverse stiffeners on one side were constructed. These specimens were used to study the fatigue behavior of transverse stiffener details as well as the effects of stress parameter on fatigue crack growth.

All of the specimens were tested as simply supported beams with a span between 7 ft 6 in. and 10 ft. Two point loads were applied at the center of the span with a distance between the loading point of 1 ft. 6 in. to 5 ft. 6 in. During the fatigue tests, the loads were applied sinusoidally to every specimen at an operating speed between 200 and 800 cycles per minute similar to the tests of Phase I. The stress reversal tests were preloaded with two jacks to the bottom flange. These jacks applied constant load throughout the entire tests such that the minimum stress in the bottom flange was compression in a similar fashion to the Phase I tests. Testing of beams was once again limited to constant magnitude cyclic loading.

Results and Discussion

Similar to Phase I, it was observed for design purposes that stress range was the dominant stress variable for all stiffeners and attachment details, and that the type of steel had negligible effect on the fatigue strength.

The empirical exponential model relating stress range to cycle life was observed to provide the best fit to the test data for every detail. This resulted in a log-log linear relationship between the stress range and the cycle life for each detail.

Failure occurred mainly in the tension flange (i.e., bottom flange) of all beams with flange attachments. These failures usually originated in regions of high residual tensile stresses. Many cracks were also observed in the compression flange in regions of residual tension stress such as at the weld toe terminations. The crack causing failure of all attachment details originated at the most highly stressed weld toes of the welds connecting the attachment to the beam flange. More specifically cracks formed at the fillet weld toes connecting the stiffeners and attachments to the flange and web in the compression regions of the specimens. These cracks usually grew more slowly after they had grown out of the residual tension region. When these beams that had developed cracks in the compression region were subjected to tension, during stress reversal, several of them failed by fracture of the compression flange. These failures of the compression flange sometimes occurred within the life of the tension flange. However, none of the beams failed at a detail subjected to a nominal compression-compression stress cycle.

The final relationship of nominal stress range and the fatigue life provided for design purposes (i.e., S-N curves) were derived from the lower 95 percent confidence limits for 95 percent survival. This is performed with a regression analysis of the fatigue test data. By basing the S-N curves on the 95 percent confidence limit variations in the test results are accounted for as well as providing a uniform estimate of the survival. Only stress ranges that cause tensile or reversal stresses were included. The results from this experiment correlate with earlier studies in that cracks that form in the residual tension regions of specimen subjected to compression slow down and stop when they grow out of the residual tensile zone. However, when the specimens are subjected to stress reversal the applied tension is enough to prolong the crack propagation after it grows out of the residual tensile zone.

Pham, L., 1979 (a)

Design Strength of Stud Shear Connectors

Introduction

In this paper, a new procedure for designing stud shear connectors is presented. A large number of results from previous push-out tests were collected and analyzed in order to develop these new design equations. The method is based on a probabilistic approach for determining the design values for stud shear connectors.

Theoretical Discussion

In general, the strength of stud shear connectors, q_u, can be expressed as:

$$q_u = K \cdot \sqrt{f'_c} \cdot f_s / \sqrt{d} \qquad \text{(B.16)}$$

where

K = arbitrary constant

f'_c = compressive strength of concrete, Mpa

f_s = ultimate tensile strength of stud shear connector steel, Mpa

d = stud diameter, mm

The author determined that the characteristic 5 percentile value of the strength of stud shear connectors, $q_{u0.05}$, may be determined by:

$$q_{u0.05} = q_{umean} - 1.65 \cdot \sigma_{qu} \qquad \text{(B.17)}$$

where q_{umean} is the mean value of the strength of stud shear connectors and σ_{qu} is the standard deviation of q_u. σ_{qu} is related to the coefficient of variation of the failure stress of the stud connector in push-out test, V_{qu}, and q_{umean} by:

$$\sigma_{qu} = V_{qu} \cdot q_{umean} \tag{B.18}$$

V_{qu} depends on the coefficients of variation of K, f'_c, and f_s in the following way:

$$V_{qu}^2 = V_K^2 + \frac{1}{4} \cdot V_{f'c}^2 + V_{fs}^2 \tag{B.19}$$

where V_K can be taken as 0.157, $V_{f'c}$ = 0.119 to 0.164, depending on the characteristic strength of concrete (MPa) and V_{fs} = 0.060.

