STEEL CAPACITY OF HEADED STUDS LOADED IN SHEAR

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Abstract

The Precast/Prestressed Concrete Institute (PCI) sponsored a comprehensive research program to assess the shear capacity of headed stud group anchorages. This program was initiated in response to new provisions introduced into the ACI 318 Building Code. These new provisions are based on an extensive experimental database consisting mostly of post-installed anchor tests. Tests of headed stud anchorage groups loaded in shear, as used in precast construction, are not extensively reported in the literature.

The test program, conducted by Wiss, Janney, Elstner Associates, Inc. (WJE), examined headed stud connections loaded toward a free edge (de3), loaded toward a free edge (de3) near a corner, loaded parallel to one free edge (de1), loaded parallel to two free edges (de1 and de2), loaded away from a free edge (de4), and *in-the-field* of a member, such that edge distance was not a factor. The information reported herein addresses one aspect of the overall test program, the steel capacity failure mode.

1. Introduction

Headed stud anchorages are used throughout the concrete industry in both cast-in-place and precast construction. Welding studs to steel plates provides an economical structural connection by allowing larger variability in construction dimensions and tolerances. Commonly, studs in precast members are 75 to 200 mm long and found almost always in multi-stud group connections. The load capacities of these connection types are affected by stud spacings, edge distances, and member depth or thickness. This research work[1] focused on anchorages and geometric conditions typically used in precast / prestressed members. The research concentrated on diameter, embedment depth, and number of welded headed studs on connection plate configurations commonly used in precast applications; the study excluded post-installed anchors.

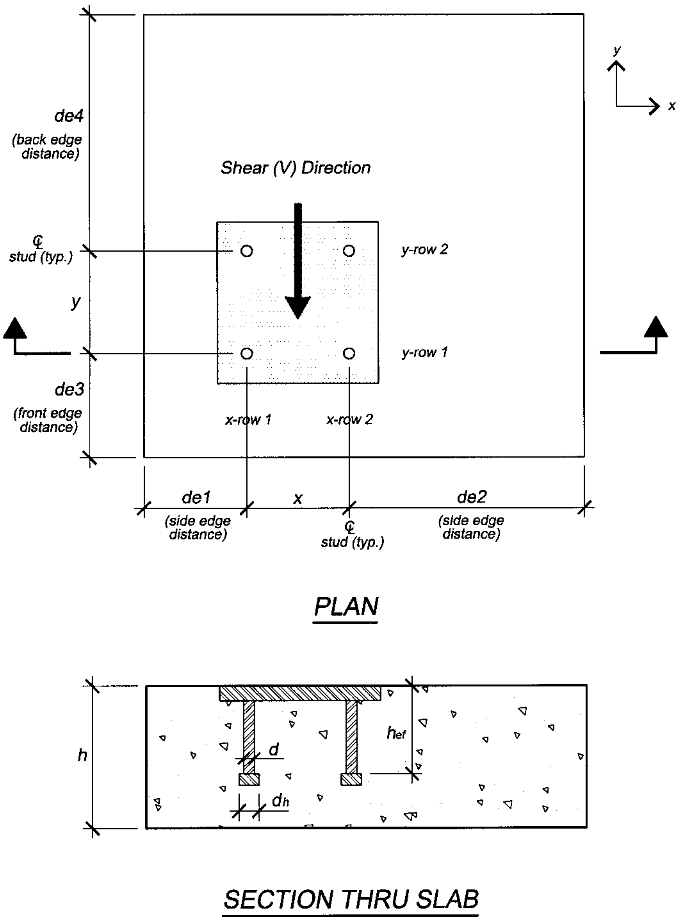

Figure 1: PCI notation for anchorage geometry.[1]

In the United States, headed stud anchorage design usually followed procedures set forth in the PCI Handbook [2] or the nuclear structures code of ACI Committee 349.[3] The Concrete Capacity Design (CCD) approach for anchorage to concrete has recently been approved as Chapter 23 of the upcoming 2002 version of the ACI 318 Building Code.[4, 5]

The work reported herein summarizes stud anchorage behavior when the connection is loaded in shear away from a free edge and *in-the-field*. These two conditions cause the ultimate capacity to be dictated by the stud steel. Referring to Figure 1, the overall research program tested anchorages toward, parallel, and away from a free edge. Several test series were repeated in both 152 and 406-mm thick specimens to evaluate member thickness effects. This paper is limited to defining the stud steel capacity in shear.

