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Headed Steel Stud Anchors in Composite Structures: Part II – Tension and Interaction

Luis Pallarés
Jerome F. Hajjar

Department of Civil and Environmental Engineering
University of Illinois at Urbana-Champaign

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Contact:

Prof. B.F. Spencer, Jr.
Director, Newmark Structural Engineering Laboratory
2213 NCEL, MC-250
205 North Mathews Ave.
Urbana, IL 61801
Telephone (217) 333-8630
E-mail: bfs@illinois.edu

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ABSTRACT

The current AISC Specification for Structural Steel Buildings (AISC, 2005a) is the lead specification for composite construction in the U.S. However, these provisions do not provide a recommendation for computing the strength of headed steel stud anchors (traditionally used as shear connectors) under tension or combined tension and shear. Headed stud anchors are subjected to these types of forces in composite structures such as infill walls, composite coupling beams, the connection region of composite columns, or composite column bases. While ACI 318-08 Appendix D (ACI, 2008) and PCI 6th Ed. (PCI, 2004) includes provisions for such conditions, those provisions are geared for more general anchorage conditions than are typically seen in composite construction. It would thus be beneficial to have design guidance specifically for the case of headed steel stud anchors subjected to tension or combined tension and shear in composite construction, evaluated within the context of the AISC Specification. In this work, different strength equations to compute the nominal tensile strength of a headed stud are reviewed and compared to experimental results. The resulting recommendations seek to ensure a ductile failure in the steel shank instead of a brittle failure within the concrete. Several criteria are proposed to ensure that a ductile failure controls in composite construction, and, different headed stud configurations and detailing reinforcement recommendations are proposed to improve the ductile behavior of headed stud anchors subjected to tension and combined tension and shear.

KEYWORDS

Composite Construction; Composite Wall; Steel Anchor; Shear Stud; Headed Stud; Shear Connector

CONTENTS

CHAPTER 1

INTRODUCTION

For composite structures, headed stud anchors (shear connectors) have traditionally been used to transfer shear loads between steel and concrete, especially in composite beams. As the use of composite construction increases, conditions that lead to tension and combined shear and tension in headed studs are becoming more prevalent, such as infill walls, coupling beams, connections to composite columns, or composite column bases. This type of connector has been investigated by numerous researchers worldwide, including work focused on anchoring either to reinforced or prestressed concrete.

Headed stud anchors subjected to tension and interaction have been investigated as an anchorage for steel frames to concrete walls or footings. The most advanced knowledge regarding anchorage to concrete is embodied in ACI 318-08 Appendix D (2008), CEB (1997), and PCI (2004). Provisions were also included in Appendix B of ACI 349 (1980) that were meant to ensure ductile behavior of cast-in-place anchors. This Standard required that the tensile strength of the anchor was less than or equal to the tensile strength of an idealized concrete cone surface (Fig. 1(a)). ACI-349 (2006), in turn, incorporates the approaches presented in ACI 318-05 Appendix D (ACI, 2005). However, the philosophy remains based on assuring a ductile failure mode so that the embedment anchor yields before the concrete fails. Ductility is desired to avoid brittle failures by the concrete, particularly in such important structures as nuclear power plants. In Pallarés and Hajjar (2009), regarding headed steel stud anchors subjected to shear forces, a minimum ratio between the height of the stud and the diameter of the shank is proposed so as to help ensure a ductile failure of the stud in the steel shank.

However, the main objective of a number of prior studies of tension in steel anchors has been to determine the behavior of anchors when the length is not sufficient to develop a ductile failure in the steel. As Cannon (1995) reported, in the 1960's there were no established criteria for the design of cast-in-place anchors other than those published by manufacturers of welded studs (e.g., Nelson Stud Welding, 1974). These anchors were of limited depth and were used primarily as shear connectors.

Many studies have also investigated post-installed anchors [e.g., Cook et al (1992, 1996); Zamora et al. (2003); Shirvani et al. (2004); Eligehausen et al. (1995, 2006)] subjected to tension forces. Cannon (1995) indicates that manufacturers of post-installed expansion anchors usually designed anchors to fail in the concrete, and expounded this feature to promote the quality of their product. However, an advantage of headed studs in composite structures versus post-installed is that they provide more reliable performance since they are cast-in-place and reinforcement can be provided around the anchor to increase the resistance of the connection.

CHAPTER 2

OBJECTIVES

This work reports on the behavior of headed studs embedded in solid concrete slabs both parallel to (as in an infill wall) and perpendicular to (as in a steel reinforced concrete composite column) the longitudinal axis of the shear connector subjected to both monotonic and large amplitude cyclic (i.e., seismic) forces. An extensive set of test results of headed steel anchors in configurations applicable to composite construction has been collected and analyzed relative to the design provisions provided in Appendix D ACI 318-08 (2008) and PCI 6th (2004). Recommendations and design guidelines specific to composite construction are then proposed within the context of the AISC Specification (AISC, 2005a) for headed steel stud anchors subjected to tensile and combined tensile and shear forces in composite construction.

The scope of this work is limited to headed steel stud anchors highlighted by AISC (2005a) Chapter I with diameters less than or equal to 1 in. (25 mm) Section I1.4 of the commentary of AISC (2005a) specifies the nominal yield and tensile strengths for typical ASTM (1999) A108 Type B studs as 51 ksi (350 MPa) and 65 ksi (450 MPa), respectively. Tests from the literature were not considered for this study if there were likely edge effects. Specifically, tests were only considered if they had a minimum distance of 1.5 times the effective length of the anchor to any edge. Moreover, tests collected in this work were included only if the concrete strength (for both normal and lightweight concrete) was larger than 3 ksi (21 MPa), which is the minimum strength permitted by AISC (2005a).

CHAPTER 3

MONOTONIC BEHAVIOR OF HEADED STUDS SUBJECTED TO TENSION FORCES

Two general philosophies exist to predict a brittle tension failure of an anchor, the 45-degree cone method and the concrete capacity design (CCD) method. For the 45-degree cone method (Courtois, 1969), the concrete strength of an anchor is computed assuming a conical surface (Figure 1(a)) taking the slope between the failure surface and concrete surface as 45 degrees. As the depth of embedment of the headed anchor increases, the area of the conical section increases proportionately up to the point of full embedment. Following this philosophy and deducing results from experiments, Nelson Stud Welding (1974) stated an embedment depth of 8 to 10 times the anchor shank diameter was required for the concrete breakout strength to be larger than the tensile strength of the steel in headed anchors. From the Nelson report, a reduction factor of 0.75 was proposed to determine the concrete capacity strength in the case of lightweight concrete.

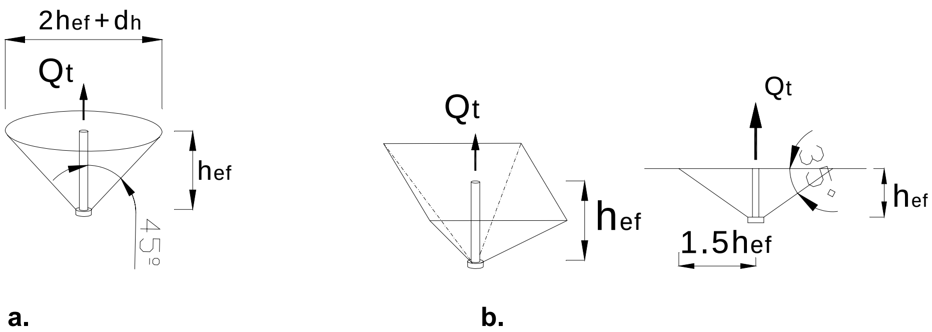


Figure 1. a) Conical failure surface. b) Four-sided pyramid failure surface

Cannon et al. (1981) proposed one of the first guides to compute anchors subjected to tension, shear and combined loads. These recommendations included using a conical failure surface to compute the tensile strength and were adopted by ACI 349-80 (1980). The design strength of concrete for anchorage was based on a uniform tensile stress of $\phi\left(4\sqrt{f_c'}\right)$ acting on an effective stress area, which was defined by the projected area of stress cones radiating toward the attachment from the bearing edge of the anchors. The resistance factor, ϕ, was 0.65.

PCI (1978) adopted the conical failure surface to predict a brittle failure of the concrete and this method was retained in the PCI Handbook until the 5th Ed (1999). However, PCI (2004) adopted the provisions in ACI 318-02 Appendix D (ACI, 2002), which were based on CCD, to compute tension strength of anchors assuming uncracked concrete. In the CCD method (Fuchs et al., 1995), the concrete strength of a single anchor is calculated assuming a four-sided pyramid failure surface, with a slope between the failure surface and the surface of the concrete member of 35 degrees (Figure 1(b)). The most recent versions of ACI 318 Appendix D (ACI, 2005, 2008) retained this approach.

With respect to documenting the behavior of headed steel stud anchors in connections between steel and concrete through embedded plates, the anchors need to be analyzed as a group when the separation between anchors is less than $3h_{ef}$ (ACI 318-08). As examples of this type of construction, Roeder and Hawkins (1981) focused on the behavior of connections between steel beams and concrete walls, and Murray (1983) and Marsh and Burdette (1985) carried out a study of anchor plates in concrete footings. These types of connections are covered extensively by ACI 318 Appendix D (ACI, 2008).

Bode and Roik (1987) conducted 106 tests deducing a formula to predict the tensile strength of Nelson headed studs: The characteristic value of tensile strength, i.e., is the value of the strength exceeded by 95% of the cases with a 90% confidence, is presented in by Eq. 1 and the average value is given by Eq. 2.

$$N_b = 8.90\sqrt{h_{ef}}\left(h_{ef} + d_h\right)\sqrt{f'_c} \qquad (\textit{Units: N, mm}) \qquad [1]$$

$$N_{b,avg} = 10.96\sqrt{h_{ef}}\left(h_{ef} + d_h\right)\sqrt{f'_c} \qquad (\textit{Units: N, mm}) \qquad [2]$$

Cook et al. (1992) carried out 178 tests with headed studs and retrofit anchors subjected both to static and dynamic loads, comparing the deflection behavior and mode of failure. The main conclusion about cast-in-place anchors was that the embedment length provisions of ACI 349-80 (ACI, 1980) ensure the anchors fail in a ductile mode.

Saari et al. (2004) simulated the edge conditions of infill walls, taking into consideration typical reinforcement used in these structural elements. The conclusions gathered from this work for anchors subjected to tension and combined tension and shear interaction indicate that proper confinement (e.g., steel cages around the anchor) can mitigate concrete breakout failure.

A comprehensive state-of-the-art in cast-in-place and post-installed anchors can be found in ACI 355 (1997) and CEB (1994). The above review summarizes the basis for current, general anchorage provisions of embedded anchors subjected to tension and shear plus tension interaction. There are several special circumstances for anchors in composite structures that should be considered when reviewing this work: a) Headed steel stud anchors in composite construction are attached to hot-rolled sections that provide substantial stiffness as compared to flexible plates that are often used in the literature to study the effect of embedded anchorage conditions; this affects the distribution of force between adjacent anchors. b) Composite construction that has cases of anchors subjected to tension or shear plus tension interaction generally includes steel reinforcing bars in the concrete to confine the steel anchors. c) Edge conditions that cause premature failure are virtually always avoided in the vicinity of cast-in-place anchors in composite construction due to the presence of typical perimeter reinforcement.

3.1. Comparison of PCI 5th and ACI 318-08 / PCI 6th for tension

Throughout the different editions of the PCI Handbook, there have been several formulations to compute the tensile strength of an anchor. Conical failure surface for an anchor in tension was adopted up to PCI (1999), as presented in Table 1, with a resistance factor of 0.85.

Table 1. Pull-out and breakout strength formulas by PCI 5th, ACI 318-08 and PCI 6th.

	PULL OUT	BREAK OUT
PCI 5th	-	$12.6h_{ef}\left(h_{ef}+d_h\right)\lambda\sqrt{f'_c}$
ACI318-08 (5% fractile)	$8A_{brg}f_c'\psi_{c,P}$	$24\lambda\sqrt{f_c'}\left(h_{ef}\right)^{1.5}\psi_{c,N}$
ACI318-08 (Average)	$13A_{brg}f_c'$	$40\lambda\sqrt{f_c'}\left(h_{ef}\right)^{1.5}$
PCI 6th (5% fractile)	$11.2A_{brg}f_c'C_{crp}$	$3.33\lambda\sqrt{\frac{f_c'}{h_{ef}}}\left(9h_{ef}^{\ 2}\right)$

Units: pounds, inches;

As discussed earlier, PCI (2004) changed the approach to a four-sided pyramid cone, adopting a similar formulation to ACI 318 (2002, 2005, 2008), though working with coefficients related to uncracked concrete. The results given by PCI 6th (2004) then coincide with results given by the ACI 318-08 Appendix D (ACI, 2008).