From earlier tests, it has been discovered that before failure in beam tests, the load among the shear connectors redistributes completely. Therefore, the load sharing factor, LSF, which is the ratio of 5 percentile values, is given by the following expression:

$$LSF = \frac{1 - 1.65 \cdot V_{qu} / \sqrt{N}}{1 - 1.65 \cdot V_{qu}} \tag{B.20}$$

where N is the number of studs on a beam between the sections of zero and maximum moment.

The final design load per stud can be computed from:

$$\text{Load per stud} = (LSF) \cdot q_{u0.05} \cdot \frac{\pi \cdot d^2}{4} \tag{B.21}$$

This design equation for stud shear connectors has been derived from previous experimental data and accounts for all major contributing factors that influence the behavior of stud shear connections in composite bridges. By comparison, it was found that this design procedure was more reliable than current design specifications.

Keating, P. B., and Fisher, J. W., 1986 (a)

Evaluation of Fatigue Tests and Design Criteria on Welded Details

Introduction

The objective of this research study was to more accurately determine new fatigue resistance curves and revise the fatigue provisions of the AASHTO Specification. This study includes an evaluation of the findings from previous fatigue studies, which are analyzed to confirm the validity of the present fatigue design recommendations.

Experimental Study

The early fatigue provisions of the 1970's AASHTO Specification were developed from the fatigue test result on beams with welded attachments, documented by Fisher et al. (1970, 1974). Since then many more fatigue studies have been conducted and collected into a database. This database is a collection of data from the original NCHRP test program, all following NCHRP test programs, as well as Japanese, Canadian, European, and other fatigue test data.

The intent was to make this database as comprehensive as possible by including all main fatigue test data performed on beam specimens with welded attachments. This database mainly contains fatigue test data from large-scale test specimens. The small-scale specimens were used only where there was no alternative fatigue data. The final database has been broadened to include a wider range of detail types and sizes of welds. The purpose of this database was to develop a design equation that would in general terms determine the fatigue resistance of welded attachments on beams.

From analysis of this broadened database, a new set of fatigue design curves was developed. These new curves predicted the fatigue resistance of welded bridge details more accurately. As a result of this review, the S-N curves were made parallel and the slopes were standardized at –3.0. The value of the slope was selected such that the AASHTO Specification (1986) would be consistent with many other international design rules, including the Eurocode.

As determined from previous experiments, the stress range was the main stress variable that determines the fatigue resistance of a steel member. From this fact a stress range-cycle life relationship was developed. Studies of test results showed that the relationship between the stress range and cycle life is log-log in nature, with approximately constant slope between each detail. In log form, the S-N curves are defined by:

$$\log N = \log A - B \cdot \log S_r \quad \text{(B.22)}$$

and in exponential form by:

$$N = A \cdot S_r^{-B} \quad \text{(B.23)}$$

where

N = number of cycles to failure
A = constant dependent on detail category
S_r = applied constant amplitude stress range
B = the inverse of the slope of the S-N curve

The allowable stress range values (i.e., S-N curves) were derived from the lower 95 percent confidence limits for 95 percent survival. This was performed with a regression analysis of the final database. By basing the S-N curves on the 95 percent confidence limit, variations in the test results were accounted for as well as providing a uniform estimate of the survival.

Results and Discussion

The original database, on which the early 1970's AASHTO fatigue provisions were based, included approximately 800 fatigue test failure results. This study included test results from several NCHRP reports as well as many others around the world, which resulted in a database with over 2300 fatigue test failure results covering many more weld details and sizes. This increased study allowed for a more accurate result, and hence provided a better estimate of the fatigue resistance of welded bridge details. By analysis of this more comprehensive database it was possible to determine more accurately the classification of each weld detail category. Base metal adjacent to stud shear connectors was also revised and determined appropriate as a Category C detail, as long as the diameter is less than two inches.