2. Literature Review

2.1 Push-off testing

The welded, headed stud received research attention in the late 1950s and through the 1960s in concrete slab-steel beam composite construction.[6-13] Testing to evaluate composite beam behavior utilized a push-off specimen, consisting of a wide flange beam section sandwiched between two concrete slabs. Headed studs were welded to both flanges of the beam in some prescribed spacing pattern and embedded into a thin concrete slab, representing the composite deck slab.

Early push-off test results provide the design basis for headed stud groups loaded in pure shear. Push-off test failures were sometimes due to stud steel shear. The push-off specimen having one transverse stud row (one y-row) is viewed to be analogous to a headed stud anchorage located *in-the-field* of a member, away from all edge influences, and is relevant to this paper. When stud groups with multiple longitudinal rows were tested using the push-off specimen, the test results become more difficult to interpret because large y-spacings reduce anchor group efficiency due to shear lag effects; these tests were thus excluded from our analysis. Significant findings are summarized below.

2.2 Embedment depth and steel capacity

The reviewed data indicates $1.0A_sF_{ut}$ (see *Eq. (1)* notation) is a good predictor for a steel failure when the effective embedment depth / stud diameter (h_{ef}/d) exceeds about 4.5. This is slightly greater than the value of 4.2 identified by Driscoll and Slutter.[12] A value reduced for tensile yield ($F_y = 0.9\ F_{ut}$), where F_y is the offset tensile yield stress, is not as good, although more conservative. Likewise, A_sF_{ut} is a much better capacity predictor than using shear yield ($F_{vy} = F_{ut}\ /\sqrt{3}$), where F_{vy} is the shear yield stress per the Huber-von Mises-Hencky yield criteria.

Work performed by Ollgaard, Slutter, and Fisher[13] at Lehigh University produced a longstanding prediction equation, independent of failure mode, basing individual stud strength on stud area, concrete compressive strength, and elastic modulus of the concrete. Studs with an h_{ef}/d of 3.26 and different types of lightweight and normal-weight concrete were used. Failures were noted in both stud steel shear or by a concrete mechanism. Their final prediction equation used in composite beam design was:

$$Q_u = 0.5A_s\sqrt{f'_c\,E_c} \ \leq\ A_sF_{ut} \tag{1}$$

where:

Q_u = Nominal shear stud connector strength embedded in a solid concrete slab (N)
A_s = Effective cross-sectional area of a stud anchor (mm^2)
f'_c = Cylinder compressive strength of concrete (MPa) [= 0.8 x cube strength (f_{cc})]
E_c = Modulus of elasticity of concrete (MPa)
F_{ut} = Ultimate tensile strength of the stud steel (MPa)

When headed studs have $h_{ef}/d < 4.5$, a concrete pryout failure mechanism can occur. Pryout failure is a concrete breakout failure mode not associated with edge distance but a function of the headed stud "stiffness." *Eq. (1)* predicts this failure mode well.

2.3 Lightweight aggregate concrete

Our analysis of reported steel shear failures for headed studs embedded in lightweight concrete indicates test strengths less than a $1.0A_sF_{ut}$ prediction. Lightweight aggregate concrete apparently provides an embedment environment whereby the stud induces greater concrete crushing, producing more stud bending deformation resulting in larger overall relative slip between the stud and concrete. The increased concrete deformation produces more bending in the stud and attachment weld, thereby making the failure mode appear to be one of combined shear and tension stress on the stud at the tension stressed region of the weld. In our analysis, this higher bending deformation combined with shear deformation reduces the headed stud capacity to a value lower than $1.0A_sF_{ut}$.

2.4 Connection plate thickness

Minimum plate thickness research is limited to work by Goble at Case Western Reserve University, [10] where he focused on the minimum flange thickness required in light-gage steel in order to fully develop a welded stud connection. Goble determined the minimum flange thickness required must be greater than 0.37d to develop the stud weld.

2.5 Minimum slab thickness

Steel stud failures in the push-off specimens were achieved in some relatively "thin" slabs ranging in thickness from 102 to 178 mm. We have concluded that slab thickness is not a variable influencing a stud steel shear failure.

3. Experimental Program

3.1 Background

The literature search and analysis of existing headed stud and cast-in-place anchor bolt data was used to formulate an experimental program, conducted in the WJE structural laboratory. The program tested 312 plate configurations in shear and 16 push-off type specimens. The tests were typically conducted in slabs measuring 1.2 x 3.0 m, or 1.5 x 1.5 m with either a 152 or 406-mm thickness. Push-off specimen tests simulated shear loading conditions when an embedded anchor group is adjacent to two side edges.

A total of 14 different combinations of plate size, stud spacing, stud embedment depth, and stud diameter were evaluated. Plate thickness and concrete compressive strength were not testing variables in the program. Headed stud diameters of 12.7 and 15.9 mm were tested in this program. Both tension and double shear guillotine tests were performed on the studs, "in-air," in support of this work.