Concrete failure occurs, when not influenced by edge conditions, when the minimum of either the “pull out strength” or “breakout strength” is reached before the steel strength is reached. The expressions used to check the pullout and breakout strength are presented in the Table 1, presenting the 5% fractile formula (which is used as the nominal strength formula) for PCI 5th and distinguishing between the 5% fractile (nominal strength) formula and the average formula for the ACI 318-08 (CCD method), since the average formulas of CCD may be found in Fuchs et al. (1995). PCI 6th adopted the ACI 318 formulas in the particular case of uncracked concrete. The nominal strength (5% fractile) formula used in ACI 318 Appendix D for anchoring, such as Wollmershauser (2004) reported, presents a 90 percent confidence that 95 percent of the anchor ultimate loads exceed the 5 percent fractile value.

The steel strength may be expressed by the formula $\phi_t C_t A_s F_u$, where ϕ_t and C_t are summarized in the Table 2 for the specifications assessed in this work.

Table 2. Resistance factors in tension adopted by PCI 5th, ACI 318-08 and PCI 6th.

	Steel Failure			*Concrete Failure*		
	ϕ_t	C_t	$\phi_t \cdot C_t$	ϕ_t	C_t	$\phi_t \cdot C_t$
PCI 5th	1.00	0.90	0.65	0.85	1.00	0.85
ACI 318-08(*) / PCI 6th	0.75	1.00	0.75	0.70	1.00	0.70

(*) Ductile element has been adopted

In total, 222 tests were considered when examining the monotonic behavior of headed studs in tension, with results obtained from Sattler (1962), Nelson Stud Welding (1966, 1974), McMakin et al. (1973), Cannon et al. (1975), Roik et al. (1978), Keuser (1989), Balogh et al. (1991), Cook et al. (1992), Zhao (1993), and Saari et al. (2004). Tests that were heavily influenced by group effects due to two or more studs being closely grouped together were omitted from this data set. A summary of the tests found in the literature with their hef/d ratios, number of tests reported by authors, type of configuration, type of concrete, and range of concrete strength is shown in Table 3. The strength was computed for each test using the PCI 5^{th} and ACI 318-08/PCI 6^{th} approaches and compared with the experimental results. For all tests, the ratio between the experimental strength and the predicted strength is presented in the graphs. The different authors usually tested anchors subjected to tension forces without carrying out tests on single anchors to determine the measured strength of the steel. The nominal values reported by the authors are thus used for the predictions in this work.

Assessing the accuracy of the provisions for all tests analyzed in this work (Figure 2), it can be seen that PCI 5^{th} (Figure 2(a)) presents slightly more conservative results and larger scatter than ACI 318-08 and PCI 6^{th} (Fig. 2(b)). While the scatter of the ACI 318-08 and PCI 6th is lower, the accuracy is still affected by several outliers, including two from the infill wall tests reported by Saari et al. (2004), tests #1 and #60 in the graph. The main difference between configurations 1 (test #60) and 2 (test #1) in Saari et al. (2004) was the amount and location of steel reinforcement. Configuration 1 (Figure 3(a)), designed as a “perimeter bar” scheme, consisted of tying two #4 reinforcing bars directly to each line of steel anchors. This configuration did not facilitate the transfer tension load from the stud to the reinforcement due to the small overlap between them. The failure was due to breakout of the concrete for this configuration. Configuration 2 (Figure 3(b)) was designated as a “confinement cage” and was designed specifically to avoid all likely failure modes in the concrete in infill walls. The steel cage provides two beneficial effects: first is the confinement of the concrete provided by the reinforcing cage, which is tangible but difficult to quantify; second, a reinforcement detail crosses each likely failure surface of the concrete. In this way, the steel anchor can transfer the axial tensile load to the reinforcement following the scheme shown in Figure 3(c).

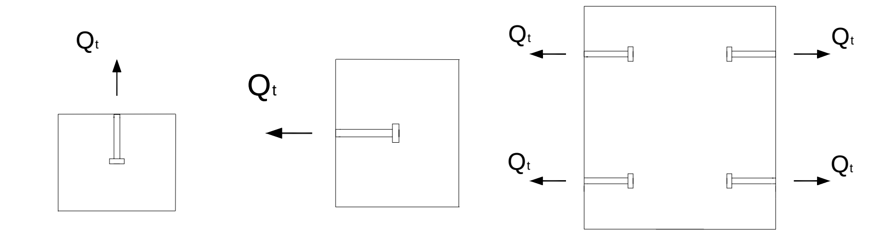
Analyzing the results with resistance factors, PCI 5^{th} (Fig. 2(c)) applies a resistance factor of 0.85 for the concrete formula and a resistance factor of 1.00 for the steel formula. ACI 318-08 and PCI 6^{th} provide a resistance factor 0.7 for the concrete formula and 0.75 for the steel formula. ACI 318-08/PCI 6^{th} (Fig. 2 (d)) then becomes more conservative than PCI 5^{th} (Fig. 2(c)) when resistance factors are applied.

Tests failing within the steel shank or weld (59 tests) are well predicted by the steel formula $A_s F_u$, as can be seen in Figure 4, particularly Figure 4(b) for ACI 318-08 and PCI 6^{th}. Without resistance factors, PCI 5^{th} is more conservative in its prediction of the strength (Fig. 4(a)) due to the coefficient C_t = 0.9 (see Table 2). However, with resistance factors, ACI 318-08/PCI 6^{th} (Fig. 4(d)) become more conservative, with an average value of the test-to-predicted ratio of 1.443, as compared to PCI 5^{th} (Figure 4(c)).

Table 3. Headed steel anchor test configurations for headed studs subjected to tension

TYPES OF CONFIGURATION

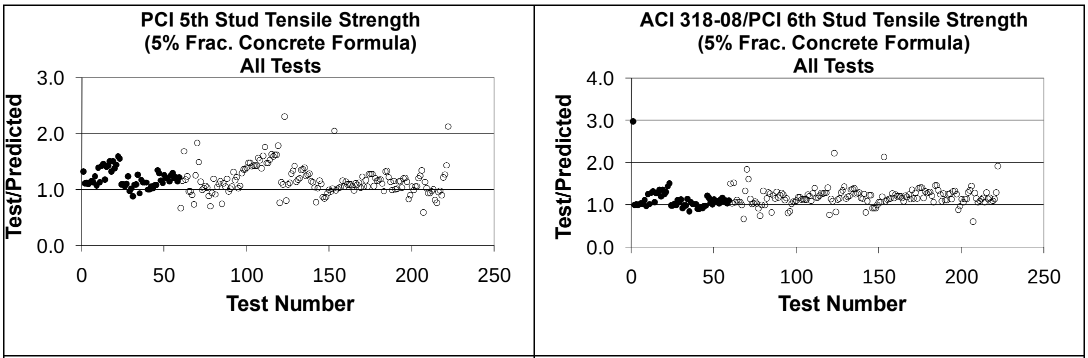
Type 1	***Type 2***	***Type 3***



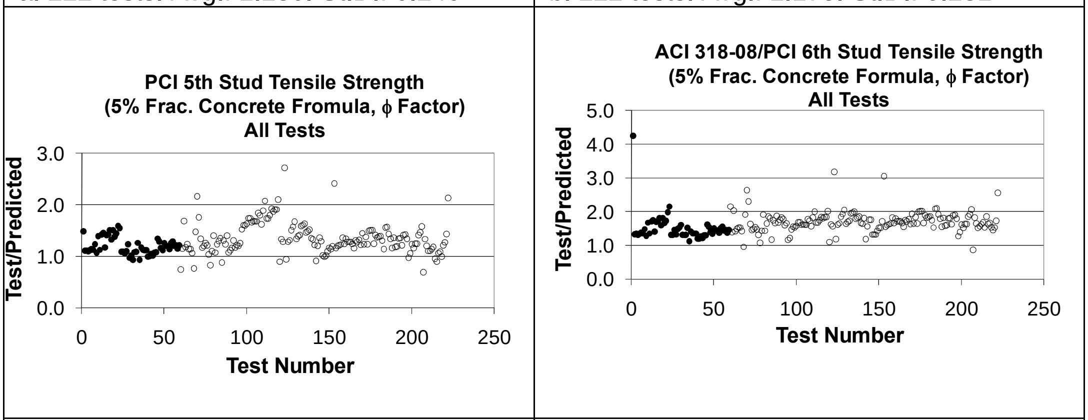
Reference	***h_{ef}/d***	***# Tests***	***Type***	***# Studs***	***Concrete # Tests***	***Range of f_c' (ksi)***
Sattler (1962)	3.57	2	1	1 stud	Normal : 2 Lightweight: 0	6.51 – 6.86
Nelson Stud Weld (1966)	4.00, 4.58, 4.66, 5.50, 7.37, 8.57, 10.00, 10.33,	22	1	1 stud	Normal : 22 Lightweight: 0	2.95 – 3.11
McMakin et al. (1973)	5.33, 9.33, 10.67	15	2	1 stud	Normal : 12 Lightweight: 3	4.65 – 5.26
Cannon et al. (1975)	4.00, 5.33, 6.67, 8.00, 9.33, 10.20, 10.67	30	1	1 stud	Normal : 30 Lightweight: 0	3.11 – 5.05
Klingner and Mendonca (1982)	2.99, 3.25, 4.00, 4.55, 4.71, 5.00, 5.50, 4.60, 4.67, 9.17, 9.89, 10.44	27	1	1 stud	Normal : 27 Lightweight: 0	3.52 – 6.25
Bode and Roik (1987)	2.93, 4.06, 4.22, 4.51, 5.18, 5.80	90	1	1 stud	Normal : 90 Lightweight: 0	3.64 – 6.73
Keuser (1989)	2.27, 4.54, 6.81	9	1	1 stud	Normal : 2 Lightweight: 0	4.54
Balogh et al. (1991)	4.68, 4.75, 5.00, 5.18, 5.43	6	1	1 stud	Normal : 6 Lightweight: 0	3.20 – 3.41
Cook et al. (1992)	7.60, 11.2	8	1	1 stud	Normal : 8 Lightweight: 0	5.00
Zhao (1993)	4.09, 7.27, 8.63	11	1	1 stud	Normal : 11 Lightweight: 0	3.59 – 5.05
Saari et al. (2004)	6.67	2	3	4 studs	Normal : 2 Lightweight: 0	4.00

Tests failing in the concrete (163 tests, Figure 5) presented larger scatter than tests failing in the steel, and the scatter increased when the resistance factors were applied. In addition, the prediction of failure by a conical surface (PCI 5th) became more accurate, however it was unconservative for many tests analyzed in this work, as can be seen in Figure 5.

From the analysis of the tests failing in the concrete, it may thus be deduced that the ACI 318-08/PCI 6th approach (four-sided pyramid) better predicts the behavior of the concrete failure modes due to the similar average value as compared to the PCI 5th equation, but with a smaller standard deviation. Since the model of the four-sided pyramid that forms a slope of 35º with the horizontal surface (Figure 1(b)) is concluded to be more accurate for concrete failures, it may be said that the edge effects for studs in tension are avoided if the distance to a free edge is larger than $1.5h_{ef}$.

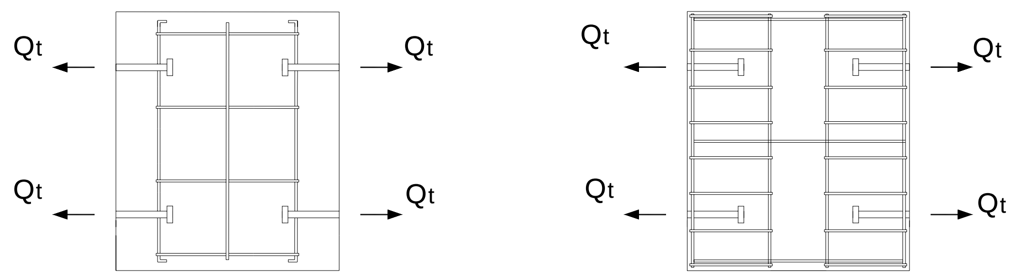


a. 222 tests. Avg.: 1.180. St.D.: 0.240	b. 222 tests. Avg.: 1.175. St.D.: 0.231

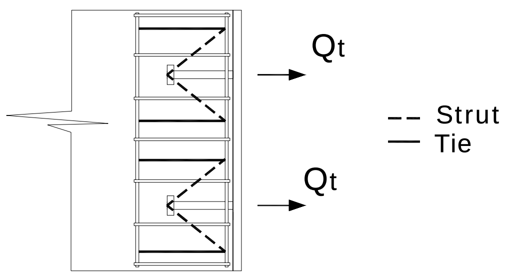


c. 222 tests. Avg.: 1.319. St.D.: 0.282	d. 222 tests. Avg.: 1.649. St.D.: 0.339

Figure 2. Assessment of tensile strength using the minimum of the steel and concrete failure formulas in PCI 5th (1999) and ACI 318-08 (2008) / PCI 6th (2004) for all tests.