Oehlers, D. J., and Johnson, R. P., 1987 (p; m)

The Strength of Stud Shear Connections in Composite Beams

Introduction

A study of stud shear connectors in composite beams was undertaken by gathering results from 110 push-out tests. In order to determine the differences between strengths of stud shear connectors in beams compared to in push-out specimens, eight additional push-out tests were performed in which axial tensile and compressive forces were applied to the connections. A thorough analysis of this information was done in order to derive an equation for the strength of stud shear connectors in beams.

Experimental Study

A series of eight new push-out specimens were constructed. They differed from the conventional push-out specimens by having knife-edge supports below the specimens. In order to eliminate any frictional forces, these knife-edge supports were placed on steel plates with roller bearings resting on the ground. By changing the lateral displacement of the knife-edge supports the direction of the axial force in the stud shear connectors could be changed and even eliminated. All of the specimens were constructed with one stud shear connector embedded in each concrete slab. Load was applied to the top of the I-shaped steel section through a ball joint to eliminate any frictional forces at the top of the specimen. The applied load was uniformly distributed through a spreader beam located at the top of the I-shaped steel section.

Results and Discussion

The authors indicate that, in general, the shear strength of a stud shear connector, P_p, is a function of the cross-sectional area of the connector, A, modulus of the concrete and the steel, E_c and E_s respectively, cube strength of concrete, f_{cu}, and tensile strength of stud connector, f_u:

$$P_p = f\left(A, E_c, E_s, f_{cu}, f_u\right) \qquad \text{(B.24)}$$

In previous experimental tests it was discovered that the difference in height of different commercial studs did not affect their overall shear strength, and may therefore be excluded as a variable.

Based on numerous experimental and theoretical studies, it was previously found that the shear strength of stud shear connectors in push-out tests may be computed as:

$$P_p = 5.0 \cdot A \cdot f_u \cdot \left(\frac{E_c}{E_s}\right)^{0.40} \cdot \left(\frac{f_{cu}}{f_u}\right)^{0.35} \qquad \text{(B.25)}$$

Due to differences of the normal forces across the interface of the steel-flange and the concrete-slab, in composite beams and in push-out specimens, the strength of stud shear connectors in beams are lower. From additional push-out tests in which axial tensile and compressive forces were applied to the specimen, it was determined that the strength of stud shear connectors in composite beams was approximately 81 percent of the shear strength of standard stud shear connectors in push-out tests. The following equation gives the shear strength of stud shear connectors in beams:

$$P_p = K \cdot A \cdot \left(\frac{E_c}{E_s}\right)^{0.40} \cdot f_{cu}^{\,0.35} \cdot f_u^{\,0.65} \tag{B.26}$$

where K is the 5 percent characteristic strength expressed in terms of the number of studs subjected to similar displacements:

$$K = 4.1 - n^{-1/2} \tag{B.27}$$

This equation represents the lower 90 percent confidence limit (i.e., the characteristic strength of stud shear connectors). By comparison to other previously reported tests, it was verified that these two equations predict the shear strength of stud shear connectors adequately.

Oehlers, D. J., 1990a (p; m, c)

Deterioration in Strength of Stud Connectors in Composite Bridge Beams

Introduction

In this paper the author discusses the deterioration in strength of composite beams due to fatigue load. He also presents a new design technique that better represents the actual behavior of stud shear connectors compared to the ones that are currently in use. This design technique incorporates the use of a newly derived equation, which determines the endurance strength of stud shear connectors, with three other design equations that have been derived in previous literature.

Experimental Study

A series of 14 identical push-out specimens were constructed. The specimens consisted of stud connectors welded to a steel plate, which had been waxed to reduce bond or friction between the steel and concrete slab. The stud connectors were placed in two rows with two connectors in each to allow for more uniform redistribution of the shear loads. The plate was oriented horizontally during the casting and curing of the concrete. The plate was finally bolted to an I-shaped steel section to form the final specimens.

Three types of tests were conducted. The first one was a static strength test where three of the specimens were tested in order to determine the static strength, P_s, of the manufactured specimens. The specimens were loaded under displacement control, where the load was increased in increments until failure of the connectors occurred.

