A Review of Headed-Stud Design Criteria in the Sixth Edition of the *PCI Design Handbook*

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The Precast/Prestressed Concrete Institute (PCI) is sponsoring a comprehensive research program to assess the shear capacities of groups of headed-stud anchors. This program was initiated in response to new provisions introduced in ACI 318–02, which were based on an extensive database dominated by results of post-installed anchor tests. Tests of headed-stud anchors loaded in shear, as used in precast concrete construction, are not prevalent in the literature. This test program, conducted by Wiss, Janney, Elstner Associates Inc., examines headed-stud connections in several geometric configurations and edge conditions. This paper provides a summary of the background studies and the research work that culminated in the design equations presented in Section 6.5 of the sixth edition of the PCI Design Handbook.

In the precast concrete industry, precast components are typically connected by use of an embedded plate, a majority of which are anchored with welded-headed studs. Welded-headed studs have been used to connect concrete components to other structural elements for decades. In fact, formal design concepts for headed-stud anchors have existed in the Precast/Prestressed Concrete Institute's (PCI) *PCI Design Handbook* since the early 1970s.[1]

Although concepts related to headed-stud design that account for multiple anchors, variable spacing, or anchors close to free edges have existed for years, recent design provisions in the American Concrete Institute's (ACI) *Building Code Requirements for Structural Concrete (ACI 318)* Appendix D[2,3] have raised questions about the older design models. Specifically, it raised questions about the PCI design model for headed-stud anchors, which has been used successfully since 1971. The PCI design model was adopted by ACI Committee 349, Concrete Nuclear Structures, in its publication.[4,5] This model is collectively known as the 45-degree-cone model. Testing and analytical studies in Germany in the 1980s led to the development of a design procedure for headed-stud anchors known as the Kappa method.[6] Additional refinements of the Kappa method produced the concrete capacity design (CCD) method, which is the basis of the ACI 318 Appendix D provisions.

In the mid-1990s, PCI initiated a headed-stud research program to create a capacity database for headed-stud group connections. This research program responded to industry concerns that the then-proposed provisions for ACI 318 Appendix D on headed-stud connection design were more conservative than the 45-degree model approach used in the *PCI Design Handbook*. Because the database for the CCD design procedures for tension and shear loading of headed studs were based on a limited amount of research data, the purpose of PCI's industry-sponsored research project was to satisfy two primary points:

- Provide justification for the PCI design procedures used in the past, which through ACI 318 implementation and adoption were now considered unconservative; and
- Create a database of test results to justify (a) accepting and conforming to the provisions of ACI 318-02 Appendix D, (b) modifying the ACI 318 procedures, (c) refining the design procedures as published in the fifth edition of the *PCI Design Handbook*,[7] or (d) writing a new design procedure independent of ACI 318, which is permitted in the code.

This paper examines the background of the ACI 318 Appendix D design provisions and how the provisions apply to headed-stud connections. It also reviews the evolution of the PCI design method through its various editions. Finally, the research work sponsored by PCI is summarized and the design provisions for headed-stud connections in the new sixth edition of the *PCI Design Handbook*[8] are presented.

The testing program and experimental work in deriving the equations has been presented in three papers to date.[9–11] Additional background information and test data on the various influences, to substantiate the *PCI Design Handbook* provisions, will be presented in detailed research papers in future issues of the *PCI Journal*.

ORIGINS OF ACI 318 APPENDIX D

In ACI 318-02, the design provisions for anchorage were finally codified into one document. The ACI 318 Appendix D provisions are based on the CCD method,[6] which was an adaptation of the original European Kappa method proposed by Eligehausen and Fuchs in the late 1980s. The refined Kappa method, the CCD method, was summarized in a paper by Fuchs, Eligehausen, and Breen in the *ACI Structural Journal*.[12] This method was molded into ACI 318 due to successful use of the design provisions in Europe. The European code had used CCD concepts for about 10 years prior to their incorporation into ACI 318-02.

Anchorage design requirements were placed in ACI 318 because a rational design method for post-installed anchors was needed. Post-installed anchors are those that are installed in hardened concrete. The need for these requirements was obvious: Several anchor manufacturers do business in the United States and, traditionally, the only source for the capacity of a given anchor was the manufacturers' catalogs.

Prior to ACI's codification of the design provisions, post-installed anchors were generally designed using manufacturers' catalogs and/or procedures and an accepted factor of safety, usually between 3 and 4. The tabular design values in the catalogs were based on standard concrete strengths and prescribed anchor patterns. In addition, some manufacturers provided additional design guidelines for edge distance and spacing effects; however, there was no uniform treatment of the design approach throughout the anchor industry on such topics. Likewise, designers could not easily make numerical comparisons among different manufacturers' designs. Visualizing the tension or shear behavior of an anchor was also not readily apparent by looking in a table of ultimate and safe working capacities.

The codification of the anchorage requirements was intended primarily for the post-installed anchorage market, but cast-in anchors, including headed studs and bolts, were also incorporated into the design provisions. These new provisions present a simple physical concrete breakout model, which can readily accommodate the effects of anchor spacing in two directions and the effects of edge conditions. Hence, the model and procedures provide the design engineer with the tools to design an anchor and readily consider many of the geometry and member influences that can affect its capacity.

The ACI 318 Appendix D provisions are not necessarily a one-size-fits-all design method; the behavior of post-installed and cast-in-place anchors can be different, however, especially given the individual actions of shear, tension, or some combination thereof, coupled with the variability of field installation versus plant production conditions. Because of the nature of past anchorage research in the United States and Europe, the CCD method was primarily based on a database dominated by post-installed-anchor test results. The work sponsored by PCI was an effort to expand the database of headed-stud test results in order to confirm the CCD method's applicability or provide new design guidelines that better fit the plant-cast headed-stud anchor behavior. Continuing this research will provide additional information to improve the reliability of headed-stud design.

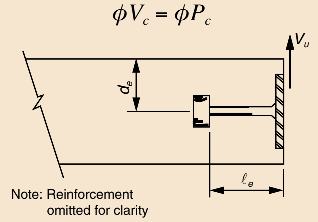
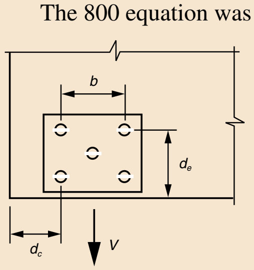
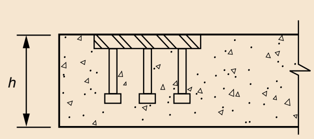
PCI'S RESEARCH INITIATIVE

The ACI design method was calibrated using an extensive database where most of the shear tests are on post-installed anchor test results. Upon thorough review of the published literature on headed studs and embedded bolts, the authors found many more results from tests performed on post-installed anchors loaded in shear compared with the number of tests performed on headed and post-installed anchors. Thus, there is a potential for the codified shear design method to be biased toward post-installed anchors. The database containing results from tension tests is more equally divided with regard to headed studs and post-installed anchors. Although there are no known systemic design problems with anchors designed using the pre-2002 procedures, it is fair to question the earlier design procedures given that the new and old design provisions yield different solutions. Consequently, the

Table 1. Review of Past *PCI Design Handbook* Requirements for Concrete Breakout Capacity

Handbook Edition (Year Published)	Basic Concrete Capacity Equation	Modification Factors	
First (1971)	$V_u' = \phi\left(2500d_e - 3500\right)$	None	
Second (1978)	$\phi V_c = 3250\phi\left(d_e - 1\right)\lambda\sqrt{\frac{f_c'}{5000}}$ and $\phi V_c \le \phi P_c = \phi A_o 4\lambda\sqrt{f_c'}$	None	
Third (1985)	Away from an edge $\phi V_c = \phi 800 A_b \lambda\sqrt{f_c'}$ Near a free edge $\phi V_c = \phi 2\pi d_e^2 \lambda\sqrt{f_c'}$	None	
Fourth (1992)	Away from an edge $\phi V_c = \left(\phi 800 A_b \lambda\sqrt{f_c'}\right)n$ Near a free edge $\phi V_c = \phi V_c' C_w C_t C_c$ $\phi V_c' = \phi 12.5 {d_e}^{1.5} \lambda\sqrt{f_c'}$	See below	
Fifth (1999)	$\phi V_c = \phi V_c' C_w C_t C_c$ with $\phi V_c' = \phi 12.5 {d_e}^{1.5} \lambda\sqrt{f_c'}$	Thickness $C_t = \left(\frac{h}{1.3d_e}\right) \le 1.0$ Group width $C_w = \left(1 + \frac{b}{3.5d_e}\right) \le n_s$ Corner $C_c = \left(0.4 + 0.7\frac{d_c}{d_e}\right) \le 1.0$	

Notes: b = center-to-center distance between the outermost studs in the back row of the group (in.); d_c = the distance, measured perpendicular to the direction of the load, from the free edge of concrete to the center line of the nearest stud (in.); d_e = distance from free edge of concrete to back row of studs in direction of load (in.); h = thickness of the concrete member (in.); n_s = number of studs in the back row; λ = concrete unit weight factor (1.0 for normal weight, 0.85 for sand lightweight, 0.75 for all lightweight); and ϕ = 0.85 (strength reduction factor).