Test specimen concrete was a commercially available 34.5 MPa, normal-weight concrete containing 19-mm limestone coarse aggregate. All slabs were cast with the anchorages on the bottom of the form to ensure good concrete consolidation around the studs. Reinforcement, used only in the 152-mm thick slabs for handling purposes, was placed as not to interfere with the stud anchorage plates or provide anchorage confinement.

All slabs were tested flat (horizontal) on the laboratory test floor. A specially fabricated channel pulling test rig had a welded shoe plate, which reacted on the back edge of the stud anchorage plate. This loading scheme was used to practically eliminate the eccentricity from the shear tests, which theoretically was one-half the plate thickness or 6.4 mm. All slab shear tests were instrumented with a load cell and two linear variable displacement transformers (LVDTs).

3.2 Individual stud tests

For design, it is convenient to base the headed stud capacity on the tensile yield or strength values and relate the steel shear capacity to a fraction of either value. Steels used for manufacturing headed studs do not generally exhibit well defined yield point values. The headed stud steel shear strength was thus correlated to the measured stud tensile strength properties.

WJE independently measured the geometry and tested the physical characteristics for the various steel heats in the project stock. Four different stud length and diameter configurations were received, manufactured from six different steel wire heats. Headed studs were tested for their tensile and shear strength properties, "in air." The test fixture was similar to that suggested in the American Welding Society (AWS) D1.1-2000 structural welding code.[14] Double shear, guillotine tests were conducted on the middle third of the shank to determine the steel shear strength.

A universal testing machine was adapted to tension test headed studs welded to a square plate. Tension test results for the various steel heats showed ultimate strengths of 536 to 563 MPa for the 12.7 mm diameter and 538 MPa for the 15.9 mm diameter studs. Each stud exhibited a roundhouse load-deformation curve, requiring the 0.2% offset determination of yield strength. The measured stud yield strength was approximately 80% of the tensile strength; the strength of each steel heat exceeded the AWS D1.1-2000 requirements shown below in Table 1. AWS Type B studs are headed, bent, or of other configuration. They are an essential component in composite beam design and construction, and constitute those most used in precast concrete construction.

Table 1 – Minimum mechanical property requirements for headed studs (from AWS D1.1-2000[14])

Property	Type A	Type B
Tensile Strength (min.)	420 MPa	450 MPa
Yield strength *(0.2% offset)*	340 MPa	350 MPa
Elongation *(min. % in 2 in.)*	17%	20%
Reduction of area (min.)	50%	50%

The double shear, guillotine tests were conducted with a three plate fixture. Double shear tests showed ultimate strengths of 328 to 381 MPa for the 12.7-mm diameter and 352 MPa for the 15.9-mm diameter studs. These tests imply the shear strength would be about 65% of the tensile strength. Earlier reported push-off test results exhibited a shear strength that is better than what these "in-air" material test results imply.

3.2 Tests loading away from a free edge (de4)

Shear load on anchorages directed away from a free edge is not commonly encountered in precast construction. However, special framing conditions may dictate use of this type of connection. In this study, 23 tests were conducted with the shear force directed away from the back free edge (refer to Figure 1). Two series had single studs and the third series had two headed studs oriented in one y-row. The two single stud anchorage series examined both 12.7 and 15.9 mm diameter studs. The two stud anchorage groups used 12.7-mm diameter studs, spaced 4.5d apart. All three series were tested in 406-mm thick specimens; h_{ef}/d for these tests were 5.34 and 5.93.

For the 12.7-mm diameter single stud connection, five de4 distances (4d to 12d) were evaluated with two tests performed per edge distance. Eight tests failed due to steel stud failure, and two failed at the stud weld. After failure, only minor concrete damage was observed. Concrete crushing at the stud front was accompanied by hairline, transverse cracks (normal to the shear load) propagating 50 to 100 mm each side of the stud center.

Seven tests were conducted with 15.9-mm diameter studs. Edge distances evaluated were 4d, 8d, and 12d, with three tests conducted at 4d. All tests failed in a steel shear mode, with no weld failures in this series. The two-stud anchorage tests used 12.7-mm diameter studs at nominal 4d, 8d, and 12d edge distances with six total tests. Two tests exhibited weld failures in one or both studs, while the other four tests failed by stud shearing through both stud shanks. From these tests, it was concluded that the de4 edge distance variable is not a factor causing concrete breakout of stud anchorages in shear.