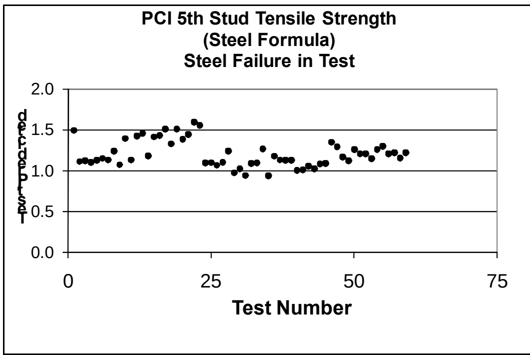


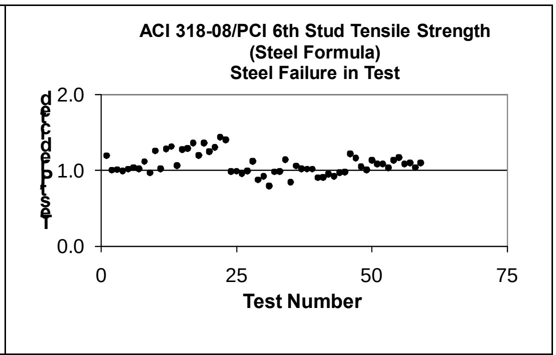
a. **b.**



c.

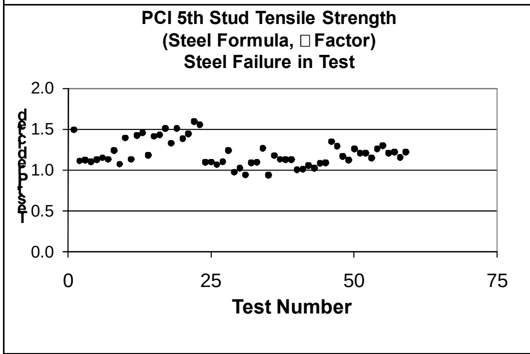
Figure 3. Configuration of reinforcement in tests of Saari et al. (2004). a) Configuration with "perimeter bar". b) Configuration with cage bar. c) Strut-and-tie model developed for Saari's test.

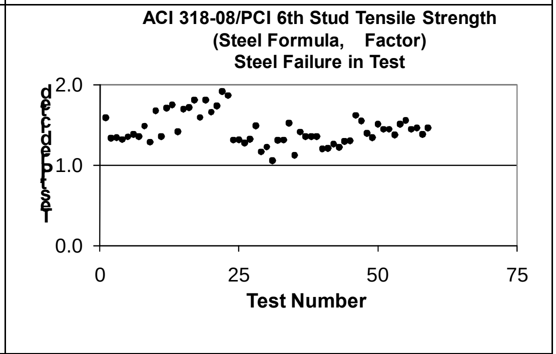




a. 59 tests. Avg.: 1.202. St.D.: 0.159

b. 59 tests. Avg.: 1.082. St.D.: 0.144





c. 59 tests. Avg.: 1.202. St.D.: 0.159

d. 59 tests. Avg.: 1.443. St.D.: 0.191

Figure 4. Assessment of tensile strength due to steel formula of PCI 5th (1999) and ACI 318-08 (2008) / PCI 6th (2004) for tests that failed in the steel.

3.2. Assessment of headed steel stud tensile strength using $A_s F_u$

Using recommendations by Ravindra and Galambos (1978), resistance factors can be computed to provide a required level of safety using the steel prediction formula ($A_s F_u$) for the tests failing by the steel. Given a reliability index, the resistance factor can be computed using (Eq. 3).

$$\phi_t = \frac{R_m}{R_n} e^{(-\alpha \beta V_R)} \qquad [3]$$

where:

$\frac{R_m}{R_n}$ is the average of the ratio between test result and predicted value (equals to 1.082 for the analyzed test). Predicted values are computed using the nominal material properties reported by the authors.

α equals to 0.55, given by Ravindra and Galambos (1978).

β is the reliability index that is equal to 4 in this study.

$$V_R = \sqrt{V_F^{\ 2} + V_P^{\ 2} + V_M^{\ 2}}$$

where:

V_F is the coefficient of variation on fabrication ($V_F = 0$) since variation is embedded in test results because tests from numerous experimentalists have been used in this work;

V_P is the coefficient of variation of $\frac{R_m}{R_n}$ (equals to 0.144 in the analyzed tests);

V_M is the coefficient of the variation of the materials ($V_M = 0.09$, from Ravindra and Galambos (1978) since the strength of the studs is not reported by the authors and the nominal value has been taken to assess $A_s F_u$).

Galambos and Ravindra (1978) suggest a reliability index of 3 for members and 4.5 for connections. In this work, a reliability index of 4 is targeted.

Equation 4 presents a sample calculation of the resistance factor for β = 4.

$$\phi_t = \frac{R_m}{R_n} e^{(-\alpha\beta V_R)} = 1.082 e^{(-0.55\cdot 4\cdot 0.144)} = 0.73 \quad [4]$$

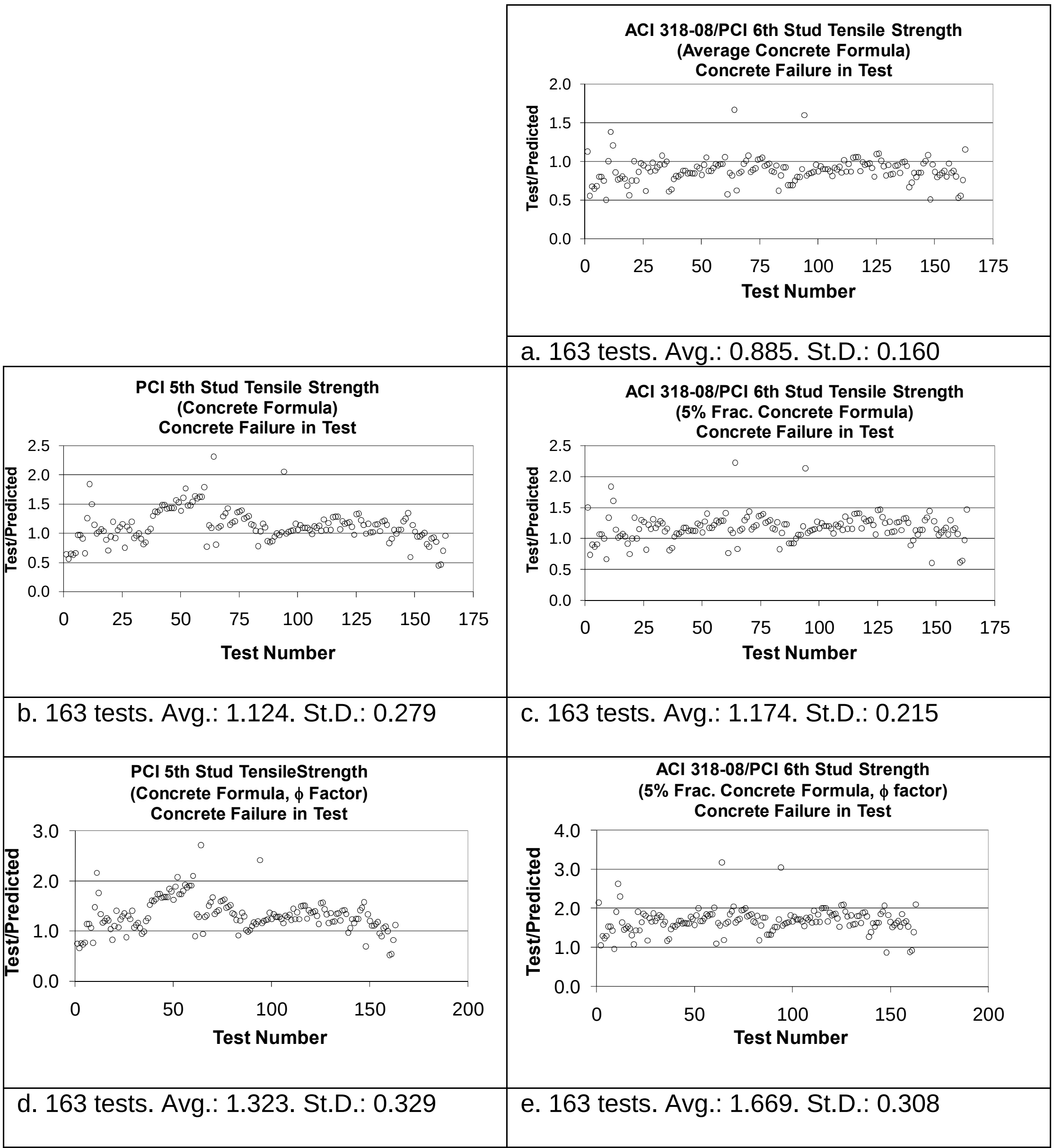
In conclusion, from the analyzed tests, a C_t factor of 1 and resistance factor (ϕ) of 0.75 is recommended to predict safely the behavior of headed studs in tension when the failure occurs in the steel.

3.3. Headed steel stud tensile strength for $h_{ef} / d > 7.5$.

Within the context of AISC design (AISC, 2005a), the philosophy of design for composite structures is typically based on a ductile failure in the composite members. Hence, the provisions proposed herein for tensile strength seek to ensure a ductile failure by the steel. A flow chart summarizing the types of failures for studs in the concrete subjected to tension forces is given in Figure 6. Recommendations to avoid brittle failures by the concrete are presented in this section. The properties of typical headed studs [e.g., Nelson Stud Welding (2004)] are listed in Table 4. The estimated steel strength ($A_s F_u$), assuming a nominal strength of 65 ksi (450 MPa) is also presented in Table 4.

Three main types of failure may occur in the concrete in an anchor subjected to tension, namely: side-face blow-out, pullout, and concrete breakout (Figure 6). Side-face blow out is not considered in this study because it occurs only when an edge is located near the anchor (Figure 6). This type of failure is automatically avoided (Section D.5.4. of ACI 318-08, Appendix D) when two times the distance to an edge is larger than h_{ef}.

Regarding pull-out failure, the strength given by the 5% fractile formula and the average formula (Table 1) for concrete pull-out failure and steel shank strength are computed and listed in Table 4 for the most common headed stud anchors. Assuming that the concrete strength in the composite component equals 3 ksi (21 MPa) to reach a conservative result and assuming uncracked concrete (due to the typical reinforcement, mitigating the effects of cracking, that exists in composite components), if the steel strength formula is equated to the concrete pullout average strength formula, then a minimum ratio of 1.63 between the diameter of the head and the diameter of the shank is found to be needed to obtain a steel failure instead of a brittle pullout failure (this minimum required ratio decreases with increasing concrete strength).



a. 163 tests. Avg.: 0.885. St.D.: 0.160

b. 163 tests. Avg.: 1.124. St.D.: 0.279

c. 163 tests. Avg.: 1.174. St.D.: 0.215

d. 163 tests. Avg.: 1.323. St.D.: 0.329

e. 163 tests. Avg.: 1.669. St.D.: 0.308

Figure 5. Assessment of tensile strength due to concrete formula of PCI 5^{th} (1999) and ACI 318-08 (2008) / PCI 6^{th} (2004) for tests that failed in the concrete.

For breakout failure, a theoretical deduction may be presented in the same manner. A ratio of effective height to shank diameter (h_{ef}/d) necessary to have the concrete breakout strength be equal to the steel shank strength of the headed studs may be determined. The 5% fractile formula and the average value to compute the breakout strength of a single anchor subjected to tension in uncracked concrete are presented in Table 1. Ratios for the value of h_{ef}/d that provide a ductile failure of the headed stud rather than a failure in the concrete assuming a nominal steel strength of 65 ksi (450 MPa) and several concrete strengths [between 3 ksi (21 MPa) and 10 ksi (69 MPa)] are presented in the Fig. 7 for both the 5% fractile formula without resistance factors and the average formula of the ACI 318-08. For normal weight concrete, using the average formula to develop the full strength of steel [65 ksi (450

MPa)] before concrete fails by breakout (Figure 7(a)), a minimum ratio of h_{ef}/d between 4 and 8.1 is necessary to avoid failure in the concrete, depending on the concrete strength and the diameter of the headed stud. A formula of the minimum value of h_{ef}/d required to ensure failure in the steel may also be deduced (Eq. 5) from the graphs presented in the Figure 7(a) (ACI 318-08, average formula) as a function of the concrete strength and the steel anchor diameter:

$$h_{ef}/d = [(-0.91 f_c'+155)d + (-30.5 f_c'+5060)]/1000 \quad (\textit{Units: N, mm}) \qquad [5]$$
$$h_{ef}/d = (-0.16 f_c'+3.94)d + (-0.21 f_c'+5.06) \qquad (\textit{Units: kips, inches})$$

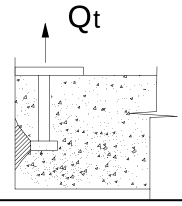
A ratio of 7.8 (the maximum from Eq. 5 for the range of values shown in Figure 7) is obtained from the Eq. 5 to ensure a ductile failure when f'c = 3 ksi (21 MPa) and d = 1 in. (25 mm). Using the 5% fractile formula of ACI 318-08 (Fig. 7(c)), a ratio of 7.8 (the maximum for the range of values shown in Fig. 7) is obtained to ensure a ductile failure when f'c = 3 ksi (21 MPa) and d = 1 in. (25 mm).