	Comments
	Because a shear cone failure has been observed in shear tests: $\phi V_c = \phi P_c$ 
	From the *PCI Design Handbook*: For groups of studs, the design shear strength, based on concrete strength, should be taken as the least of: • Strength of the weakest stud, based on the above equations, times the number of studs, • Strength based on the d_e of the weakest row of studs times the number of rows, or • Strength based on the d_e of the row of studs farthest from the free edge. Note: These are based on 9 "normal" arrangement of studs. For arrangements that are very unsymmetrical or unusual, a separate analysis, which considers the "zipper" effect, should be made.
	The 800 equation was eliminated from the fifth edition.  

same provisions for cast-in anchorages and post-installed anchorages may not be appropriate.

PCI initiated research work on a large-scale testing program to expand the database of headed-stud test results. Phase 1 of this study examined headed-stud anchors loaded in shear only. The Phase 1 testing program included 364 tests of headed studs loaded in shear. In the comprehensive program, the shear research variables included front-edge distance, corner conditions, side-edge distance, rear-edge distance, and in-the-field-type connections. Phase 2 of this study included a literature review of the tension-only test database compiled for headed-stud connections. In addition to tension behavior, Phase 2 also included some experimental studies to evaluate the effects of combined tension and shear on a headed-stud connection. In the Phase 2 testing, 69 tests were conducted that subjected headed-stud groups to different combinations of tension and shear. Wiss, Janney, Elstner Associates Inc. (WJE) conducted both of these project phases in its Northbrook, Ill., structural laboratories.

While there have been no failures or serious design procedure problems reported that can be attributed to the use of the design methods published in previous editions of the *PCI Design Handbook*, modifications to the welded, headed-stud design procedures were made in the sixth edition in response to the new test data available through this research program.

BACKGROUND

Historical Perspective

Concrete breakout behavior, as a function of edge distance, has been part of anchorage design for a number of years. Reference should be made to **Fig. 1**, which illustrates a typical headed-stud anchorage connection and the geometric terminology used. For a majority of cast-in-place anchorage designs subject to shear, the front-edge distance d_{e3} is the most critical factor governing the connection behavior and strength. However, in precast concrete framing connections, consideration of the corner and side-edge distances is also necessary.

Discussions of the equations used to predict concrete breakout capacity over the past 30 years correspond well to the provisions contained in the first five editions of the *PCI Design Handbook*. The progressive development of the concrete breakout equations for headed-stud anchors in the United States and Europe is reflected in the various *PCI Design Handbook* editions. Thus, the handbook provisions form an excellent basis to introduce and review the previous work, research, and design philosophies for concrete breakout capacities.

Since its inception in 1971, the *PCI Design Handbook* has recognized the need for headed-stud connections in precast concrete design. The first five editions of the handbook incorporated design guidelines for headed-stud connections, with improvements to the design equations made in successive editions. Aside from design information published by manufacturers, the information contained in the handbook represents some of the only recognized and published informa-

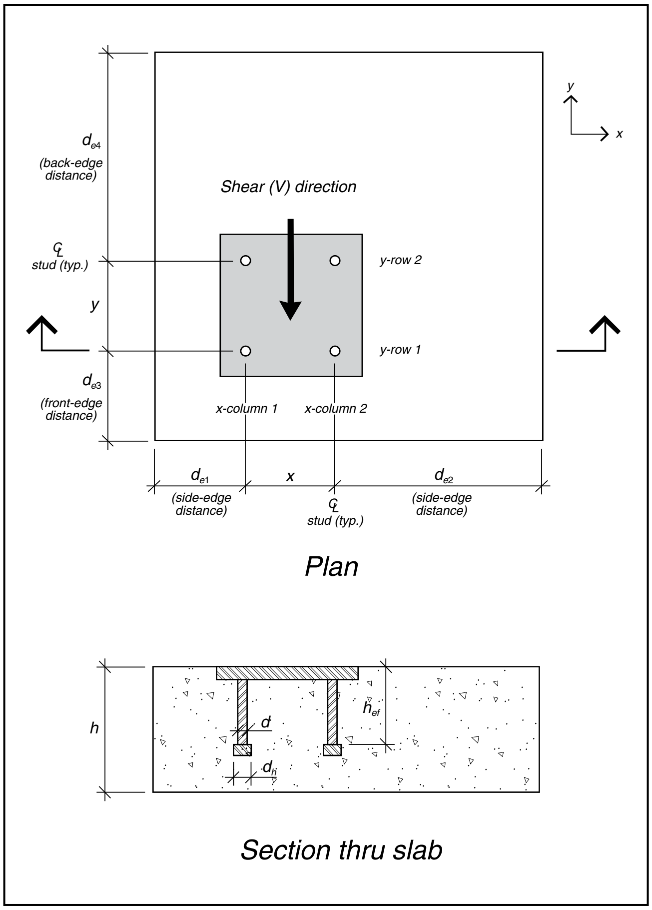


Fig. 1. Geometric terminology used to define a headed-stud anchorage.

tion on this subject during the 1970s and 1980s. During this time, design equations for concrete breakout capacity were included in an ACI Committee 349 report and in the Uniform Building Code.[13] As addressed earlier, however, only recently has a comprehensive design method for anchorage to concrete been incorporated into the widely accepted ACI 318 *Building Code Requirements for Structural Concrete*.[3]

Table 1 presents the concrete shear breakout design equations provided in the first five editions of the *PCI Design Handbook*. These equations are applicable for concrete breakout when the anchorage is loaded in shear perpendicular to a free edge. As shown in Table 1, the first three editions of the handbook provided no modifiers to the basic concrete capacity for groups or spacing because there was a lack of research on these influences at that time. Research at Oklahoma State University (OSU) and in Europe brought about three modification factors that were included in the fourth and fifth editions of the handbook.[14,15] These modification factors to the basic concrete breakout capacity accounted for corner, thickness, and group-spacing effects.

In the first edition of the *PCI Design Handbook*, the basic equation used to calculate concrete breakout when subject to shear was an extrapolation of a breakout equation developed by Superior Concrete Products in the late 1960s.[16] Superior was a predecessor to Dayton Superior, previously known as Dayton Richmond. This equation was developed from in-house research on cast-in-place specialty concrete inserts that Superior Concrete Products produced for the precast concrete industry. As shown in Table 1, this early equation was quite simple. The only variable was the distance to the free edge. Concrete strength, headed-stud area, and diameter were not included as variables in this equation.

The concrete-breakout equation contained in the second edition of the *PCI Design Handbook* originated from work at Lehigh University that was sponsored by Nelson Stud Welding.[17] This equation is only a slightly modified version of the one developed by McMackin, Slutter, and Fisher (the PCI equation has added a lightweight concrete factor λ).[18] Also, the McMackin, Slutter, and Fisher equation was apparently modified or normalized with respect to a 5000 psi concrete, thus eliminating the $\sqrt{f_c'}$ term from the V_c equation.

In addition to checking the basic concrete shear capacity defined by ϕV_c, a secondary check was required in the second edition of the handbook to ensure a tensile pullout (actually meaning tension concrete breakout) failure did not occur. Therefore, ϕV_c was to be checked against the tensile capacity ϕP_c. This edition of the handbook notes that this check was instituted because cone-type failures had been observed in some shear tests, probably meaning the shear failure was what we now call a concrete pryout failure.

The third edition of the *PCI Design Handbook* saw the introduction of the 45-degree–cone breakout model incorporated into the shear design provisions.[19] This model assumed a semi-conical failure surface defined by a height equal to the edge distance and a 45-degree projected concrete breakout surface to the free edge. The design equation included the effect of edge distance squared and the square root of the concrete compressive strength. Originally proposed by the Tennessee Valley Authority (TVA) in a design standard,[20] this equation was adopted by ACI Committee 349 in the late 1970s. In 1982, Klingner, Mendonca, and Malik reviewed the literature on shear capacity of short anchor bolts and headed studs.[21] Their work confirmed the applicability of this equation. These three sources also note that the shear equation is applicable for fully embedded studs.