In the second test, six of the remaining specimens were tested in a fatigue endurance test. This test consisted of applying a constant range of cyclic load of 0.25 P_s to each push-out specimen. The specimens were tested until the monotonic strength of the shear connectors reduced to the peak of the applied cyclic load, which resulted in rapid failure of the connectors.

In the last test series, the remaining five specimens were tested in a monotonic strength test. In these tests, a constant range of cyclic load of 0.25 P_s was applied at a constant peak load of 0.29 P_s, followed by a static strength test as in the first test.

Typical failure modes that occurred in the last two tests where fracture at the interface of the weld collar and the shank, fracture of the weld collar and the flange, causing a concave depression into the beam flange, and fracture through the stud shank above the weld collar.

Results and Discussion

In order to establish a relationship between the static strength per stud as a function of number of cyclic loads, a plot was created where all the test results were included. From extensive analysis of these test results, the following linear regression line was determined to represent the fatigue behavior of stud shear connectors:

$$N_e = N_f \cdot \left(1 - \frac{P_m}{P_s}\right) \tag{B.28}$$

where

N_e = number of cycles to cause P_s to reduce to P_m

N_f = theoretical fatigue life, number of cycles

P_m = static strength after N_e cycles, kN

P_s = original static strength, kN

By incorporating three other design equations, which were derived in previous literature and presented below, it was possible to account for reduction in the static strength due to fatigue loads in the design.

The author derived the following design equation using a similar procedure developed by Johnson and Buckby (1986) to determine the distribution of the static strengths of the stud shear connectors, P_s, throughout the length of a bridge:

$$\frac{R_1}{P_s} = \left[\frac{10^K \cdot \left(1 - \frac{P_m}{P_s}\right)}{C \cdot n_t + C \cdot n_t \cdot \left(\frac{R_2}{R_1}\right)^m + \ldots + C \cdot n_t \cdot \left(\frac{R_t}{R_1}\right)^m} \right]^{\frac{1}{m}} \tag{B.29}$$

where

R = range of load on a shear connector induced by the standard fatigue vehicle, kN

R_t = total range of load on a shear connector induced by the standard fatigue vehicle, kN

P_m = minimum static strength required at the end of the fatigue life, kN

P_s = static strength of shear connection when structure is built, kN

K = constant of fatigue endurance equation

m = constant of fatigue endurance equation

C = spectral constant that is a function of the frequency and weight of the vehicles

n_t = number of loading events to which the structure is subjected

This equation can be used either for design or for analysis purposes in an iterative process to determine P_m and P_s. The range of load, R, represents the wheel load imposed on a bridge where R_1 and R_2 represent the positive and negative shear forces throughout the span.

From Oehlers (1990), the following equation determines the fatigue life of a connector:

$$N_f = 10^K \cdot \left(\frac{R}{P_s}\right)^{-5.1} \tag{B.30}$$

and

$$K = 2.68 - \frac{0.704}{\sqrt{n}} \tag{B.31}$$

where

n = number of connectors

From Johnson (1987), the following equation determines the static strength, kN:

$$P_S = k \cdot f_u \cdot A \cdot \left(\frac{E_c}{E_s}\right)^{0.40} \cdot \left(\frac{f_{cu}}{f_u}\right)^{0.35} \tag{B.32}$$

and

$$k = 4.1 - \frac{1.0}{\sqrt{n}} \tag{B.33}$$

where

f_u = tensile strength of the stud material

A = cross-sectional area of the shank of the stud, mm^2

E_c = Young's modulus for the concrete, kN/mm^2

E_s = Young's modulus for the stud material ≈206 kN/mm^2

f_{cu} = cube strength of the concrete ≈1.25*fc

<u>Fisher, J. W., et al, 1993</u> (b; c, r)

<u>Development of Advanced Double Hull Concepts</u>

Introduction

The main objectives in this report were to compare the fatigue strength of longitudinal fillet welds, welded attachments, and transverse butt welds in HSLA-80 steel to the existing database on fatigue results. Only studies with longitudinal fillet welds are included in this summary.