On the other hand, using the PCI 5th formula presented in Eq. 3, it is determined (Figure 7(e)) that the ratio is between 5.1 and 6.6 and does not depend significantly on the diameter of the stud, only on the concrete strength.

Furthermore, from the experiments, a summary of failure modes found in the 222 tests of headed steel stud anchors subjected to tension is given in Table 5. All tests have been classified as greater than or less than a given h_{ef}/d ratio of 5.5, 7.5, or 9.5. Assuming that h is a few percent larger than hef to account for the depth of the stud head, it can be reasoned that for a headed stud whose h_{ef}/d value is right at the limit equal to 7.5, a proposed minimum value of 8 for h/d may be adequate to check the steel formula alone, assuming no edge effects are engaged in the region of the steel anchor, since 82% of the 61 tests with ratios larger than this limit failed in the steel.

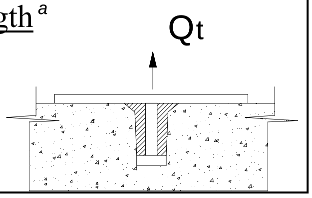
In order to predict the 18% of the tests that failed by the concrete with h_{ef}/d >7.5, Fig. 8(a) through 8(c) assess the accuracy of using the steel strength formula, based on using different values of ϕ Ct equal to 1.0, 0.75, and 0.65. A value of ϕC_t of 0.75, comparable to what is currently in ACI (2008) and PCI (2004) as per Table 2 and Figure 4, proves to be adequately conservative for all tests. In addition, use of only the steel formula, for simplicity, is reasonable because so few anchors specifically in composite construction are likely to fail in the concrete if h/d is greater than or equal to 8.

Side Blow Out



According to Section D.5.4 of ACI 318-08 Appendix D, side blow out failure is avoided if $h_{ef} < 2.5 \cdot$(distance to an edge).

Pull Out Strength[a]



According to ACI 318-08, it can be demonstrated that pull out failure is avoided if $d_h < 1.71 \cdot d$. Since:

Steel Strength: $N = A_s F_u$

Pull out strength (5% fractile): $N_c = \Psi_{c,P} 8 A_{brg} f_c'$

Pull out strength (Average): $N_c = 13 A_{brg} f_c'$

where:

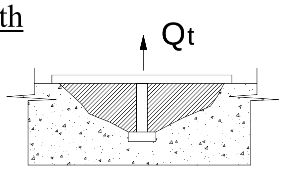
$$A_s = \pi \frac{d^2}{4}, \; A_{brg} = \frac{\pi}{4}\left(d^2{}_h - d^2\right), \text{ and } \Psi_{c,P} = 1.4$$

and to be conservative, if it is assumed that $f_c' = 3\,\text{ksi}$ (21 MPa), $F_u = 65\,\text{ksi}$ (450 MPa), then the steel strength should be larger than the pullout strength:

5% frac.: $\pi \frac{d^2}{4} 65 = 1.4 \cdot 8 \cdot \frac{\pi}{4}\left(d^2{}_h - d^2\right) 3$, then $\frac{65}{1.4 \cdot 8 \cdot 3} = \frac{d^2{}_h}{d^2} - 1$, $\rightarrow d_h = 1.71 \cdot d$

Average: $\pi \frac{d^2}{4} 65 = 13 \cdot \frac{\pi}{4}\left(d^2{}_h - d^2\right) 3$, then $\frac{65}{13 \cdot 3} = \frac{d^2{}_h}{d^2} - 1$, $\rightarrow d_h = 1.63 \cdot d$

Breakout Strength



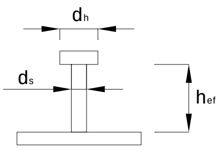
According to the work presented in this work, breakout failure is avoided if h_{ef} > *7.5d*

[a]*Units: kips, inches*

Figure 6. Summary of concrete failure modes.

Figures. 8(a) through 8(c) are based on assuming a constant, conservative, minimum value of 7.5 for the value of h_{ef}/d above which only the steel formula needs to be checked. However, Eq. 5 can also be used to calculate this value, often resulting in a lower cutoff value for this ratio. For example, if one assumes that the minimum concrete strength in a composite component is 5 ksi (35 MPa) and the headed steel stud anchors are 3/4 in. (19 mm) in diameter, the value of h_{ef}/d above which only the steel formula needs to be checked is 6.36 from Eq. 5. Figure 8(d) then shows the mean test-to-predicted ratios for tests having h_{ef}/d greater than or equal to 6.36, with concrete strength greater than or equal to 5 ksi (35 MPa), shank diameters equal to 3/4 in. (19 mm), and using only the steel formula to check the strength of the headed steel stud anchors in tension, assuming a value of ϕC_t of 0.75. Only one such test in this category fails in the concrete, and all tests are predicted safely. This exemplifies the characteristics of Eq. 5, and it is thus proposed that Eq. 5 may be used to determine the minimum value of h_{ef}/d above which only the steel formula needs to be checked.

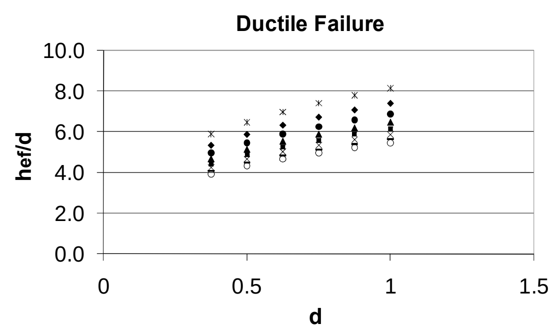
Table 4. Geometric dimensions, ultimate steel strength [assuming 65 ksi (450 MPa)] and pullout (5% fractile and average) strength for uncracked concrete and $f_c^{'} = 3$ ksi (21 MPa) for common Nelson studs



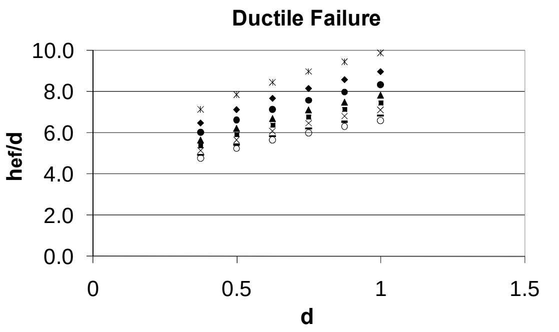
d	d_h	A_{brg}	d_{head}/d	Steel Strength[a]	Pullout[a]	
					5% fractile	Average
0.375	0.75	0.33	2.00	7.18	11.11	12.92
0.5	1	0.59	2.00	12.76	19.82	22.97
0.625	1.25	0.92	2.00	19.94	30.93	35.90
0.75	1.25	0.79	1.67	28.72	26.39	30.63
0.875	1.375	0.88	1.57	39.09	29.69	34.46
1	1.625	1.29	1.62	51.05	43.30	50.75

[a] *Units: kips, inches*

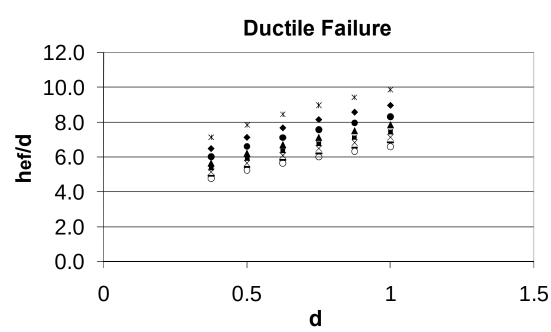
In the same sense, a minimum ratio of h_{ef}/d to ensure failure in the steel anchor rather than in the lightweight concrete may be deduced by reducing the concrete strength by the reduction factor for lightweight concrete (λ equal to 0.75) given by Nelson Report (1974) and ACI 318-08. This results in a minimum ratio of 9.5 for h_{ef}/d (approximately equal to a minimum value of h/d of 10) when using the average formula of ACI 318-08 (Figure 7(b)) to reach a ductile failure in the steel instead of concrete breakout failure for steel anchors subjected to tension for lightweight concrete. When using the 5% fractile formula, the envelope to ensure a ductile failure is 12 (Figure 7(d)).



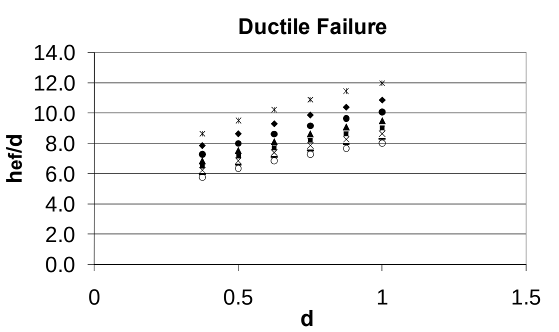
a. ACI 318-08 (Average formula). Normal Weight Concrete[a]



b. ACI 318-08 (Average formula). Lightweight Concrete[a]

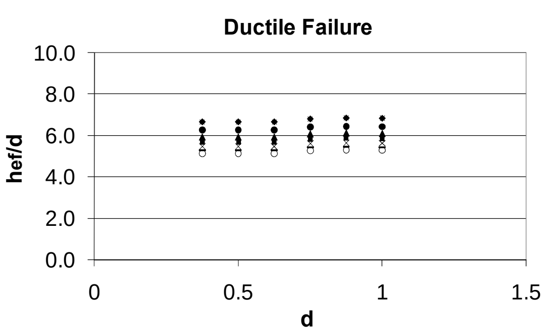


c. ACI 318-08 (5% Frac. Formula). Normal Weight Concrete[a]



d. ACI 318-08 (5% Frac. Formula). Lightweight Concrete[a]

e. PCI 5th. Normal Weight Concrete[a]



f. PCI 5th. Lightweight Concrete[a]

- ✳ f'_c = 3 (ACI-318-08)
- ◆ f'_c = 4 (ACI-318-08)
- ● f'_c = 5 (ACI 318-08)
- ▲ f'_c = 6 (ACI 318-08)
- ■ f'_c = 7 (ACI 318-08)
- × f'_c = 8 (ACI 318-08)
- − f'_c = 9 (ACI 318-08)
- ○ f'_c = 10 (ACI 318-08)

[a] *Units: inches*

Figure 7. h_{ef} / d ratios to reach a ductile failure in tension before the breakout strength of the concrete.

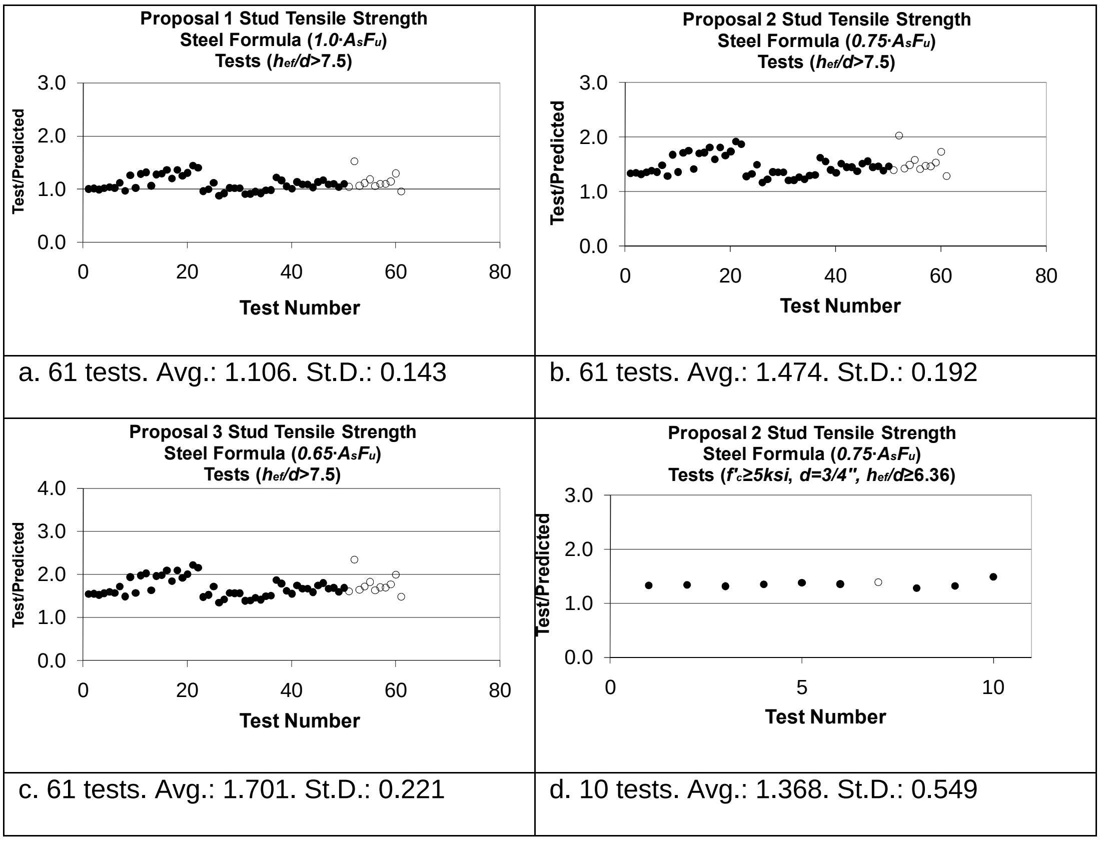
Table 5. Summary of test failures subjected to tension for several h_{ef} / d ratios.