The third edition of the *PCI Design Handbook* contained verbiage on computing the capacity of stud groups, which is reproduced in Table 1. Guidelines were presented in this edition for determining the stud-group capacity based on the weakest stud, weakest row, or row farthest from the free edge. Also, caution was given that unusual arrangements of studs should be analyzed to prevent zipper-type failures. Although this edition did not give guidance as to the nature of this type of failure, the verbiage implies a sequential-type failure based on the critical studs in an unusual arrangement. The design engineer could garner additional guidance on the zipper effect from the construction/design booklet *Embedment Properties of Headed Studs*,[22] published by TRW Nelson. **Figures 2** and **3** reproduce the relevant figures from this publication, showing the interaction of a stud group.

For anchors away from an edge,[23] the third edition introduced the 800 coefficient shear capacity equation proposed by Shaikh and Yi. This equation was a modification of an original relationship developed by Ollgaard, Slutter, and Fisher with proposed simplifications by Martin and Korkosz and later by Shaikh and Yi.[24,25]

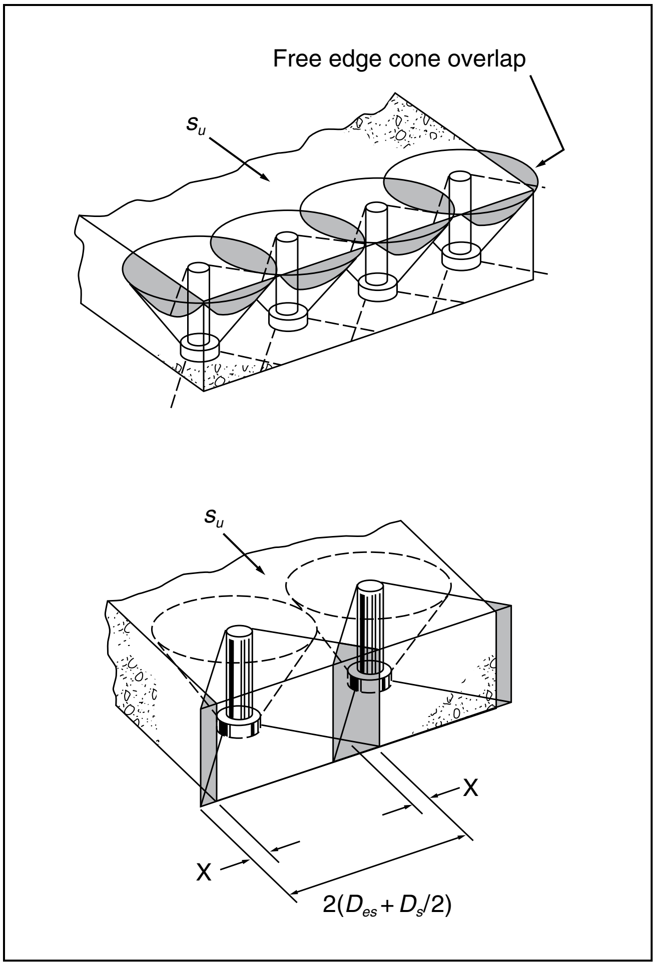


Fig. 2. The influence of overlapping failure cones in shear.[22]

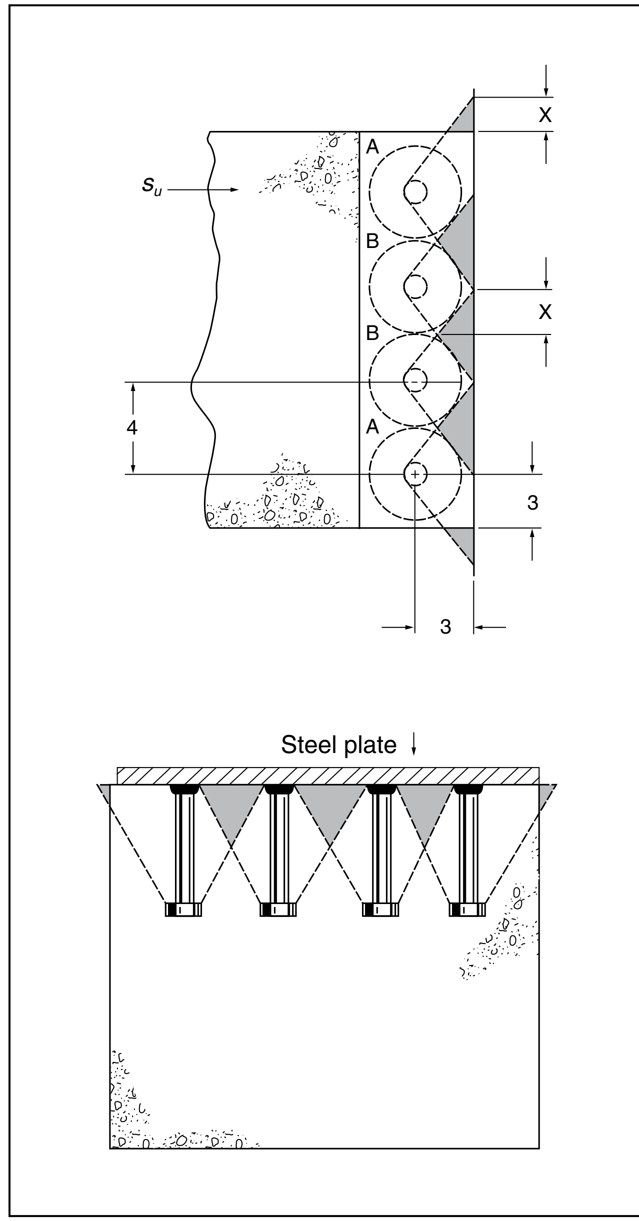


Fig. 3. A zipper-type breakout for a stud-group anchorage.[22]

The fourth edition of the *PCI Design Handbook* did not change the third edition's concrete shear breakout capacity equation when the anchorage was far from a free edge.[26] That is, the equation with the 800 coefficient was reserved for studs located far from a free edge. This equation was intended to apply specifically to edge distances greater than or equal to 15 times the stud diameter, that is $d_e \geq 15d_b$. The fifth edition did not include this equation, under the premise that the equation may not fully represent concrete capacity when the edge distance was greater than $15d_b$ and because of an apparently incomplete understanding of the behavioral origins of the equation.

Both the fourth and fifth editions of the *PCI Design Handbook* reflected new information on the concrete shear breakout capacity near a free edge. Based somewhat on the European experience, Shaikh proposed the basic, empirically derived equation for calculating concrete shear breakout capacity ϕV_c near an edge as:

$$\phi V_c = \phi 12.5\left(d_e\right)^{1.5}\sqrt{f_c'} \qquad \text{(lb)} \qquad \text{(Eq. 1)}$$

This empirical equation represented a lower bound capacity equation, derived from test data with concrete compressive strengths f_c' in the 4000 psi to 5300 psi (27.6 MPa to 36.5 MPa) range, and accounted for the distance to the front free edge d_e. This equation was also based on the analysis of headed stud and cast-in, ASTM A307, anchor bolt test data.[27]

For an anchor group, the edge distance term d_e was based on the rear stud row, as shown schematically in the fifth edition entry of Table 1. Research conducted at OSU by Cruz and Wong on groups of stud anchors confirmed that the rear stud row,[14,15] or stud row farthest from the free edge, controlled the ultimate concrete breakout surface of the assembly and, hence, the capacity. Rong and Fafitis also observed this behavior for headed studs in PCI-sponsored research work at Arizona State University.[28]

Prior to the fourth edition of the *PCI Design Handbook*, German researchers Eligehausen and Fuchs developed the Kappa (κ) method to define the concrete shear breakout capacity near a free edge. This equation was derived from the analysis of data on headed, expansion, and bonded anchors. The latter two anchor groups are post-installed. Their original average concrete shear breakout capacity equation V_c in SI units was:

$$V_c = 1.3\sqrt{d_b}\sqrt{f_c^{'}}\left(c_1\right)^{1.5} \quad \text{(N)} \qquad \text{(Eq. 2)}$$

where

d_b = diameter of the anchor (mm)

$f_c^{'}$ = concrete cube compressive strength (N/mm^2)

c_1 = edge distance (mm)

The authors note that this equation is limited to anchors with embedment depths ranging from $4d_b$ to $8d_b$ and to those embedded in slabs where there were no thickness effects, as addressed in the state-of-the-art report from the CEB.[29] Similar limits are associated with the current code language of ACI 318-05 Appendix D. Moreover, the equation is based on concrete compressive strengths in the range of 1740 psi to 6960 psi (15 MPa to 60 MPa) cube strength.