3.3 Tests in-the-field

Some anchorages used in precast concrete members are located at such large edge distances that all concrete breakout capacities exceed the capacity developed by the individual studs failing in steel shearing; these test series are classified as *in-the-field* tests. Six series were conducted to test two and four anchor connections, with an emphasis on evaluating x- and y-row spacing and embedment depth effects on capacity.

These test series had 24 total tests in 406-mm thick test slab specimens using 12.7-mm diameter studs. The first two tests in a series used studs with an effective embedment depth (h_{ef}) of 67.7 mm; longer studs with h_{ef} = 124 mm were used for the second two tests. Based on the push-off testing review, steel stud failure can be achieved in relatively thin slabs. As such, we conclude slab thickness influence on the anchorage's ability to develop steel failure was viewed to have little effect, especially with the 12.7-mm diameter studs used in this study.

For the 24 tests conducted in the six *in-the-field* test series, the test-to-predicted steel stud shear capacities ranged from 0.90 to 1.05, using A_sF_{ut} as the calculation basis. When the short and long stud results are compared for all series, there is no discernable difference in the ultimate steel shear capacity due to stud length. For the x- or y-spacings investigated, 4.5d and 7.0d in different combinations and loading orientations, stud spacing did not have a significant effect on the ultimate shear strength.

4. Steel Failure Analysis

4.1 Data review and proposed design equation

Testing has shown that the steel stud failure mode typically occurred for back edge (de4) and *in-the-field* tests performed in 406-mm thick slabs. In all cases, steel failures were marked by two failure modes: a ductile, shear yielding-type stud failure, accompanied by appreciable lateral deformation, or a stud weld failure at the plate interface. When the corresponding failure area on the concrete slab specimen was observed, the still embedded studs had elliptical-shaped fracture surfaces with the major axis parallel to the load direction. The concrete in front of the stud was locally crushed, due to stud shank bearing; this concrete crushing also created a void (pocket) behind the stud.

The second steel failure type experienced was the weld of the stud to the plate. Varying degrees of weld region porosity, confined within the shank diameter, marked the weld fracture surface. Weld porosity often ranged from 25 to 75% of the shank area. In this test program, the stud failures due to welding were a random occurrence, attributed to weld machine malfunctions and operator error.

The steel shear failure database from this program is based on the de4 and *in-the-field* testing, and other tests (de1 and de3 testing), where the distance to a free edge was large enough to transition from a concrete to steel stud failure. Anchorage capacity governed by steel stud shank failure can be predicted by the number of studs in the group (n) times the stud area (A_s) multiplied by the ultimate stud tensile strength (F_{ut}). Stud weld failures, however occurred at steel shear stresses less than the ultimate tensile strength.

When the weld failure data are omitted from the population, the WJE database represents stud steel shear failures only; the number of tests is 80 with an average test-to-predicted ratio of 1.00. The sample standard deviation is 0.07, thus indicating the relative tightness of the data. A frequency distribution is plotted to the left in Figure 2. Given that the steel stud shank shear failures can be used as a database, the characteristic strength equation from a 5% fractile analysis (κ factor = 1.957) when the actual ultimate tensile strength is known, becomes:

$$V_{steel} = 0.86\, nA_sF_{ut} \qquad \text{(using actual } F_{ut}\text{)} \qquad (2)$$

However, actual tensile strength is generally not used in design. An analysis using the minimum **design** ultimate strength of 450 MPa from Table 1, shows the average test-to-

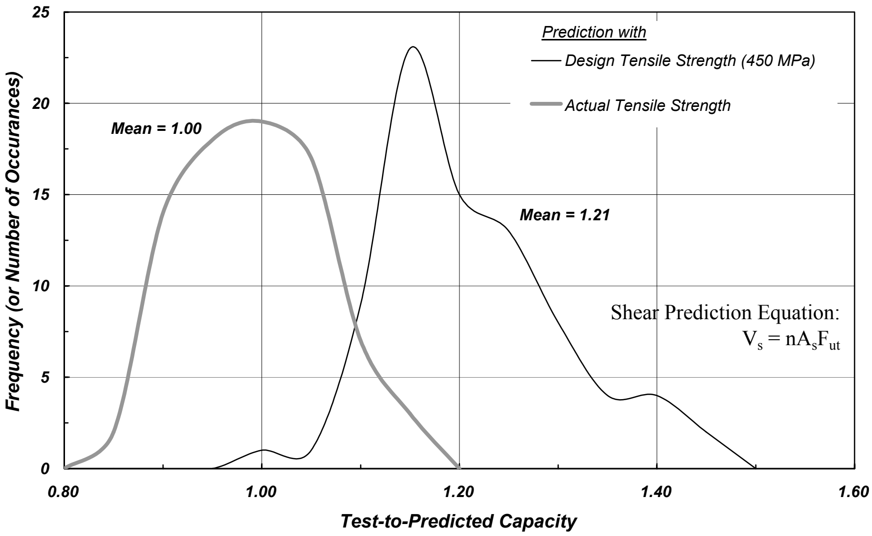

Figure 2: Frequency distribution plot of steel stud failures.