Experimental Stud

These objectives were accomplished through a large testing program, which included fatigue tests on over 170 full-scale welded HSLA-80 steel I-section beams with typical weld details. By repairing these specimens after cracking had occurred, each specimen could be forced to yield more than one crack. At the end of the testing program over 240 data points had been collected from these specimens.

The I-beam specimens were tested as simply supported members with a 10 ft span. Two loading points 4 ft. apart were centered on the span. By implementing two equally centered point loads, a constant moment region on the large beam specimens was obtained, enabling the weld details to be subjected to a constant stress range. The fatigue setup used a spreader beam that was clamped around the specimen to allow for reversed loading. The load was applied sinusoidally to the spreader beam by a hydraulic jack at an operating speed of 2 to 4 cycles per second.

The main control variables in these experiments were the stress range and minimum stress. Many of the combinations of stress ranges and minimum stress values were chosen so that the results could be compared to previous experiments conducted under similar conditions.

Results and Discussion

Several histograms were created, including both Gaussian and Weibull distributions, to fit to the data to facilitate probabilistic analysis from all the test results. By a comparison of the statistical distribution of occurrence of cracking of longitudinal fillet welds for all stress ranges, for negative load ratio only (i.e., reversal), and positive load ratio only (i.e., tension), it was discovered that the mean and the standard deviation were greater for the reversal data than compared to the tension only data in all tests for the Gaussian distribution. Based on an analysis of these test results, it was determined that the means and the standard deviation of the distributions were dependent on the stress range, since they were not all the same.

Inspection of the 95 percent confidence limit (i.e., mean stress minus two standard deviations) of the S-N curves showed that there was no significant difference between the tension only and the stress reversal data. Similar to previous studies, it was noted that cracks typically initiated in local regions of high residual tensile stresses, such as the tension flange of an I-beam. It was also noticed that cracks sometimes formed in the compression flange during reversal tests. These cracks also initiated at locations of high residual tensile stresses, because the sum of the residual tensile stress and the applied stress was almost always tensile. Therefore, there was basically no difference between tension only and stress reversal tests except from that stress reversal specimen had longer lives than tension only specimens. The reason for this is that the compression component, throughout half of the cyclic loading, decreases the tensile stresses in the localized regions of high residual stresses and therefore reduces the rate at which the fatigue cracks are growing.

<u>Roberts, T. M., and Dogan, O., 1997</u> (p; c)

<u>Fatigue of Welded Stud Shear Connectors in Steel-Concrete-Steel Sandwich Beams</u>

Introduction

The main purpose of this paper was to investigate the fatigue behavior of tension plate stud connectors in steel-concrete-steel sandwich (SCSS) beams. Both static and fatigue tests were conducted on both push-out specimens and beam specimens. A comparison was made between beam and push-out fatigue test results with the Eurocode 3 detail category 80 fatigue strength curve. A detailed analysis describing determination of stud shear forces from measured tension plate strains and from theoretical predictions was included.

Experimental Study

A series of conventional push-out specimens were constructed consisting of an I-shaped steel beam with reinforced concrete slabs on each side. Two rows with 4 stud shear connectors were embedded in each slab.

In the static test, the load was applied in increments while, readings of the load and slip were taken. In both the static and fatigue tests, the load was applied to the center of the steel beam. In order to obtain a uniform distribution of the reactions from the floor, wood panels were placed below the concrete slabs.

In the fatigue tests, a hydraulic jack was used to apply an axial load sinusoidally to the steel beam. The load was applied at a frequency of five cycles per second between the minimum and maximum load levels until failure. A predetermined loading range was selected for each push-out specimen. The loading ranges varied from 40 to 110 kN.

Nine identical SCSS specimens were constructed. They consisted of two layers of relatively thin steel plates on each side of a concrete slab. The connections in the specimens were made of stud shear connectors welded onto each steel plate in rows of two and embedded into the concrete. In order to prevent bond and friction between the steel and concrete, a layer of mould oil was placed on the steel plates. Strain gages were placed on both the inside and outside of the steel tension plate to measure axial strains.