The Kappa equation (Eq. 2) originally had a dimensionless term of $(h_{ef}/d_b)^{0.2}$ as a multiplier on the right side. It was simplified to its form in Eq. 2 by assuming h_{ef} (the effective stud embedment length) is approximately $4d_b$, thus including the $(h_{ef}/d_b)^{0.2}$ term as the 1.32 in the constant coefficient. If the h_{ef}/d_b term is included in the Kappa equation, limits are placed on the embedment depth such that $4d_b \leq h_{ef} \leq 8d_b$.

The conversion of this average equation to English units (and with concrete cube strengths assumed to be about 1.25 times the concrete cylinder strengths) becomes:[30]

$$V_c = 17.5\sqrt{d_b}\sqrt{f_c^{'}}\left(c_1\right)^{1.5} \quad \text{(lb)} \qquad \text{(Eq. 3)}$$

where

d_b = diameter of the anchor (in.)

$f_c^{'}$ = concrete cylinder compressive strength (psi)

c_1 = edge distance (in.)

By substituting a $^1/_2$-in.-diameter (12.7 mm) stud into Eq. 3 for d_b, the resultant average equation becomes similar to the fourth edition PCI equation (Eq. 1).

There has been considerable debate by ACI committees as to the proper anchor row to consider when examining a multiple-row, headed-stud connection. Early versions of the CCD method used the front row to compute the area breakout factors. The shear capacity computed was then doubled for a second, back row. However, this computational model conflicts with actual observed behavior and is not always a good capacity predictor for headed-stud connection groups.

The OSU test results showed that multirow (front and back rows) anchor group connections loaded in shear exhibit a behavior consistently indicating that the front stud row was ineffective and not part of the concrete failure surface. This behavior was repeated in testing at the University of Wisconsin–Milwaukee and Arizona State University.[28,31] The failure crack surface always propagated through the back stud row and then forward at an angle toward the free edge.

All of these test results, and in particular the OSU stud group tests, show that a headed-stud group connection has the ability to redistribute the applied load through plastic distribution. The front-row studs are ineffective because of the anchorage plate rigidity. Consequently, the back-row studs dictate the strength in concrete breakout. The rigidly attached connection plate distributes the shear load in accordance with the relative stiffness of each headed stud, though at ultimate, the breakout crack is concentrated at the back row. This design philosophy has been reflected as early as the fourth edition of the *PCI Design Handbook*.

Anchorage Design Guidelines

An important factor in the performance of headed studs, when their design is governed by concrete capacity, is the confinement of the failure area with reinforcement. In shear, design capacity and ductility can be increased with such reinforcement, likewise in tension. It has been recommended in the fourth, fifth, and sixth editions of the *PCI Design Handbook* that reinforcement be placed to cross failure planes around headed-stud anchors. However, the design provisions presented in the handbook represent a lower bound on capacity, determined by the capacity at first cracking in an unreinforced member. Providing reinforcement can augment the anchorage capacity; however, this load-carrying mechanism requires a separate design that develops reinforcement beyond postulated failure planes. In some cases, that is difficult to detail properly.

Welded, headed studs are designed to resist direct tension, shear, or a combination of the two. The design equations given in the sixth edition of the *PCI Design Handbook* are applicable to studs that are welded to steel plates or other structural members and embedded in unconfined concrete. It is assumed that the steel plates are of sufficient thickness to prevent significant plate deformation and to adequately transfer applied load to and between the studs.

Where feasible, headed-stud connections should be designed and detailed such that the connection failure is precipitated by failure (typically defined as yielding) of the stud material rather than failure of the surrounding concrete (unless reinforcement crosses the concrete failure surface). Generally, the in-place strength of the anchor group should be taken as the smaller of the design values based on concrete and steel. This requirement necessitates the computation of individual steel and concrete capacities in all cases. Unfortunately, with so many variables affecting concrete capacity, each connection type and configuration will have a unique capacity. For this reason, for shear loading it is impossible to globally define the edge distance where an anchor group failure mode transitions from concrete to steel.

The New Section 6.5

Anchorage design provisions in the sixth edition of the *PCI Design Handbook* are the result of a combination of the WJE research, provisions included in past editions, and the ACI 318 provisions. In light of the end user of the *PCI Design Handbook*, the provisions contained therein are geared toward headed-stud design. Caution should be exercised in the application of these provisions to post-installed anchor design. The new provisions are postulated to uncracked concrete, which is typical in precast concrete products, with reduction modifiers for instances of cracked concrete. This philosophy is opposite that of ACI, where cracked concrete is considered typical.

The information provided in this paper details the background information used to develop the sixth edition of the *PCI Design Handbook* provisions, as outlined in the following sections. Specific individual section background is also provided in an effort to offer explanatory information (commentary) to the handbook provision philosophy.

STEEL MATERIALS

Minimum Plate Thickness

Conventional carbon steel used for anchorages should conform to the minimum requirements of ASTM A36 for plates or ASTM A992 for shapes.[32,33] Stainless steel plates shall conform to the minimum requirements of ASTM A666,[34] Type 304 or 316. Other steel types can be used, but their applicability to the stud welding process should be verified. The minimum plate thickness t_p to which studs are attached should be:

$$t_p \geq \frac{1}{2} d_0 \qquad \text{(Eq. 4, handbook Eq. 6.5.1.1}^8\text{)}$$

where

d_0 = the stud diameter (in.)

This provision is a carryover from several past editions of the *PCI Design Handbook* and is based on the research of Goble at Case Western Reserve University.[35] Increased plate thickness may be required for bending resistance or to ensure a more uniform load distribution to the attached studs. Perry, Funk, and Burdette provide more information on plate stiffness.[36]

Headed-Stud Properties

The Structural Welding Code, AWS D1.1-04, has recognized that mild steels conforming to ASTM A108 (Grades 1010 through 1020) and used for headed studs have increased material properties.[37,38] **Table 2**, adapted from Table 7.1 in AWS D1.1-02,[39] shows the current minimum tensile strength F_{ut} and yield strength F_y for Type B studs to be 65 ksi and 51 ksi (450 MPa and 350 MPa), respectively, which is incorporated into the present *PCI Design Handbook* provisions, as Table 6.5.1.1. These material property values have slightly increased from those listed in the fifth edition.

Currently, AWS classifies Type B studs as those that are headed, bent, or of other configuration; in $^1/_2$, $^5/_8$, $^3/_4$, $^7/_8$, and 1 in. diameters (13, 16, 19, 22, 25 mm); and used as an essential component in composite beam design and construction. These stud diameters represent the majority of those also used in precast concrete construction.

Type A studs cover the $^1/_4$- and $^3/_8$-in.-diameter (6 and 9 mm) stud sizes used occasionally in precast concrete construction. As shown in Table 2, Type A studs currently have a 61 ksi (420 MPa) minimum tensile strength F_{ut} and a 49 ksi (340 MPa) minimum yield strength F_y. AWS defines Type A studs as "general purpose of any type and size used for purposes other than shear transfer in composite beam design and construction."

Stainless steel studs can be welded to either stainless or mild carbon steel. Fully annealed stainless steel studs are recommended when welding stainless-steel studs to a mild carbon steel base metal. Using annealed stainless-steel studs has been shown to be imperative for welding to carbon steel plates subject to repetitive or cyclic loads. In such cases, stress corrosion failure in the weld can occur,[40] and use of the annealed stud minimizes the chance of weld cracking and failure. Consult the headed-stud supplier to obtain additional information on stainless-steel stud use and availability.

Steel Stud Capacity

As presented in an earlier paper,[9] the design ultimate shear or tensile strength governed by steel failure can be expressed by:

$$\phi V_s = \phi N_s = \phi_s n_{studs} A_s F_{ut} \qquad \text{(Eq. 5, handbook Eq. 6.5.2.1}^8\text{)}$$

where

ϕ = 0.65 (steel capacity reduction factor for studs in shear)

= 0.75 (steel capacity reduction factor for stud in tension)

N_s = nominal tensile strength of an anchorage based on steel capacity (kip)

V_s = nominal shear strength of an anchorage based on steel capacity (kip)

n_{studs} = number of headed studs in the anchorage

Table 2. Minimum Mechanical Property Requirements for Headed Studs (from AWS D1.1-02 Table 7.1)[39]

Property (Diameters)	Type A ($^1/_4$ to $^3/_8$ in.)	Type B ($^1/_2$ to 1 in.)
Tensile strength F_{ut} (min.)	61,000 psi	65,000 psi
Yield strength F_y (0.2% offset)	49,000 psi	51,000 psi
Elongation (min. % elongation in 2 in.)	17%	20%
Reduction of area (min.)	50%	50%

Note: 1 in. = 25.4 mm; 1 psi = 6.895 KPa.