	# Tests	**S. F.**[1]	**C.F.**[2]	**Comments**
ALL TESTS				
$h_{ef}/d > 5.5$	78	59	19	75.6% failed in the steel
$h_{ef}/d < 5.5$	144	0	144	100% failed in the concrete
$h_{ef}/d > 7.5$	61	50	11	82.0% failed in the steel
$h_{ef}/d < 7.5$	161	9	152	94.4% failed in the concrete
$h_{ef}/d > 9.5$	32	27	5	84.4% failed in the steel
$h_{ef}/d < 9.5$	190	32	158	83.1% failed in the concrete
NORMAL WEIGHT CONCRETE				
$h_{ef}/d > 5.5$	75	58	17	77.3% failed in the steel
$h_{ef}/d < 5.5$	144	0	144	100% failed in the concrete
$h_{ef}/d > 7.5$	58	49	9	84.5% failed in the steel
$h_{ef}/d < 7.5$	161	9	152	94.4% failed in the concrete
$h_{ef}/d > 9.5$	30	27	3	90% failed in the steel
$h_{ef}/d < 9.5$	189	31	158	83.6% failed in the concrete
LIGHTWEIGHT CONCRETE				
$h_{ef}/d > 5.5$	3	1	2	33.3% failed in the steel
$h_{ef}/d < 5.5$	0	0	0	-
$h_{ef}/d > 7.5$	3	1	2	33.3% failed in the steel
$h_{ef}/d < 7.5$	0	0	0	-
$h_{ef}/d > 9.5$	2	0	2	0% failed in the steel[3]
$h_{ef}/d < 9.5$	1	1	0	0% failed in the concrete

[1] *S.F.*: Steel failure (weld failures are included as steel failures).
[2] *C.F.*: Concrete failure
[3] The two tests in lightweight concrete with $h_{ef}/d > 9.5$ are within the limit of edge conditions of the breakout strength. The concrete strength is probably limited by the edge conditions in these tests.

As in the case for the normal weight concrete, a formula for the minimum value of h_{ef}/d required to ensure failure in the steel may also be deduced from the graphs presented in the Fig. 7(b) (ACI 318-08, average formula) as a function of the concrete strength and the steel anchor diameter for lightweight concrete. Equation 6 is thus derived from the average formulas Fig. 7(b).

$$h_{ef}/d = [(-1.48 f_c' + 241)d + (-27.6 f_c' + 4770)]/1000 \quad (\textit{Units: N, mm}) \qquad [6]$$
$$h_{ef}/d = (-0.26 f_c' + 6.13)d + (-0.19 f_c' + 4.77) \quad (\textit{Units: kips, inches})$$



a. 61 tests. Avg.: 1.106. St.D.: 0.143

b. 61 tests. Avg.: 1.474. St.D.: 0.192

c. 61 tests. Avg.: 1.701. St.D.: 0.221

d. 10 tests. Avg.: 1.368. St.D.: 0.549

Figure 8. Assessment of steel strength formulas with: a, b, c) ϕC_t=1, 0.75, and 0.65 for experiments having $h_{ef} / d > 7.5$; d) ϕC_t = 0.75, for experiments having $f'_c \geq 5$ ksi (35 MPa), d = 3/4 in. (19 mm), and $h_{ef} / d > 6.36$

A ratio of 9.55 (the maximum for the range of values shown in Figure 7) is obtained from Eq. 6 to ensure a ductile failure when f'c = 3 ksi (21 MPa) and d = 1 in. (25 mm). Using the 5% fractile formula of ACI 318-08 (Figure 7(d)), a ratio of 12 (the maximum for the range of values shown in Figure 7) is obtained to ensure a ductile failure when f'c = 3 ksi (21 MPa) and d = 1 in. (25 mm).

In the case of lightweight concrete, it is more difficult to conclude results directly from the experiments due to the scarce data found in the literature for tension tests (Table 5). The tests found in the literature are by McMakin (1973) whose test configuration may generate edge conditions for some of the large studs considered. The two tests with h_{ef}/d larger than 9.5 present an effective depth of 8 in. (203 mm) and a edge distance of 12 in. (305 mm). Both tests are thus right at the limit to permit the full development of the four-sided pyramid of CCD since the distance to an edge should be at a minimum 1.5 times the effective height. The concrete strength is thus possibly limited by the edge conditions in these tests. Therefore, no consistent conclusions may be derived from the experimental data.

As a conclusion, to avoid the breakout strength of the concrete, i.e., to be able to check only the steel formula to assess the tension strength of headed stud anchors, and considering the relationship between the overall height (h) and the effective height (h_{ef}) of the studs such as discussed above, h/d ratios of 8 and 10 (h_{ef}/d of 7.5 and 9.5) may be proposed for normal and lightweight concrete, respectively. In addition, the minimum ratio of the diameter of the head to the diameter of the shank should be taken as 1.63 either normal or lightweight concrete if only the steel formula is to be checked. Finally, edge effects and group effects (placing headed steel stud anchors too closely together) must be avoided for it to be adequate to only check the steel strength formula; this is addressed further below.

CHAPTER 4

MONOTONIC BEHAVIOR OF HEADED STUDS SUBJECTED TO SHEAR AND TENSION FORCES

For studying the interaction of tension and shear in headed steel stud anchors, McMakin et al. (1973) carried out 54 experimental tests and proposed an elliptical interaction equation with exponents on both the tensile and shear strength terms equal to 5/3 to predict the associated limit states. This formula is used in several codes, including ACI 318-08 (2008) and PCI 6^{th} (2004) to describe the behavior of anchors subjected to combined tension and shear (Figure 9).

Bode and Roik (1987) proposed a tri-linear equation as an approach to describe the behavior of anchors. One advantage of this model over an elliptical equation is that interaction need not be checked if the axial or shear force applied to the anchor is smaller than 20% of the ultimate strength in tension or shear, respectively.

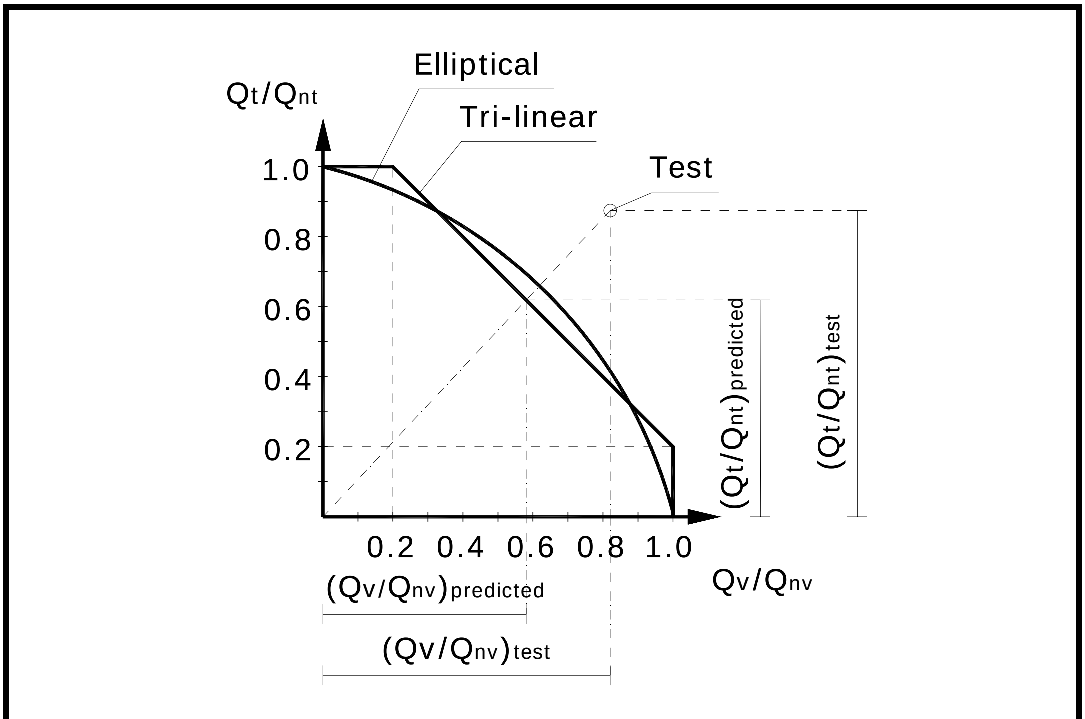
Saari et al. (2004) tested 2 specimens simulating the conditions in infill walls with two levels of confinement (see Figure 3), showing that additional confinement and better reinforcement detailing are important to provide ductile failure due to combined forces, just as for tensile forces.

4.1. Comparison of PCI 5^{th} and ACI 318-08 / PCI 6^{th} for combined shear and tension.

Equations found in the literature that predict the strength of an anchor subjected to shear and tension interaction are shown in Figure 9. The tri-linear and elliptical equations are adopted by the ACI 318-08 (2008) and PCI 6^{th} (2004) to predict the behavior of headed steel stud anchors subjected to combined tension and shear. The differences of the results obtained from the ACI 318-08 (2008) and PCI 6^{th} (2004) in the interaction equations are due to the shear strength prediction, since the tensile strength formulas match for both codes when uncracked concrete is assumed.

The radial ratio between the experimental and predicted values presented by Eq. 7 (Figure 9) is used to compare the experimental results with the predictions by the tri-linear and the elliptical approach.

$$Ratio = \frac{\sqrt{\left(Q_t / Q_{nt}\right)^2_{test} + \left(Q_v / Q_{nv}\right)^2_{test}}}{\sqrt{\left(Q_t / Q_{nt}\right)^2_{predicted} + \left(Q_v / Q_{nv}\right)^2_{predicted}}} \qquad [7]$$