A series of the SCSS beams were subjected to fatigue tests. The beams were simply supported during the tests, and the load was applied at the center in increments, under deflection control, up to the maximum load. The load was then reduced to the minimum load, before load was dynamically applied between minimum and maximum at a frequency between one and six cycles per second. Different loading ranges were predetermined for each beam specimen, which varied from 30 to 70 within different intervals.

Results and Discussion

Shear forces in the tension plate stud connectors were computed assuming fully composite action and a linear variation of strain throughout the SCSS section. A comparison was made between the theoretical stud shear forces and experimentally determined shear forces from the measured tension plate strains. The comparison showed that experimentally determined shear forces were less than the theoretical forces. This difference was mostly due to neglect of bond and frictional forces between the steel plates and concrete when theoretically determining the forces.

Common failure modes that occurred were fracture through the weld and through the shank of the stud connectors.

Eurocode 3 recommends that welded stud connectors be based on the detail category 80 fatigue strength curve for the shear stresses, defined by the equation

$$\log N = 15.801 - 5 \cdot \log \tau_r \qquad \text{(B.34)}$$

where

N = number of load or stress cycles to fatigue failure

τ_r = shear stress range in stud connectors

A comparison was made between the test results obtained from the beam and push-out tests, experimental beam studies conducted earlier, and with the fatigue strength curve recommended in the Eurocode 3. This analysis showed that the fatigue strength curve and the push-out tests were considerably conservative, compared to the beam tests. Also the push-out test seemed to correlate well with the fatigue strength curve.

APPENDIX C

TABLES OF EXPERIMENTAL DATA

APPENDIX C

TABLES OF EXPERIMENTAL DATA

This appendix includes all the significant experimental data that is presented in the studies summarized in Appendix B. This appendix is divided into two categories: Push-Out Tests and Beam Tests.

TABLES OF PUSH-OUT TESTS

Table C.1. Push-Out Tests

Article	Experiment Synopsis	Number of Tests	Dimension of Concrete Slab (in)	Beam Size	Number of Concrete Slabs
Viest, ‘56	Monotonic loading	12	24 x 30 x 7	8 WF 48	2
Fountain and Sinclair, ‘56	Fatigue and reversed loading	14	N. A.	N. A.	N. A.
Thürlimann, ‘59	Monotonic and fatigue loading	10	20 x 28 x 6	8 WF 40	2
Slutter and Fisher, ‘66	Fatigue and reversed loading	35-3/4 studs 9-7/8 studs	20 x 26 ¾ x 6	8 WF 40	1
Mainstone and Menzies, ‘67	Monotonic, fatigue, and reversed loading	34	12 x 18 x 6 and 12 x 18 x 9	12 in. x 5 in. x 32lb	2
Menzies, ‘71	Monotonic and fatigue loading	25	305 x 457 x 229 (mm)	305 mm x 127 mm x 47.7 kg/m	2
Ollgaard, Slutter, and Fisher, ’71	Monotonic loading	48	18 x 28 x 6	W8 x 40	2
Oehlers and Johnson, ‘87	Monotonic loading	8	N. S.	254 x 146 x 43 UB	2
Oehlers, ‘90	Monotonic and fatigue loading	14	900 x 650 x 130 (mm)	N. S.	2
Roberts and Dogan, ‘97	Fatigue loading	6	600 x 600 x 175 (mm)	200 x 100 x 8 RHS (mm)	2

Beam Material	**f'$_c$** (psi)	**Reinforcement**	**Stud Diameter** (in)	**Stud Height** (in)	**Arrangement of Studs** (Rows/Columns)
N. S.	3060-4720	4 x 4-10/10 (wire mesh)	½-1 ¼	≈4	2 x 2 and 2 x 4
N. A.	N. A.	N. A.	¾	3	1 x 1
A7	2650-6200	#4 bars	¾ ½ L-shaped	4 2 ¼	2 x 2
ASTM A36	3320 and 4300	#4 bars	¾ and 7/8	4	2 x 2
N. S.	≈5600 (Water Stored Cube Strength)	#3 bars	¾	4	1 x 2
N. S.	2210 1590 and 1230 kg/m^3	9.5 mm bars	19 mm	102 mm	1 x 2
N. S.	2067-5080	#4 and #5 bars	¾ and 5/8	4	1 x 2 and 2 x 2
N. S.	N. S.	N. S.	13 mm	65 mm	1 x 1
N. S.	47.1-49.5 N/mm^2	Yes	13 mm	75 mm	2 x 2
N. S.	24.1-37.9 N/mm^2 (Cube Strength)	10 mm diameter	10 mm	150 mm	2 x 2