A_s = nominal area of the headed stud (in.^{2})

F_{ut} = minimum design ultimate tensile strength of the stud steel (ksi)

= 65 ksi for normal Type B headed studs used in precast concrete anchorages

ACI 318 Appendix D provisions place a lower steel capacity reduction factor (ϕ = 0.65) on the steel shear strength than when loaded in tension (ϕ = 0.75). Section RD.4.4 of ACI 318 states:

"The ϕ factors for steel strength are based on using f_{uta} to determine the nominal strength of the anchor (see D.5.1 and D.6.1) rather than f_{ya} as used in the design of reinforced concrete members. Although the ϕ factors for use with f_{uta} appear low, they result in a level of safety consistent with the use of higher ϕ factors applied to f_{ya}. The smaller ϕ factors for shear than for tension do not reflect basic material differences but rather account for the possibility of a non-uniform distribution of shear in connections with multiple anchors."

The authors believe that this factor is too restrictive for headed studs welded to a plate, and a factor ϕ = 0.75 would be more appropriate. This is based on the fact that the steel plate can plastically redistribute the shear to all headed studs better than to post-installed anchors.

TENSION STRENGTH

Concrete Capacity

The design tensile strength of a single anchor governed by concrete failure is given in ACI 318-02, which was confirmed for headed-stud use by reviewing the existing database of tension test results for cast-in anchors as part of the Phase 2 WJE-PCI research. From these test results, group factors in tension are slight variations of the ACI model factors. The in-place tensile strength of the headed-stud anchorage should be taken as the minimum value based on computing both the concrete and steel capacity for a given anchorage configuration.

Single Anchor Tension Capacity

The single stud concrete breakout prediction equation is given by:

$$\phi N_u = \phi 30\lambda\sqrt{f_c'}\left(h_{ef}\right)^{1.5} \qquad \text{(Eq. 6)}$$

where

ϕ = concrete strength reduction factor = 0.75 (for tension)

N_u = concrete tensile breakout capacity for a single stud

h_{ef} = effective embedment; the effective steel stud height, after welding burn off, defined to the base of the head

= L (stud length after welding) - A (head height) + the thickness of the plate to which the studs are attached (See Table 6.5.1.2 in *PCI Design Handbook*.)[8]

burnoff = amount of stud length lost in the welding process

λ = concrete density factor

= 1.0 for normalweight concrete

= 0.85 for sand lightweight concrete

= 0.75 for all lightweight concrete

For anchors in tension, the single anchor capacity is modified by several factors to account for the effects of edges, spacing, and cracking.

$$N_{cb} = N_{cbg} = f(\phi, N_u, s_1, s_2, d_{e1}, d_{e2}, d_{e3}, d_{e4}, cracking) \qquad \text{(Eq. 7)}$$

The factors are a combination of modification and correction factors because the CCD model was used as the basis of the design procedure. In some respects, the PCI design procedure follows the philosophy first used in the Kappa method. However, because the already-accepted CCD method in ACI 318 is fundamentally based on the Kappa method, returning to the Kappa method approach in the sixth edition of the *PCI Design Handbook* did not seem appropriate.

For simplification, the group equation is presented in the following form in the handbook:

$$N_{cb} = N_{cbg} = C_{bs}A_N C_{crb}\Psi_{ed,N} \qquad \text{(Eq. 8, handbook Eq. 6.5.4.1[8])}$$

where the breakout strength coefficient is defined by:

$$C_{bs} = 3.33\lambda\sqrt{\frac{f_c'}{h_{ef}}} \qquad \text{(Eq. 9, handbook Eq. 6.5.4.2[8])}$$

C_{bs} is equivalent to the term $\dfrac{N_b}{A_{No}}\Psi_3$ in ACI 318-02:

where

N_b = basic concrete breakout strength in tension of a single headed-stud anchor in cracked concrete

Ψ_3 = modification factor, for strength in tension, to account for cracking

A_{No} = projected concrete failure area of one anchor, for calculation of strength in tension when not limited by edge distance or spacing

A_N = projected concrete failure area for a stud or group of studs

Cracking—Cracking in the vicinity of an anchor will reduce its concrete tensile breakout capacity. The amount of reduction is dependent on the width of the crack, as derived from research. In accordance with ACI 318, the cracking coefficient C_{crb} is:

C_{crb} = 1.0 for *uncracked* concrete ($f_t < f_r$) at service loads, and

C_{crb} = 0.80 for *cracked* concrete at service loads.

Edge distance factor—Anchors are affected by edge distance because, when close to a free edge, the concrete breakout surface is not fully developed. In simplistic terms, the loss of available concrete for the tension breakout surface reduces the overall capacity of the connection. The edge distance factor is effectively a model correction factor accounting for differences between the model prediction and the test results in the database. The edge distance modification factor $\Psi_{ed,N}$ only needs to be applied once, even if there are edge distances within 1.5 h_{ef} on more than one side of the anchors. If there are three or more edges, special rules are given in ACI 318 Appendix D to reduce the effective embedment depth of the anchors. See the commentary in ACI 318-05 for further details.

$$\Psi_{ed,N} = 0.7 + 0.3\left(\frac{d_{e,min}}{1.5h_{ef}}\right) \le 1.0$$

(Eq. 10, handbook Eq. 6.5.4.3[8])

where
d_{min} = minimum dimension of d_{e1}, d_{e2}, d_{e3}, or d_{e4} (in.); reference should be made to Fig. 6.5.4.2 in the handbook[8]

Eccentricity factor—Any eccentricity of the applied load relative to the geometric centroid of the anchor group causes a non-uniform distribution of resisting forces in the anchors. The eccentricity modification factor applied to Equation 8 $\Psi_{ec,N}$ accounts for the non-uniform load distribution in the anchor group.

$$\Psi_{ec,N} = \frac{1}{\left(1 + \frac{2e_N'}{3h_{ef}}\right)} \le 1.0$$

(Eq. 11, handbook Eq. 6.5.4.4[8])

where
e_N' = distance between resultant tension load on a group of anchors loaded in tension and the centroid of the group of anchors loaded in tension (in.); e_N' is always positive
e_N' ≤ $s/2$, where s = minimum individual x or y spacing, depending on the eccentricity direction being evaluated
If an eccentricity exists in two directions, $\Psi_{ec,N}$ must be applied for each direction.

Pull-Out Strength

Pull-out capacity N_{pn} is dictated by a failure of the concrete around the head of the headed stud. When the bearing area of the head is small, concrete crushing occurs at the head and the anchor can pull out and crush the concrete without forming a concrete breakout cone. Local crushing under the head of the anchor significantly reduces the stiffness of the anchor and increased displacement is associated with crushing under the head of the anchor.

$$N_{pn} = 11.2 A_{brg} f_c' C_{crp}$$

(Eq. 12, handbook Eq. 6.5.4.5[8])

where
A_{brg} = bearing area of the stud head in tension (in.^{2})
= area of the head − area of the shank (values are shown in handbook Table 6.5.1.2[8])
C_{crp} = cracking coefficient (pullout)
= 1.0 for concrete assumed uncracked (most common)
= 0.7 for locations likely to become cracked

Side-Face Blowout

For a single headed stud located close to a free edge (d_{e1} < 0.4 h_{ef}), the side-face blowout capacity is defined as:

$$N_{sb} = 160 d_{e1}\sqrt{A_{brg}}\sqrt{f_c'}$$

(Eq. 13, handbook Eq. 6.5.4.6[8])

This side-face blowout capacity N_{sb} must be multiplied by a factor if the single headed stud is located near a corner such that $d_{e3} < 3d_{e1}$. This factor is defined as:

$$\frac{\left(1 + \frac{d_{e3}}{d_{e1}}\right)}{4}$$

(Eq. 14, handbook Eq. 6.5.4.7[8])

where

$$1 \le \frac{d_{e3}}{d_{e1}} \le 3$$

For multiple headed studs located close to a free edge (d_{e1} < 0.4 h_{ef}), the side-face blowout capacity is further modified as:

$$N_{sbg} = \left(1 + \frac{s_0}{6d_{e1}}\right) N_{sb}$$

(Eq. 15, handbook Eq. 6.5.4.8[8])

where
d_{e1} = distance to closest edge (in.)
A_{brg} = bearing area of the head of stud or anchor bolt (in.^{2})
s_0 = center-to-center spacing of transverse reinforcement within the length of l_0 (in.)
N_{sb} = side-face blowout strength of a single anchor (lb), see Eq. 13

Side-face blowout failures are unique to embedded, headed anchors. This failure type is affected by an edge condition, but not the same edge condition associated with a general concrete breakout failure. If the head of an anchor is close to a free edge, the compression stress bulb at the head bearing region can cause the concrete side face to spall because it is no longer confined. Note that this condition applies to very small edge distances and relatively deeply embedded anchors.