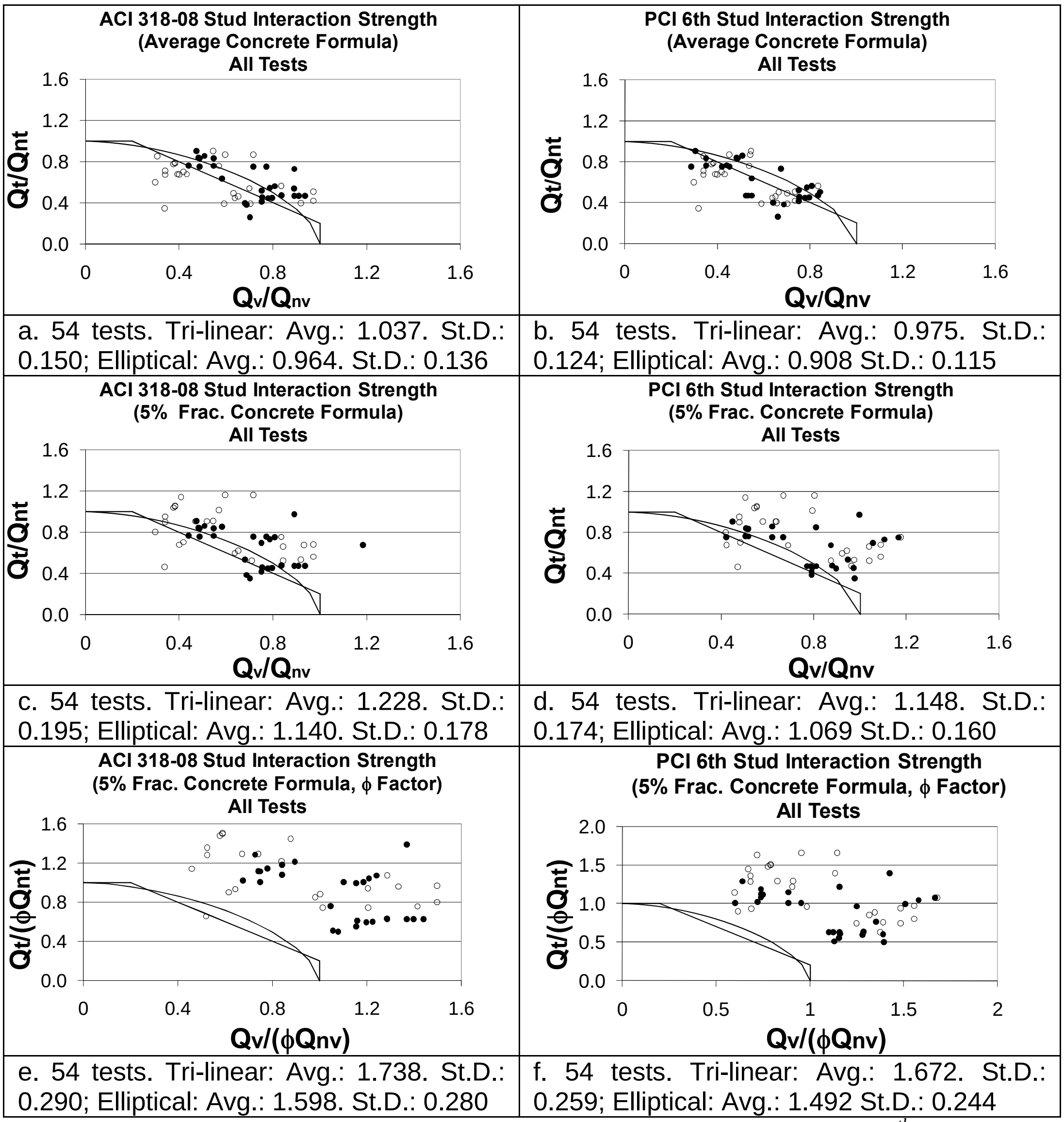


Author	*Equation*	*Comments*
Tri-linear Bode and Roik (1987)	$\left(\frac{Q_t}{Q_{nt}}\right)+\left(\frac{Q_v}{Q_{nv}}\right)\leq 1.2$	If $Q_v < 0.2Q_{nv}$ only check tension If $Q_t < 0.2Q_{nt}$ only check shear
Elliptical McMakin et al. (1973)	$\left(\frac{Q_t}{Q_{nt}}\right)^{\left(\frac{5}{3}\right)}+\left(\frac{Q_v}{Q_{nv}}\right)^{\left(\frac{5}{3}\right)}\leq 1$	

Figure 9. Equations to predict the strength of headed steel anchor subjected to interaction of shear and tension.

The assessment of the accuracy of ACI 318-08 and PCI 6th is carried out through the computation of nominal values provided by the standards (using the steel formula and 5% fractile formula for concrete). The nominal shear strength of concrete, Q_{nv}, in this work is predicted by pryout failure equations for the corresponding provisions as presented in Pallarés and Hajjar (2009). The nominal tensile strength of concrete, Q_{nt}, is predicted by the minimum strength of pull-out and breakout strength presented in this work assuming uncracked concrete.

A total of 54 tests have been found in the literature for interaction of tension and shear on headed steel stud anchors, neglecting tests that were heavily influenced by group effects due to two or more studs being closely grouped together. The 54 tests include 26 tests by McMakin et al. (1973), 2 tests by Saari et al. (2004) and 26 tests by Bode and Roik (1981). The configuration of the tests, the type of concrete and the range of concrete strength may be found in the Table 6.



a. 54 tests. Tri-linear: Avg.: 1.037. St.D.: 0.150; Elliptical: Avg.: 0.964. St.D.: 0.136

b. 54 tests. Tri-linear: Avg.: 0.975. St.D.: 0.124; Elliptical: Avg.: 0.908 St.D.: 0.115

c. 54 tests. Tri-linear: Avg.: 1.228. St.D.: 0.195; Elliptical: Avg.: 1.140. St.D.: 0.178

d. 54 tests. Tri-linear: Avg.: 1.148. St.D.: 0.174; Elliptical: Avg.: 1.069 St.D.: 0.160

e. 54 tests. Tri-linear: Avg.: 1.738. St.D.: 0.290; Elliptical: Avg.: 1.598. St.D.: 0.280

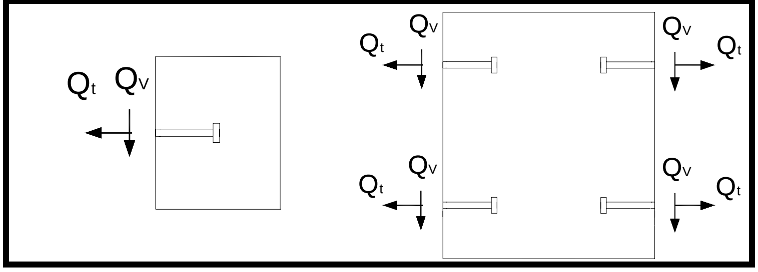
f. 54 tests. Tri-linear: Avg.: 1.672. St.D.: 0.259; Elliptical: Avg.: 1.492 St.D.: 0.244

Figure 10. Assessment of interaction strength of ACI 318-08 (2008) and PCI 6th (2004) using the minimum of the steel and concrete failure formulas for all tests.

From the comparison of experimental results between ACI 318-08 (Figure 10(a)) and PCI 6th (Figure 10(b)) using the average strength formulas, the accuracy of the elliptic approach proposed by McMakin at al. (1973) is seen to be better than the tri-linear equation. However, the tri-linear analysis presents the advantage of neglecting the interaction when the axial load is less than 20% of the nominal tension strength, as would be the majority of cases in composite structures. The application of 5% fractile formulas (Figure 10(c) and 10 (d)), and including resistance factors (Figure 10(e) and 10(f)), for the shear and tensile terms of the equation also indicates sufficient safety with both approaches. For example, ACI 318-08 presents the average ratios of 1.738 and 1.598 for the tri-linear and the elliptical equations, respectively, including the resistance factors.

Table 6. Test configurations for headed studs subjected to interaction forces

TYPES OF CONFIGURATION	
Type 1	***Type 2***



Refer.	***h_{ef}/d***	***# Tests***	***Type***	***# Studs***	***Concrete # Tests***	***Range of f_c' (ksi)***
McMakin et al.(1973)	5.33, 9.14, 9.33, 10.67	26	1	1 stud	Normal : 20 Lightweight: 6	4.90 – 7.48
Roik et al. (1981)	4.10, 4.29, 6.38	26	1	1 stud	Normal : 26 Lightweight: 0	3.20 – 7.20
Saari et al. (2004)	6.67	2	2	4 studs	Normal : 2 Lightweight: 0	4.00

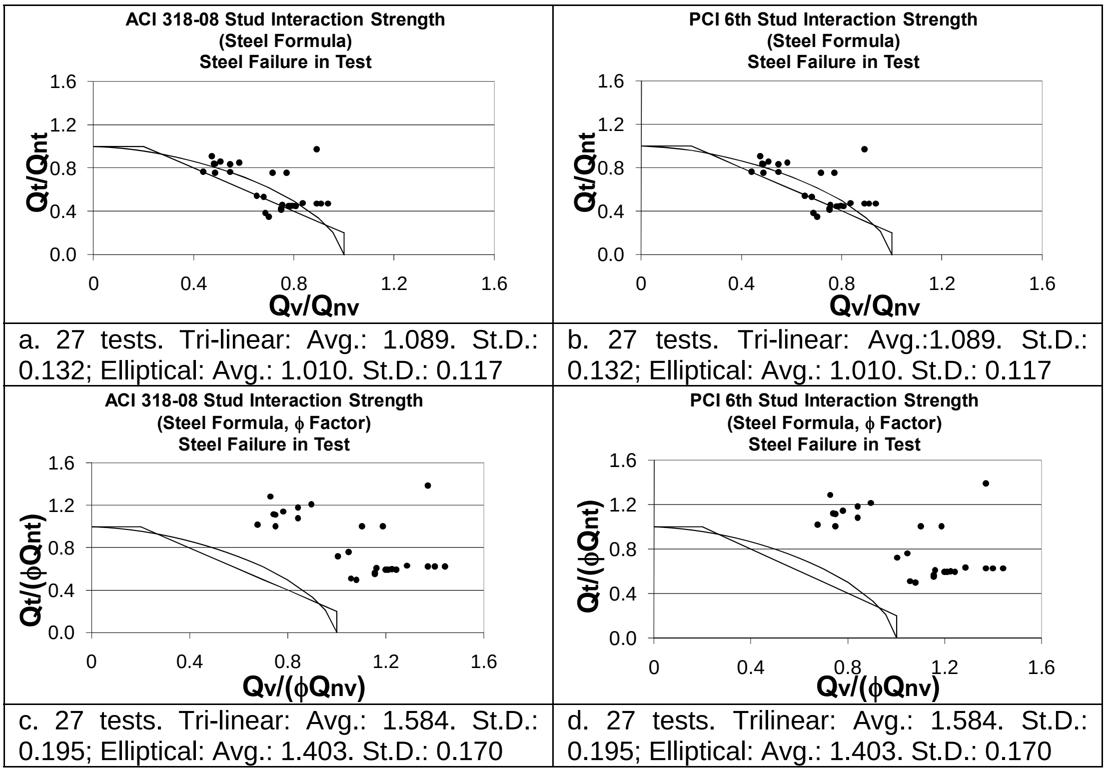
Tests failing within the steel shank or weld are well predicted by the steel formulas of ACI318-08 applied to the tri-linear equation, with a test-to-predicted ratio of 1.089 (Figures 11(a) and 11(b)), and the elliptic approach with a ratio of 1.010. Both the ACI 318-08 and PCI 6th approaches provide the same prediction with and without resistance factor (Figure 11(c) and 11(d)) since the equations and resistance factors coincide for the steel formulas.

Tests failing in the concrete (Figure 12) presented larger scatter than steel failures, with the formulas of PCI 6th (pryout for shear and the minimum of pull-out and breakout for tensile strength) more conservative than those of ACI 318-08.

In both ACI 318-08 and PCI 6th, the elliptical equation analyzing all tests, is more accurate than tri-linear, although the tri-linear equation permits to neglect interaction forces when the tension or shear force is smaller than 20% of the nominal strength value, respectively.

4.2. Headed steel stud interaction strength for $h_{ef} / d > 7.5$.

As for the case of headed steel stud anchors subjected either to tension or shear (Pallares and Hajjar, 2009), for composite construction it is advantageous to consider cases where a ductile failure mode typically controls and only the steel strength needs to be checked. A summary of failures found in the 54 tests of headed steel stud anchors subjected to interaction of shear and tensile is given in the Table 7. All tests have been classified as greater than or less than a given h_{ef}/d ratio of 5.5, 7.5, or 9.5, as was done above when investigating the behavior under tension. In the Table 7, a value of 7.5 for h_{ef}/d (i.e., a value of 8 for h/d as discussed above) may be proposed to check only the steel formulas in the elliptical equation since the 71.4% of the 14 tests with ratios larger than this limit failed in the steel.



a. 27 tests. Tri-linear: Avg.: 1.089. St.D.: 0.132; Elliptical: Avg.: 1.010. St.D.: 0.117

b. 27 tests. Tri-linear: Avg.:1.089. St.D.: 0.132; Elliptical: Avg.: 1.010. St.D.: 0.117

c. 27 tests. Tri-linear: Avg.: 1.584. St.D.: 0.195; Elliptical: Avg.: 1.403. St.D.: 0.170

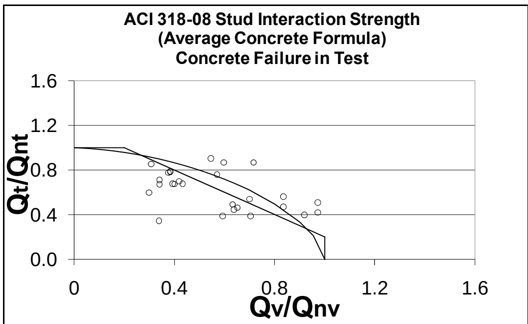
d. 27 tests. Trilinear: Avg.: 1.584. St.D.: 0.195; Elliptical: Avg.: 1.403. St.D.: 0.170

Figure 11. Assessment of interaction strength due to steel formula of ACI 318-08 (2008) and PCI 6th (2004) for tests that failed in the steel.

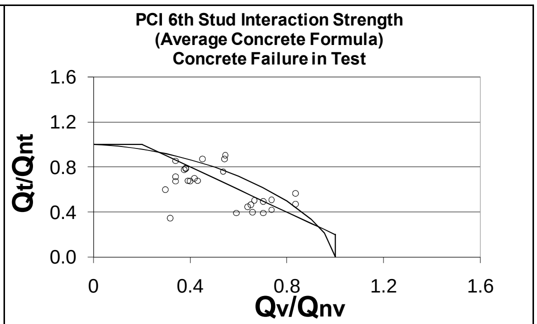
The results of considering the tests with $h_{ef}/d > 7.5$ are shown in the Figure 13(a) and 13(b). The use of steel formula alone provides more accurate but slightly unconservative results for the elliptical equation with an average of test-to-predicted ratio of 0.963 (Figure 13(a)). The tri-linear equation produces a mean test-to-predicted ratio of 1.241.