predicted ratio is 1.21 with a standard deviation of 0.10; the coefficient of variation is 7.8%. The 5% fractile characteristic prediction equation thus becomes:

$$V_{steel} = 1.0\ nA_sF_{ut} \qquad \text{(using design minimum } F_{ut}\text{)} \qquad (3)$$

From a probability standpoint, this indicates with 90% confidence that over 95% of the failure loads occur at a value represented by *Eq. (3)* above. Using the minimum design strength of 450 MPa and WJE data, no tests had test-to-predicted ratios less than 1.0.

4.2 Steel failure behavior

The reason there is an apparent steel shear strength increase when the stud is embedded in normal-weight concrete, versus “in air” results, is related to stud weld metallurgy. In the stud welding process, the shielded arc weld melts the stud end creating a shallow weld pool beneath the stud. The stud gun then plunges the stud into the molten weld pool, holding the stud in position while the liquid metal solidifies. Although this process occurs very quickly, a heat-affected zone (HAZ) is created in the weldment.

AWS defines the HAZ as that portion of the welded metal where the mechanical properties or microstructure have been influenced by the welding heat. The heat developed tends to heat-treat or temper the steel such that locally the steel’s strength and hardness will increase. This transformation hardening process is dependent on the initial material temperature after arcing, the cooling rate, and the final (ambient) temperature.[15]

Figure 3 shows a stud weld cross section submitted for metallurgical work, which had failed in a concrete breakout mode. The numbers represent locations where Rockwell B

Hardness tests were performed; the locations are shown in scale. The Rockwell B Hardness values were then converted to ultimate steel tensile strength.[16] Table 2 shows the approximate tensile strength is greater in the HAZ than the nominal stud shank strength. The stud typically sheared off above the weld flash region in the parent stud material, corresponding to hardness locations 2, 9, and 14 in Figure 3. Tensile testing of this stud heat revealed an average ultimate tensile strength (F_{ut}) of 538 MPa; on a relative basis, the indicated strength in the weld area is between 40 to 130 MPa higher.

Table 2 - Rockwell B Hardness readings.

Test Points	Rockwell B Hardness	Converted F_{ut} (MPa)
1	90.1	606.8
2 / 14	93.7	667.4
3	91.8	634.3
4	95.1	703.3
5	101.5	841.2
6 / 16	99.5	795.0
7	89.9	603.3
8	85.0	544.7
9 / 17	87.7	588.1
10 / 15	102.3	856.7
11	106.5	986.0
12	92.3	630.2
13 / 18	82.1	515.1

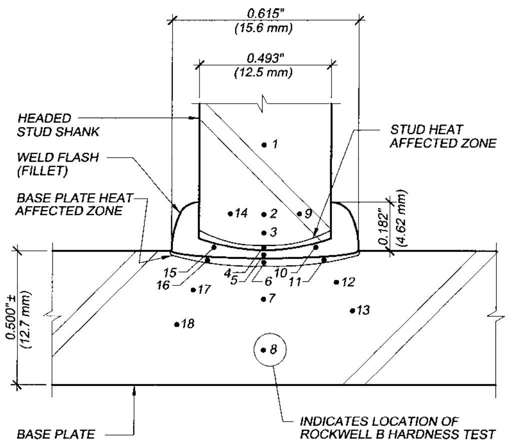

Figure 3: Stud weld cross-section.[1]

5. Summary

- Well-embedded studs are recommended to have a minimum effective embedment-to-diameter ratio (h_{ef}/d) of 4.5 to achieve steel stud failure. The minimum stud h_{ef}/d used in this study was 5.30, but the literature review justified a smaller h_{ef}/d.
- For steel failure in headed stud anchorage groups, this study shows the shear failure load is best predicted using the ultimate stud tensile strength. In normal weight concrete, *Eq. (3)* is recommended as the steel prediction equation (V_s) for headed studs with $h_{ef}/d > 4.5$. For lightweight concrete, see Reference 1 for background.
- Headed studs with an h_{ef}/d less than 4.5 will likely cause a pry out failure mode. The design ultimate capacity will be less than that predicted by $1.0A_sF_{ut}$. Again, Reference 1 provides a proposed characteristic equation for short, "stocky" studs.

6. References

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