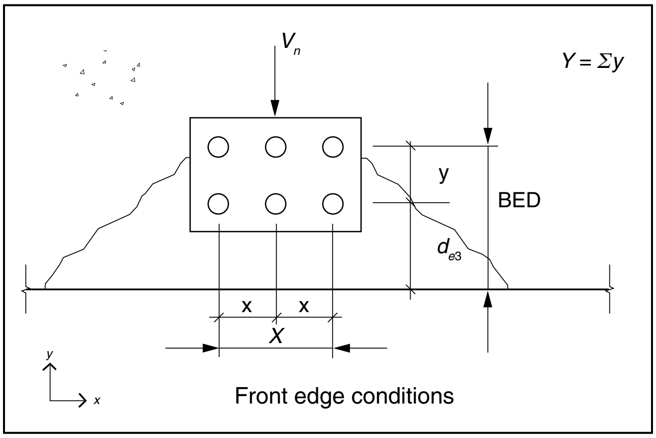


Fig. 4. Conventional concrete breakout when the shear load is applied perpendicular or normal to the free edge. Note: BED = back-edge distance.

SHEAR STRENGTH

Concrete Capacity

The procedure to determine the design shear strength governed by concrete failure is based on the models and test results from the WJE/PCI research project. The results used were those from experimental testing programs exclusively based on headed-stud anchors. The strength of an anchor should be taken as the minimum value based on computing both the concrete and steel capacity for the characteristics of the unique anchor configuration.

Front Edge (d_{e3})

The front-edge condition represents the majority of shear loaded connections in design and is the condition that has typically yielded the smallest concrete breakout capacity. A shear force is applied perpendicular, or normal, to the front edge of the concrete, as illustrated in **Fig. 4**. The design capacity, when governed by concrete front-edge breakout, is given by:

$$\phi V_{c3} = \phi V_{co3}(C_{X3})(C_{h3})(C_{ev3})(C_{vcr})$$

(Eq. 16, handbook Eq. 6.5.5.1[8])

where

ϕ = concrete strength reduction factor
= 0.70 without confinement steel
= 0.75 with confinement steel

V_{c3} = nominal concrete breakout capacity for a single or multiple stud connection, accounting for member and connection geometry

V_{co3} = nominal concrete breakout capacity for a single stud connection unaffected by connection or member geometry

C_{X3} = coefficient for overall X spacing, spacing of anchors in rows parallel to the free edge (or spacing of anchors perpendicular to the applied shear force), of a connection with two or more columns, for the d_{e3} type anchorage

C_{h3} = coefficient for member thickness (h) for the d_{e3} type anchorage

C_{ev3} = coefficient for eccentric shear force influences for a d_{e3} type anchorage

C_{vcr} = coefficient for cracking in the concrete

The prediction equation for single stud concrete breakout capacity is given by:

$$V_{co3} = 16.5\lambda\sqrt{f_c'}\left(\text{BED}\right)^{1.33}$$

(Eq. 17, handbook Eq. 6.5.5.2[8])

where the back-edge distance (BED) to the rear row of studs is defined as:

$$\text{BED} = d_{e3} + \sum y_i = d_{e3} + Y$$

(Eq. 18, handbook Eq. 6.5.5.3[8])

where

y = individual Y-row spacing (center to center) (in.); reference Fig. 4

Y = the overall, out-to-out dimension of the column of studs on the side row of the anchorage
= Σy (in.), parallel to the applied shear force

λ = ACI 318 lightweight concrete factor

d_{e3} = front-edge distance parallel to the shear load application direction and y-axis, taken from the center of a front-anchor shaft to the front concrete edge (in.)

X spacing—The influence of the stud spacing in rows perpendicular to the applied shear force between adjacent columns of anchors (when two or more studs are in the back row) requires a strength modification by the X-spacing coefficient C_{X3}:

$$C_{X3} = 0.85 + \frac{X}{3\text{BED}} \le n_{studs-back}$$

$$C_{X3} = 1.0 \text{ when } X = 0$$

(Eq. 19, handbook Eq. 6.5.5.4[8])

where

X = the overall, out-to-out dimension of the row of studs in the back row of the anchorage
= Σx (in.), perpendicular to the applied shear force
= 1.0 for a connection with only one stud in the back row

BED = back-edge distance, defined previously (in.)

$n_{studs\text{-}back}$ = number of studs in the back row

x = center-to-center spacing of stud rows perpendicular to the shear force direction

Member thickness—The influence of the concrete member's thickness is accounted for by the member thickness coefficient C_{h3}:

$$C_{h3} = 1.0 \quad \text{for } h > 1.75 \text{ BED}$$

$$C_{h3} = 0.75\sqrt{\frac{h}{\text{BED}}} \quad \text{for } h \le 1.75 \text{ BED}$$

(Eq. 20, handbook Eq. 6.5.5.5[8])

where
h = member thickness (in.)
BED = back-edge distance, defined previously (in.)

Eccentricity—The location of the applied shear force is not always concentric with the centroid of the resisting anchors. Effectively, this places the anchor group into a torsional-shear state. To account for this eccentricity, the group capacity requires modification by the eccentric load factor C_{ev3}:

$$C_{ev3} = \frac{1}{1 + 0.67\left(\frac{e_v'}{\text{BED}}\right)} \le 1.0$$

$$\text{for } e_v' \le \frac{X}{2}$$

(Eq. 21, handbook Eq. 6.5.5.6[8])

where
e_v' = eccentricity of shear force on a group of anchors
= the distance between the point of shear force application and the centroid of the group of anchors resisting shear in the direction of the applied shear (in.)

Cracking—Cracking in the vicinity of, or through, the anchors will reduce their shear capacity, as observed from research. The influence of a crack parallel to the shear force on the anchor capacity is not quite as dramatic as that from cracking in tension. The cracking coefficients are of a different magnitude than the ones shown in ACI 318. The basic concrete breakout equation in the *PCI Design Handbook* is the uncracked capacity, rather than the cracked capacity, as reflected in ACI 318. The cracking coefficient C_{vcr} is:

for *uncracked* concrete ($f_t < f_r$):

$$C_{vcr} = 1.0$$

for *cracked* concrete at service load:

C_{vcr} = 0.70 when no edge reinforcement or reinforcement is smaller than No. 4 (No. 13M) bar
C_{vcr} = 0.85 with supplementary reinforcement of No. 4 (No. 13M) bar or greater between the anchor and the edge
C_{vcr} = 1.0 with supplementary reinforcement between the anchor and the edge of No. 4 (No. 13M) bar or greater and supplementary reinforcement enclosed within stirrups with a spacing less than 4 in. (100 mm)

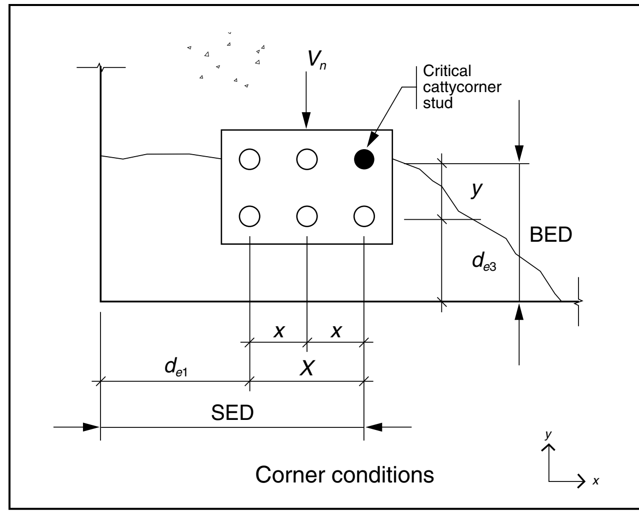


Fig. 5. Corner-concrete breakout when a headed-stud anchorage is located near a member corner. Note: BED = back-edge distance; SED = side-edge distance.