In order to predict the 28.6% of the tests that failed by the concrete with $h_{ef}/d > 7.5$, Figure 13 assesses the accuracy of using the steel formula without (Figure 13 (a)) and with (Figure 13(b)) the resistance factors obtained for tension (0.75) and shear (0.65). For simplicity, and because all the results are safe (i.e., the mean value of the test-to-predicted ratio is 1.782 and 1.385 for the tri-linear and elliptical equations, respectively) when the resistance factors are applied, the use of only the steel formula is reasonable if h/d is greater or equal to 8. Recommendations given for headed studs subjected to tension, regarding the ratio between diameter of the head and the diameter of the shank to avoid pull-out failure, and the distance to edges to avoid side blow out and premature breakout failure are applicable to studs subjected to tension plus shear.

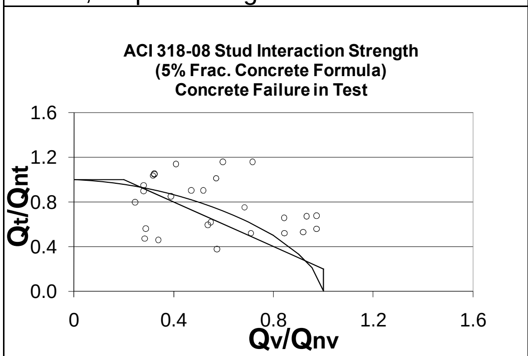
Figures 13(a) and 13(b) are based on assuming a constant, conservative, minimum value of 7.5 for the value of h_{ef}/d above which only the steel formula needs to be checked. However, as with the case of tensile loading applied to the stud, Eq. 5 can also be used to calculate this value. For example, if one again assumes that the minimum concrete strength in a composite component is 5 ksi (35 MPa) and the headed steel stud anchors are 3/4 in. (19 mm) in diameter, the value of h_{ef}/d above which only the steel formula needs to be checked is 6.36 from Eq. 5. Figure 13(c) then shows the mean test-to-predicted ratios for tests having h_{ef}/d greater than or equal to 6.36, with concrete strength greater than or equal to 5 ksi (35 MPa), shank diameters equal to 3/4 in. (19 mm), and using only the steel formula to check the strength of the headed steel stud anchors in interaction of shear and tension. The tests are predicted conservatively. This is again representative of the results of using Eq. 5 to determine the minimum value for h_{ef}/d above which only the steel formula needs to be checked.



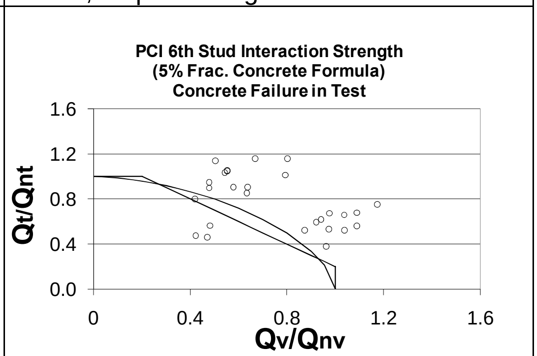
b. 27 tests. Tri-linear: Avg.:0.956. St.D.: 0.136; Elliptical: Avg.: 0.889. St.D.: 0.127



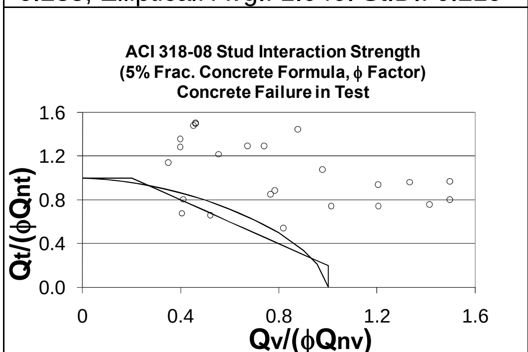
a. 27 tests. Tri-linear: Avg.: 0.989. St.D.: 0.166; Elliptical: Avg.: 0.922. St.D.: 0.154



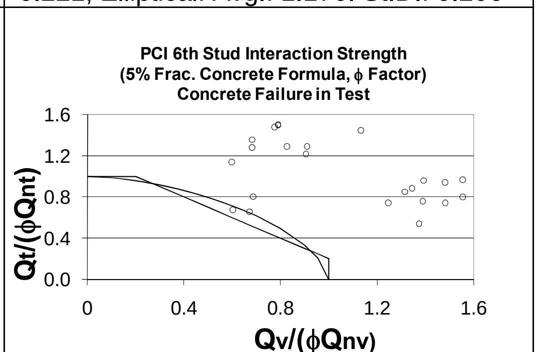
c. 27 tests. Tri-linear: Avg.: 1.103. St.D.: 0.235; Elliptical: Avg.: 1.040. St.D.: 0.219



d. 27 tests. Tri-linear: Avg.:1.265. St.D.: 0.221; Elliptical: Avg.: 1.176. St.D.: 0.206



e. 27 tests. Tri-linear: Avg.: 1.593. St.D.: 0.352; Elliptical: Avg.: 1.484. St.D.: 0.310



f. 27 tests. Tri-linear: Avg.:1.807. St.D.: 0.316
Elliptical: Avg.: 1.680. St.D.: 0.295

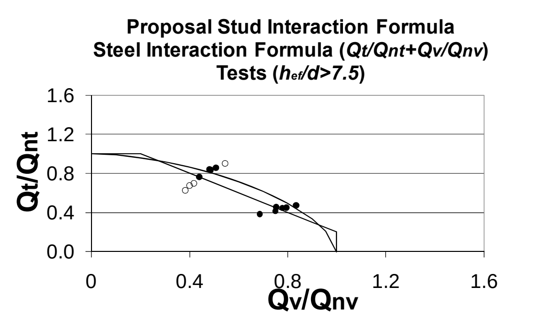
Figure 12. Assessment of interaction strength due to concrete formula of ACI 318-08 (2008) and PCI 6th (2004) for tests that failed in the concrete.

Table 7. Summary of test failures subjected to interaction for several h_{ef} / d ratios.

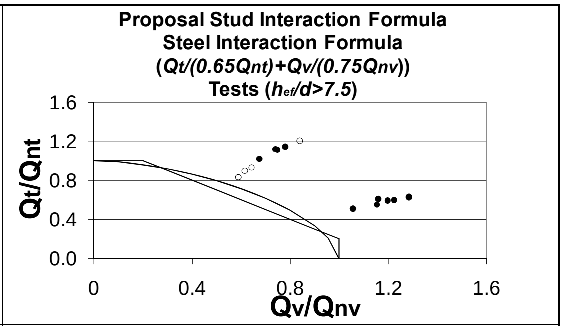
	# Tests	***S. F.***[1]	***C.F.***[2]	***Comments***
ALL TESTS				
$h_{ef}/d > 5.5$	22	17	5	77.2% failed in the steel
$h_{ef}/d < 5.5$	12	3	9	68.7% failed in the concrete
$h_{ef}/d > 7.5$	14	10	4	71.4% failed in the steel
$h_{ef}/d < 7.5$	14	4	10	57.5% failed in the concrete
$h_{ef}/d > 9.5$	4	2	2	50% failed in the steel
$h_{ef}/d < 9.5$	24	12	12	50% failed in the concrete
NORMAL WEIGHT CONCRETE				
$h_{ef}/d > 5.5$	10	8	2	87.5% failed in the steel
$h_{ef}/d < 5.5$	12	3	9	68.7% failed in the concrete
$h_{ef}/d > 7.5$	8	7	1	87.5% failed in the steel
$h_{ef}/d < 7.5$	14	4	10	57.5% failed in the concrete
$h_{ef}/d > 9.5$	0	0	0	-
$h_{ef}/d < 9.5$	22	11	11	50% failed in the concrete
LIGHTWEIGHT CONCRETE				
$h_{ef}/d > 5.5$	6	3	3	50% failed in the steel
$h_{ef}/d < 5.5$	0	0	0	-
$h_{ef}/d > 7.5$	6	3	3	50% failed in the steel
$h_{ef}/d < 7.5$	0	0	0	-
$h_{ef}/d > 9.5$	4	2	2	50% failed in the steel[3]
$h_{ef}/d < 9.5$	2	1	1	50% failed in the concrete

[1] *S.F.*: Steel failure (weld failures are included as steel failures).
[2] *C.F.*: Concrete failure
[3] The four tests in lightweight concrete with $h_{ef}/d > 9.5$ are in the limit of edge conditions of the breakout strength. The concrete strength is probably limited by the edge conditions in these tests

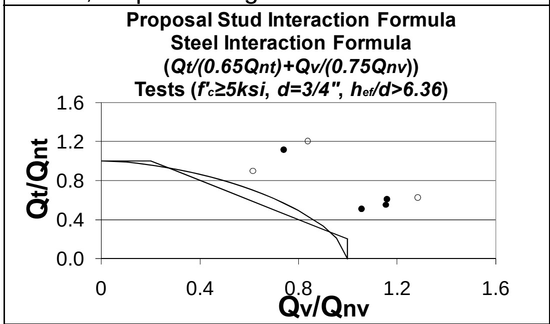
With so few experiments available for lightweight concrete subjected to combined tension and shear, it is not reasonable to propose checking only a steel strength interaction formula to ensure safe design. Headed steel stud anchors subjected to combined tension and shear with lightweight concrete should thus be checked for concrete failure modes as well (ACI, 2008).



a. 14 tests. Tri-linear: Avg.: 1.241. St.D.: 0.090; Elliptical: Avg.: 0.963. St.D.: 0.089



b. 14 tests. Tri-linear: Avg.:1.782. St.D.: 0.087; Elliptical: Avg.: 1.385. St.D.: 0.087



c. 7 tests. Tri-linear: Avg.:1.767. St.D.: 0.189; Elliptical: Avg.: 1.375. St.D.: 0.142

Figure 13. Assessment of steel interaction formula with: a, b) experiments having $h_{ef} / d > 7.5$; c) experiments having $f'_c \geq 5$ ksi (35 MPa), d = 3/4 in. (19 mm), and $h_{ef} / d > 6.36$

CHAPTER 5

EDGE CONDITIONS AND GROUP EFFECTS

ACI 318-08 (2008) and PCI 6th (2004) are based on the CCD approach for predicting tensile strength of a headed steel stud anchor in concrete, which assumes a four-side pyramid as a model for the planes of failure that occur in tension (Figure 1b). As this failure mode assumes a failure slope of 35º off the horizontal, the minimum distance to an edge that enables the development of the full strength of the four-side pyramid is $1.5h_{ef}$ (Figure 1b). In composite construction, if a stud is located in close proximity to a free edge or corner and loaded n tension, suitable reinforcement (e.g., based on using strut-and-tie procedures) should be provided to prevent the concrete from splitting or cracking, and the concrete strength provisions of ACI 318-08 (2008) Appendix D should be checked.

Similarly, when the distance between anchors is smaller than $3h_{ef}$, the planes of failure formed by the pyramids of neighboring anchors intersect and do not allow the development of the full concrete strength of each anchor (Figure 14). Use of Eqs. 5 and 6 may be used to assess an adequate value of hef for determining the minimum value of $3h_{ef}$ to avoid group effects. When the minimum required distance of $3h_{ef}$ to avoid group effects between anchors is too large for a composite element, the concrete strength should be checked as per ACI 318-08 (2008). In addition, reinforcement based on strut-and-tie models is proposed by ACI 318-08 Appendix D (2008).

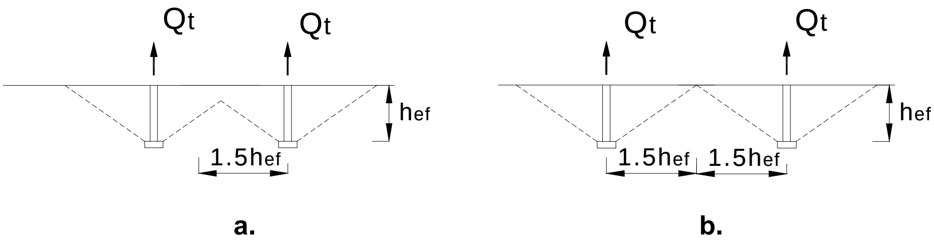


Figure 14. a) Group effects of anchors in tension; b) anchors without group effects.

While having an anchor of sufficient length increases the likelihood that the failure occurs in the steel, regardless of the surrounding reinforcement, it is always best for anchors in tension to provide supplemental reinforcement to help mitigate primary failure patterns in the concrete. This is especially important with shorter anchors subjected to tension. The commentary of ACI (2008) states the benefits of the supplementary reinforcement that may be used in the case of cast-in-place anchors.

Furthermore, several methods exist to increase the strength of short anchors that likely fail by the concrete, including:

- A reinforcement grid is often designed perpendicular to headed steel stud anchors in composite structures. The grid has a beneficial effect on the concrete failure surrounding the stud, increasing the strength up to 38% (Raposo et al., 2007).