End Constraints	**Application of Load**	**Loading Rate** (cycles per minute)	**Loading Range** (kips)	**Stress Range of Studs** (ksi)
Placed in plaster of paris	Concentrically through steel section	N. A.	2-10	N. S.
N. A.	Perpendicular onto stud	600	N. S.	Specified as alternating stress
Placed on 0.5-in. thick plywood panel	Concentrically through steel section	500	N. S.	Varied between ≈2-22
No constraints	Concentrically through concrete slab	250 and 500	N. S.	Varied between -6-26
No constraints	Concentrically through steel section	15 and 250	Varied between 0.1-9.25 (tons per connector)	N. S.
N. S.	Concentrically through steel section	250	Monotonic: Max load ≈5-13 Fatigue: 0.4-4.0 and 0.5-5.0 tonf/ connector	N. S.
Placed on sheets of 0.5-in. homosotc	Concentrically through steel section	N. A.	N. S.	N. S.
No constraints and placed on movable knife-edge supports	Concentrically through steel section	N. A.	N. S.	N. S.
Restrained at the base	Concentrically through steel section	N. S.	≈13 kN	N. S.
Placed on soft wooden boards	Concentrically through steel section	300	Varied between 55-155 kN	N. S.

Results Reported	**Main Test Parameters, Comments**
- Load slip curves	- Parameters: Stress range, f'_c, s, and d
- Stress vs. strain - S vs. N curve	- Parameter: Stress range - Concrete was not present in the tests
- N vs. slip - Average shear stress vs. slip - Fatigue design equation	- Parameters: Stress range and stud shapes - Detailed analysis of cycles vs. slip
- S vs. N curves - Comparison: stud diameter sizes - Comparison: b vs. p - Fatigue design equation	- Parameter: Stress range - Detailed analysis of S-N curves
- Load ratio and S vs. N curves	- Parameter: Load ratio - Detailed analysis of S-N curves, and failures
- Max load vs. cube strength - Load vs. slip - S vs. N	- Parameters: Stress range, w, and f'_c - Detailed analysis of the effect concrete has on the monotonic strength of stud shear connector
- Static design equation	- Parameters: A, f'_c, f'_{sv}, E_c, and w - Derivation of the AASHTO LRDF static design equation
- Load slip curves - Load vs. f_{cu}	- Parameter: Stress range - Good analysis of direction and magnitude of axial forces through the studs
- S vs. N curves - Fatigue design equations	- Parameters: Constant load range and peak load
- S vs. N curve - Comparison: b vs. p and the Eurocode 3	- Parameter: Stress range - Good comparison of test results of b and p

TABLES OF BEAM TESTS

Table C.2. Beam Tests

Article	**Experiment Synopsis**	**Number of Tests**	**Dimension of Concrete Slab** (in)	**Beam Size**	**Span Length** (ft)
Thürlimann, ‘58	Monotonic and fatigue loading	1	131 x 6	18 WF 50	30
Toprac, ‘65	Fatigue loading	7	72 x 6	24 WF 68	36
King, Slutter, and Driscoll, ‘65	Monotonic and fatigue loading	8	48 x 4	12 WF 27	15
Mainstone and Menzies, ‘67	Monotonic, fatigue, and reversed loading	9	24 x 6	12 in x 6 in x 4.4 lb. RSJ	16
Fisher et al., ‘70	Fatigue and reversed loading	374	N. A.	14 W 30 and similar sizes	10
Fisher et al., ‘74	Fatigue and reversed loading	157	N. A.	W10 x 25, and W14 x 30	7.5-10
Keating and Fisher, ‘86	Fatigue and reversed loading	≈2300	N. A.	Varied	Varied
Fisher et al., ‘93	Fatigue and reversed loading [1]	170	N. A.	N. S.	10
Roberts and Dogan, ‘97	Monotonic and fatigue loading	9	200 x 150 mm (SCSS-Beam)	200 x 8 mm	1400 mm

[1] This NCHRP Report is a summary of other experimental studies.