Corners

The corner condition is considered to be a special case of the front-edge loaded anchor. Again, the shear force is applied perpendicular or normal to the concrete's front edge as illustrated in **Fig. 5**, but the anchor is located sufficiently close to the corner (side edge) so that a different concrete breakout shape occurs. A corner condition results when:

$$0.2 < \frac{\text{SED}}{\text{BED}} \le 3.0$$

(Eq. 22, handbook Eq. 6.5.5.7[8])

where the side-edge distance (SED) to the cattycorner (anchor point) stud, as shown in Fig. 5, is defined as:

$$\text{SED} = d_{e1} + \sum x = d_{e1} + X$$

(Eq. 23, handbook Eq. 6.5.5.8[8])

The design shear capacity governed by concrete breakout at the corner is thus given by:

$$\phi V_{c3} = \phi V_{co3}\,(C_{h3})\,(C_{c3})\,(C_{vcr})\,(C_{ev3})$$

(Eq. 27, handbook Eq. 6.5.5.9[8])

where
ϕ = concrete strength reduction factor
= 0.70 without confinement steel
= 0.75 with confinement steel
C_{h3} = coefficient for member thickness (h) for a d_{e3} type anchorage (see handbook Eq. 6.5.5.6[8])

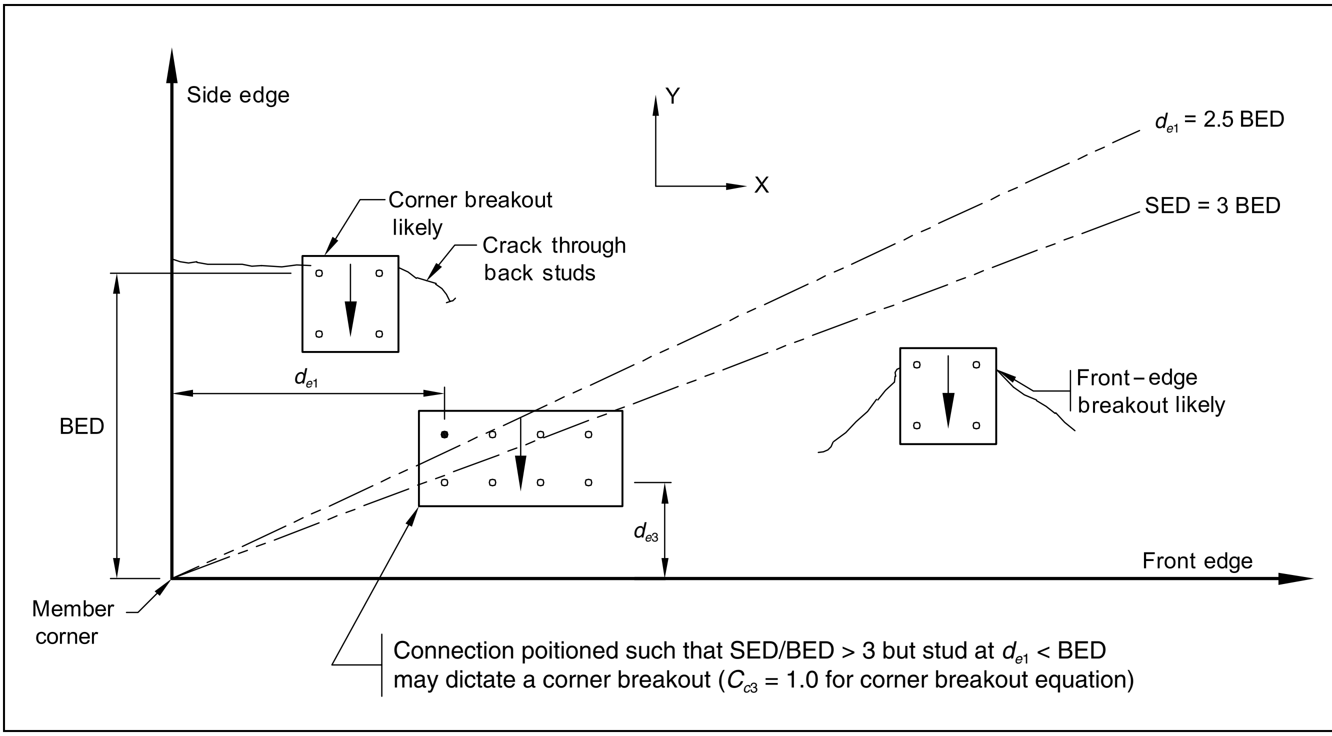


Fig. 6. Corner transition zone where a close-to-corner stud may induce a zipping type crack propagation to the corner (*PCI Design Handbook*[8] Figure 6.5.5.3). Note: BED = back-edge distance; SED = side-edge distance.

C_{vcr} = coefficient for cracking in a member, loaded in shear

C_{c3} = coefficient for the corner influence for a d_{e3} type anchorage

C_{ev3} = coefficient for eccentric shear force influences for a d_{e3} type anchorage

For stud anchors located near a corner, the corner coefficient C_{c3} is:

$$C_{c3} = 0.7\sqrt[3]{\frac{\text{SED}}{\text{BED}}} \leq 1.0$$

(Eq. 25, handbook Eq. 6.5.5.10[8])

For the special case when there is a large overall spacing in an X-row (the row perpendicular to the applied shear) and the stud anchor is located near a corner, such that SED/BED > 3.0, but one of the stud rows remains fairly close to the corner, a corner type crack and breakout may still result. A transition zone exists under these conditions, such that:

$$C_{c3} = 1.0 \quad \text{for SED} > 3.0 \text{ BED and } d_{e1}/\text{BED} \leq 2.5 \qquad \text{(Eq. 26)}$$

This situation is shown in **Fig. 6**, reproduced from Fig. 6.5.5.3 in the sixth edition of the *PCI Design Handbook*. Given the large, wide overall anchor spacing of the outside studs (large *X*) with the end stud fairly close to the corner, yet with SED/BED > 3.0, crack propagation can occur along the line of the rear stud row and be directed to the side edge, as shown in Fig. 5. This corner-type failure was experimentally observed in the WJE/PCI research for the conditions set forth in Eq. 26.

Side Edge (d_{e1} or d_{e2})

A side-edge concrete breakout failure mode is significantly different from the traditional front-edge breakout mode. In this case, the shear force is applied parallel to the side edge of the concrete member, as illustrated in **Fig. 7**. To determine if a connection is a corner or a side-edge condition, the following equation is evaluated. The anchorage will behave in a side-edge breakout mode if:

$$\frac{\text{SED}}{\text{BED}} \leq 0.2$$

(Eq. 27, handbook Eq. 6.5.5.11[8])

The design capacity governed by concrete breakout at the side edge is given by:

$$\phi V_{c1} = \phi V_{co1}(C_{X1})(C_{Y1})(C_{ev1})(C_{vcr})$$

(Eq. 28, handbook Eq. 6.5.5.12[8])

where

ϕ = concrete strength reduction factor
= 0.70 without confinement steel
= 0.75 with confinement steel

V_{c1} = concrete breakout capacity for a single or multiple stud connection, accounting for member and connection geometry

V_{co1} = concrete breakout capacity for a single stud connection unaffected by connection or member geometry

C_{vcr} = coefficient for cracking in a member, loaded in shear

C_{X1} = coefficient for overall X spacing of the outside studs in a connection with two or more x columns (rows perpendicular to the shear force) for a d_{e1} type anchor

C_{Y1} = coefficient for overall Y spacing of the outside studs in a connection with two or more y rows (rows parallel to the applied shear force) for a d_{e1} type anchor

C_{ev1} = coefficient for eccentric shear force influences for a d_{e1} type anchor

The single stud concrete breakout prediction equation is given by:

$$V_{co1} = 87\lambda\sqrt{f_c'}\left(d_{e1}\right)^{1.33}\left(d_0\right)^{0.75}$$

(Eq. 29, handbook Eq. 6.5.5.13[8])

where

d_{e1} = side-edge distance to the first line of studs (in.); for cases where two parallel sides exist and the anchorage is off center, use the lesser of d_{e1} or d_{e2}

d_0 = stud diameter (in.)