➢ Circular plates may be added to increase the failure surface of the concrete pyramid failure plane (Figure 15(a)). Hawkins (1987) reported results showing that these plates can increase the breakout strength between 20% and 30%.

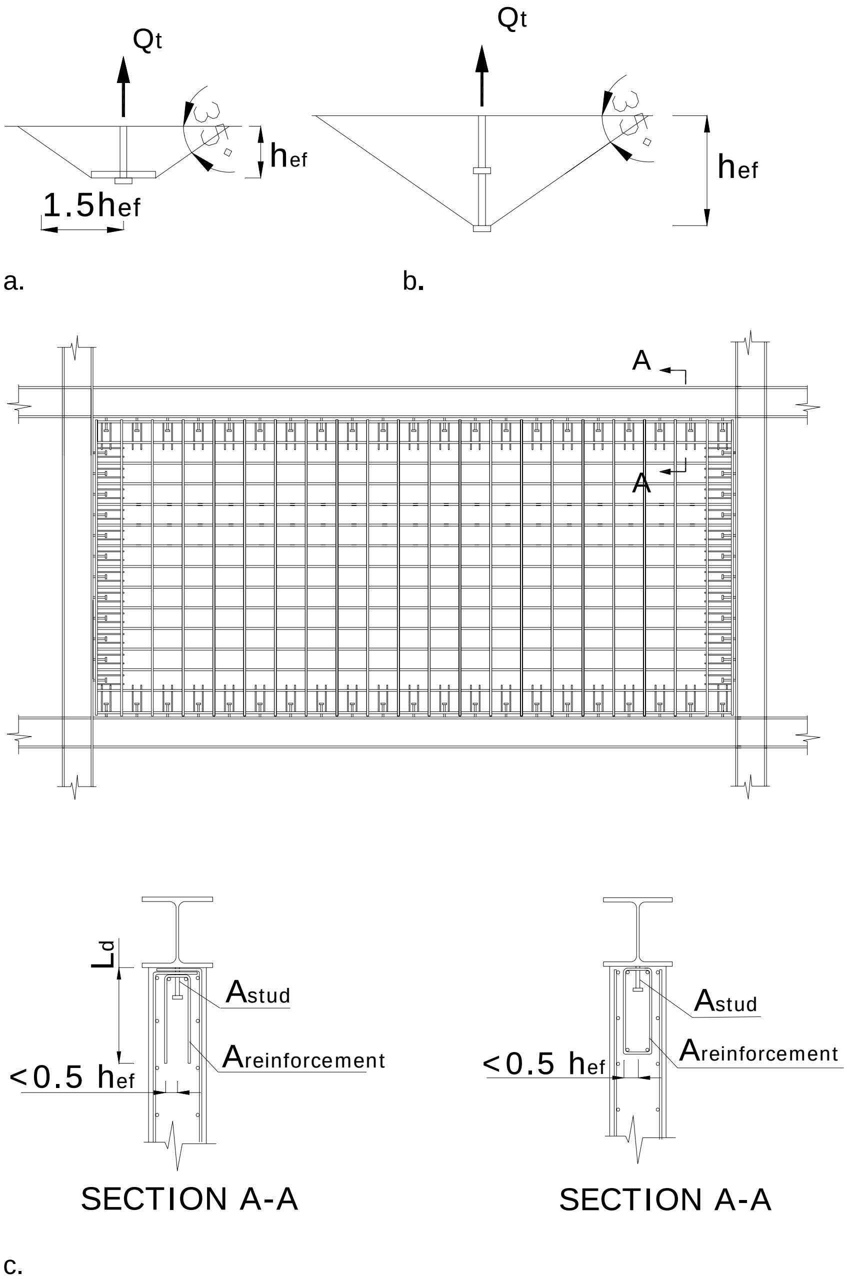


Figure 15. Details to improve the strength of concrete in tension: a) circular plates; b) welded studs increasing the effective embedment; c) supplementary reinforcement.

➢ Bode and Roik (1985) proposed a series of welded studs to increase the effective embedment as shown in Figure 15(b).

- Provide well-confined concrete surrounding the stud. Although more tests are needed to quantify the effect of confinement, the beneficial effect of confinement may be seen in the tests of Saari et al. (2004).

- Provide details such as shown in the Fig. 15(c) that avoid a failure on the breakout surface through the application of strut-and-tie models.

CHAPTER 6

SEISMIC BEHAVIOR OF HEADED STUD IN TENSION AND INTERACTION SHEAR AND TENSION

Most research on anchors subjected to cyclic (seismic) tensile loads have been on post-installed anchors, including Cannon (1981), Lindquist (1982), and Copley and Burdette (1985). Nonetheless, Usami et al. (1980) tested groups of cast-in-place anchors subjected to cyclic tension forces, analyzing the types of failures for effective embedment depths of 8.4 times the diameter of the shank, concluding that for shear forces a reduction in strength of approximately 20% and 30% is produced. However, Lindquist (1982) and Eibl and Kenitzeil (1989) researched tensile cyclic loads in post-installed anchors and stated that no reductions in strength were seen. Saari et al. (2004) stated that the greatest impact of cyclic loading was in the large reductions in the amount of displacement ductility. A reduction of 17% in shear strength reduction was seen due to cyclic shear loading in combination with the application of monotonic tension for headed steel stud anchors well confined by reinforcement, but they did not investigate the effects of cyclic tension specifically. Despite this past work, more studies are needed to determine the influence of reinforcement in the vicinity of the anchor, simulating conditions in composite structures, for cyclic tensile or cyclic tension plus shear loading on headed steel stud anchors.

CHAPTER 7

CONCLUSIONS

In this work, limit state formulas for headed stud anchors in tension and shear plus tension interaction specifically in composite construction have been assessed versus 222 monotonic and cyclic tensile experiments and 54 interaction experiments from the literature within the context of the AISC Specification (AISC, 2005a, 2005b), and comparisons have been made to the provisions in the ACI 318-08 Building Code (ACI, 2008) PCI Handbook, 6th Edition (PCI, 2004) and the PCI Handbook, 5th Edition (PCI, 1999). The experimental results are deaggregated to highlight tests that failed in the steel shank versus tests that failed in the concrete. The scope of this research includes composite beam-columns [typically concrete-encased steel shapes (SRC) or concrete-filled steel tubes (CFT)], concrete-encased and concrete-filled beams, boundary elements of composite wall systems, composite connections, composite column base conditions, and related forms of composite construction. Several conclusions can be drawn from this work:

- Conditions that lead to tension and combined shear and tension in headed steel stud anchors are becoming more prevalent, including applications such as infill walls, coupling beams, connections to composite columns, or composite column bases. A total of 222 experimental results on headed stud anchors under tension forces have been compared with both the *45 degree cone method* (PCI, 1999) and the *concrete capacity design* (*CCD) method* (PCI 2004; ACI, 2008) to predict the concrete failure mode. The *CCD* approach is seen to be more conservative and with lower scatter as compared to the *45 degree cone method.* From applying this method, a distance of $1.5h_{ef}$ to develop the full tensile strength provided by the CCD model of the four-sided pyramid delineating the concrete failure surfaces is necessary to avoid edge conditions reducing the tensile strength of the anchor in composite construction, This method, appropriate especially for computing the strength of short steel anchors, is used in this work as the basis for comparison with steel failures discussed below.

- Through a detailed review of headed steel stud anchors subjected to tension, the nominal strength formula $A_s F_u$ with a resistance factor of 0.75 with a reliability index β of 4 is confirmed as being adequate to determine the strength of headed stud anchors with sufficient enough embedment to reach a ductile failure.

- From the analysis of concrete failure modes in tension provided by the ACI 318-08 Appendix D and the 222 experimental results collected in this work, several requirements are deduced to ensure ductile failure in the steel for headed stud anchors subjected to tension or combined shear and tension, such that only the steel failure mode needs to be checked for conditions appropriate specifically for cast-i-place composite construction. Specifically, if one uses a minimum h_{ef}/d ratio equal to 7.5 (or, equivalently, h/d ratio equal to 8), a steel stud head diameter

$d_h \rangle 1.63d$, a minimum distance to any edge of $1.5h_{ef}$, and a minimum spacing between anchors of $3h_{ef}$, comparison with experimental results show that 82% of the tension failures occur in the steel shank, and those few tests that fail in the concrete are adequately predicted by the steel tensile strength equation, thus enabling simplified provisions to be used in which only the steel strength is checked for headed steel stud anchors subjected to tension. In addition, smaller values of h_{ef}/d (or h/d) may be used based on the values computed from in Eq. 5 as a function of concrete strength and anchor diameter.

- For lightweight concrete, the minimum ratio h_{ef}/d should be increased to 9.5 (i.e., a minimum h/d of 10) to reach a ductile failure in the steel for headed studs subjected to tension forces. Alternatively, smaller values of h_{ef}/d (or h/d) may be used based on the values computed from in Eq. 6 as a function of concrete strength and anchor diameter.

- From the 54 experimental results of headed steel stud anchors subjected to combined shear plus tension, it is seen the elliptical interaction equation is more accurate than tri-linear interaction equation to predict the strength. The application of corresponding resistance factors of 0.75 or tension and 0.65 for shear (Pallarés and Hajjar, 2009) provides safe results for all tests.

- If one uses a minimum h_{ef}/d ratio equal to 7.5 (or, equivalently, h/d ratio equal to 8), a steel stud head diameter $d_h \rangle 1.63d$, a minimum distance to any edge o $1.5h_{ef}$, and a minimum spacing between anchors of $3h_{ef}$, comparison with experimental results show that 71% of the interaction failures occur in the steel shank, and those few tests that fail in the concrete are adequately predicted by the steel strength interaction equation, thus enabling simplified provisions to be used in which only the steel strength is checked for headed steel stud anchors subjected to combined tension plus shear. In addition, smaller values of h_{ef}/d (or h/d) may be used based on the values computed from in Eq. 5 as a function of concrete strength and anchor diameter. Either the elliptical interaction equation or the tri-linear interaction equation presented in ACI (2008) may be used with the corresponding steel strength formulas; the elliptical equation is seen to provide more accurate results, while the tri-linear equation is more conservative for the tests studied in this work. There are too few tests with lightweight concrete subjected to combined tension and shear to validate checking on a steel strength formula.

- Edge conditions and group effects for headed studs in tension or interaction should be taken into consideration by checking concrete failure modes (ACI, 2008) when the distance between anchors is smaller than $1.5h_{ef}$ and $3h_{ef}$, respectively. Proper detailing reinforcement that mitigates failure in the concrete is always recommended in headed steel stud anchors subjected to tension or combined shear and tension to ensure ductile failure in the steel shank. Several recommendations and references relevant for composite construction are summarized for ductile detailing.

- More studies are necessary to improve the understanding of seismic behavior of headed stud anchors subjected to shear and combined tension and shear, particularly for cyclic loading. The few tests analyzed in this work confirm that the 25% reduction in the shear strength proposed by NEHRP may be adequate even for anchors subjected to combined tension plus shear.

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LYST OF SYMBOLS

A_s:	area of the headed stud anchor
A_{brg}:	bearing area (area of the head– area of the shank)
$Avg.(\mu)$:	Average
C_t:	coefficient for tension strengths
C_{crp}:	cracking coefficient (=1 for concrete assumed uncracked)
C.O.V.:	Coefficient of variation
d :	diameter of the shank of the stud anchor
d_h:	diameter of the head of the stud anchor
f_c':	specified compressive strength of the concrete
F_u:	specified minimum tensile strength of a stud shear connector
h:	height of the headed stud anchor
h_{ef} :	effective embedment depth of headed stud anchor
L_d :	development length of the reinforcement
N_b :	nominal concrete breakout strength of single anchor in tension in cracked concrete
$N_{b,avg}$:	average concrete breakout strength of single anchor in tension in cracked concrete
Q_{nv}	available shear strength of anchor
Q_{nt}	available tensile strength of anchor
Q_v	applied shear force in anchor
Q_t	applied tensile force in anchor
R_m / R_n	Average of the ratios between the test result and the predicted value
$St.D.$:	standard deviation
V_R :	Coefficient of variation of resistance
V_F :	Coefficient of variation on fabrication
V_P :	Coefficient of variation of R_m / R_n
V_M :	Coefficient of variation of materials
α :	Linearization approximation constant used to separate the resistance and demand uncertainties.
β:	Reliability index
ϕ_t:	resistance factor for tension loads
ϕ_v:	resistance factor for tension loads
$\psi_{c,P}$:	cracking modification factor for concrete pull-out strength. The value is 1.4 for non-cracked concrete
$\psi_{c,N}$:	cracking modification factor for concrete breakout strength. The value is 1.25 for non-cracked concrete
λ :	Modification factor for lightweight concrete

In the figures:

- ● steel failure
- ○ concrete failure

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