Beam Material	**f'$_c$** (psi)	**Reinforcement**	**Stud Diameter** (in)	**Stud Height** (in)	**Number of Studs per Row**
A7	2150-4240	#4 and #5	¾ ½ L-shaped	4 1 7/8	2 3
A36	4150-5730	#3 and #4 bars	¾	4	2
ASTM A7	3030-4060	#4 bars	½	2–2 1/8	2
N. S.	5900-6910 (Water Stored Cube Strength)	#5 bars	¾	4	2
A36, A441, and A514	N. A.	N. A.	N. A.	N. A.	N. A.
A441, and A514	N. A.	N. A.	N. A.	N. A.	N. A.
Varied	N. A.	N. A.	N. A.	N. A.	N. A.
Min. specified yield strength: 80-90 ksi	N. A.	N. A.	N. A.	N. A.	N. A.
N. S.	22.2-37.6 N/mm^2 (Cube Strength)	None	10 mm	150 mm	2

Total Number of Studs	**Application of Load**	**Loading Rate** (cycles per minute)	**Load Range** (kips)	**Stress Range of Studs** (ksi)
30 (3/4) and 36 (1/2) per half span	Center span	250	Varied between 3.7-50.7	N. S.
54-90	14 ft from each end	180	4-33 and 5.2-51	Varied between 1.4-20.3
40	9 in. on each side of the centerline	250	Varied between ≈1.2-14.5	Varied between ≈2-32
48	Varied	250	N. S.	Specified as nominal maximum shear stress on weld
N. A.	Fatigue: Two point loads at the top Stress reversal: Added two jacks at the bottom	260-800	N. S.	Varied between ≈-10-32 (note: these beams did not include any studs)
N. A.	Fatigue: Two point loads at the top Stress reversal: Added two jacks at the bottom	200-800	N. S.	Varied between ≈-22-32 (note: these beams did not include any studs)
N. A.	Varied	N. S.	N. S.	≈-20-35 (note: these beams did not include any studs)
N. A.	Spreader beam that was clamped around the beam with two point loads	120-240	N. S.	≈-22-20 (note: these beams did not include any studs)
38	Concentrically	60–360	Varied between 15-85 kN	N. S.

Results Reported	**Main Test Parameters, Comments**
- Load vs. Deflection - Load and Stress vs. Slip - Design Equations	- Parameter: Stress range -Detailed inelastic and plastic calculations -
- *S* vs. *N* curves - Comparison: stud diameter sizes - Comparison: *b* vs. *p*	- Parameters: Stress range, number of studs, and quality of weld - Thorough analysis of beam test
- *S* vs. *N* curves - Comparison: stud diameter sizes - Comparison: *b* vs. *p*	- Parameter: Stress range - Detailed analysis about instrumentation
- *S* vs. *N*	- Parameter: Load ratio - Application of load
- *S* vs. *N* curves - Detailed study of negative moment behavior	- Parameters: Stress range, steel type, and beams - Included several different attachments instead of studs
- *S* vs. *N* curves - Detailed study of negative moment behavior	- Parameters: Stress range, steel type, and beams - Included several different attachments instead of studs
- *S* vs. *N* curves - Proposed AASHTO fatigue design curves	- Parameters: Stress range, steel type,beams, and loading conditions - Included several different attachments
- *S* vs. *N* curves - Detailed study of negative moment behavior	- Parameters: Stress range and minimum stress - Included several different attachments
- *S* vs. *N* curve - Comparison: *b* vs. *p* and the Eurocode 3	- Parameter: Stress range - Good comparison of test results of *b* and *p*

APPENDIX D

SUPPLEMENTAL BIBLIOGRAPHY

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