λ = ACI 318 lightweight concrete factor

f_c' = concrete compressive strength (psi)

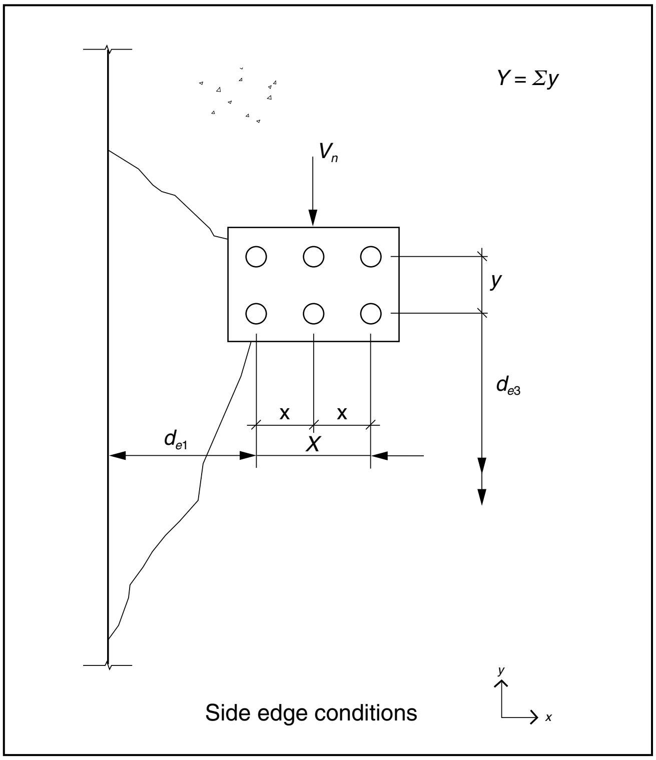


Fig. 7. Corner concrete breakout when the shear load is applied parallel to the free edge.

X spacing—The factor accounts for anchors in a row perpendicular to the shear force located near one edge of the concrete member and for anchors having studs in a row adjacent to two parallel edges of the concrete member.

Figure 8 reproduces Figure 6.5.5.4 from the sixth edition of the PCI Design Handbook and illustrates the side-edge-distance conditions. The X-spacing factor for two or more studs in a row perpendicular to the shear is given by:

$$C_{x1} = \frac{n_x x}{2.5 d_{e1}} + 2 - n_{sides}$$

$$\text{where } 1 \le C_{x1} \le n_x$$

$$C_{x1} = 1.0, \text{ when } x = 0$$

(Eq. 30, handbook Eq. 6.5.5.14[8])

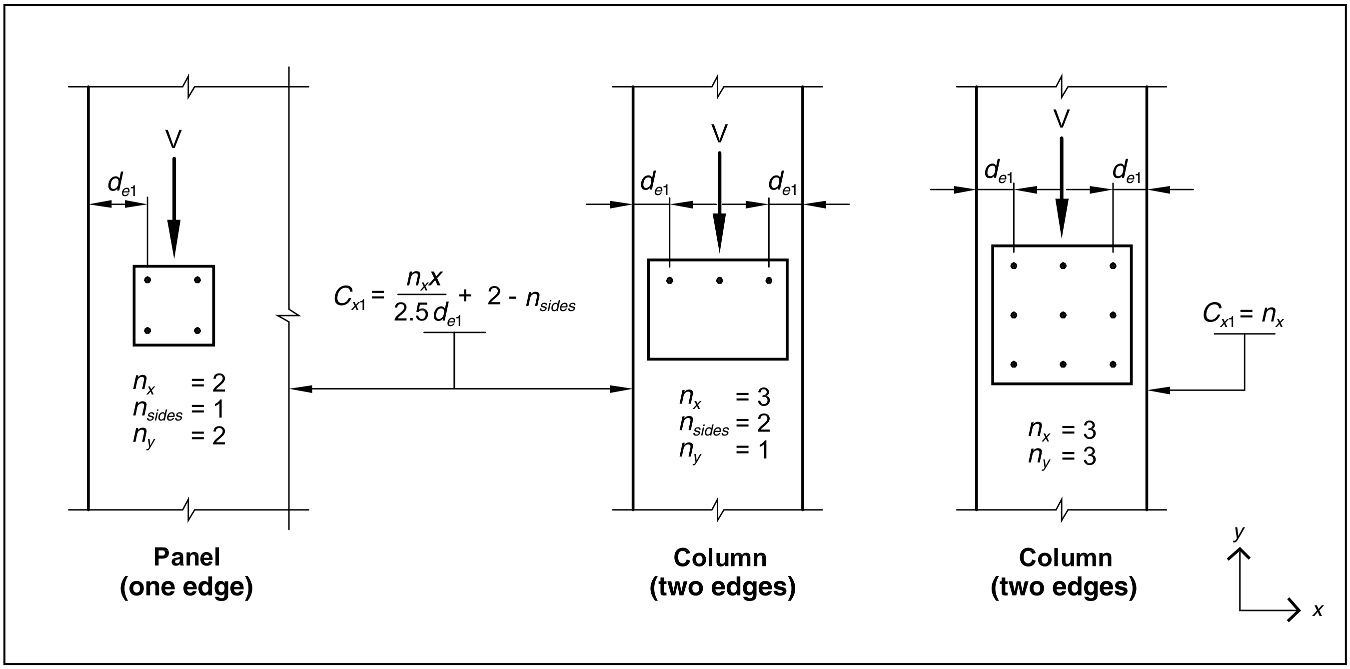


Fig. 8. Various definitions for side-edge distance factors for one or two side edges (*PCI Design Handbook* Figure 6.5.5.4[8]).

where

n_x = number of X-row stud lines

x = individual X-row spacing (in.)

n_{sides} = number of edges or sides

For all anchorages with multiple rows perpendicular to the shear force and that are located adjacent to two parallel edges, such as a column corbel connection, the X-spacing factor for two or more studs in the row perpendicular to the shear is:

$$C_{x1} = n_x \quad \text{(Eq. 31, handbook Eq. 6.5.5.15[8])}$$

Y Spacing—The Y-spacing is a factor for spacing in a column of anchors parallel to the load. This influence applies to two or more stud rows perpendicular to the shear force and adjacent to one edge of the concrete member or in the case of two parallel side edges, such as the vertical edges of a column with a shear force acting parallel to the height of the column:

$$C_{Y1} = 1.0 \text{ for } n_y = 1 \text{ (one } y \text{ row)}$$

$$C_{Y1} = \frac{(n_y Y)^{0.25}}{0.6d_{e1}} + 0.15 \le 1 \text{ for } n_y \ge 1.0$$

(Eq. 32, handbook Eq. 6.5.5.16[8])

where

n_y = number of Y-row stud lines (rows parallel to the shear)

Y = out-to-out Y-row spacing of anchors = Σy

> **Author's note**: Please note the errata in this equation for C_{y1} in the *PCI Design Handbook.* The factor is capped at n_y and not 1.0 as shown in the handbook.

Eccentric shear—When the shear load is applied eccentric to the anchorage's centroid, the load eccentricity will reduce the anchorage capacity by:

$$C_{eV1} = 1.0 - \left(\frac{e_{V1}}{4d_{e1}}\right) \le 1.0$$

(Eq. 33, handbook Eq. 6.5.5.17[8])

where

e_{V1} = the eccentricity from the shear load to the centroid of the anchorage (in.)

Back Edge (d_{e4})

A back-edge condition exists when the shear force is applied perpendicular, or normal, to the back edge of the concrete member, as illustrated in Fig. 1. Under a condition of pure shear loading, the back-edge distance has been found, through testing, to have no influence on the connection capacity when the studs are well developed. The pryout failure mode should be checked, as applicable.

IN THE FIELD

The WJE/PCI research identified a failure mode that occurs with very short studs located far enough from an edge to preclude a concrete-edge breakout failure. This failure mode is known as pryout and was somewhat evaluated in the 800 equation in previous editions of the *PCI Design Handbook*. The equation presented in the current *PCI Design Handbook*, Chapter 6, is a refinement of the 800 equation based on a review of existing literature and WJE and German test data.[41] This design condition will influence a very small population of headed-stud sizes, yet it may be a very common stud type when used in thin-wythe sandwich members.

When a headed-stud anchor is sufficiently far from all edges, termed *in-the-field of the member*, the anchorage capacity will be governed by the capacity of the steel stud(s). Pryout failure is a concrete breakout mode that may govern when short, stocky studs are used ($h_{ef}/d_0 < 4.5$). The pryout capacity in lightweight and normal weight concrete has been found to govern when $h_{ef}/d_0 < 4.5$. If this condition exists:

$$\phi V_{cp} = \phi V_{cp0}$$

$$V_{cp0} = 215 n_{studs} \Psi_y \sqrt{f_c'} (d_0)^{1.5} (h_{ef})^{0.5} \le n_{studs} A_s F_{ut}$$

(Eq. 34, handbook Eq. 6.5.7.1[8])

where

$$\Psi_y = \frac{\sqrt{y}}{4d_0} \text{ for } \frac{y}{d_0} \le 20$$

or

$$\Psi_y = 1.0 \text{ for } y = 0$$

ϕ = strength reduction factor
= 0.70 without confinement reinforcement

V_{cp} = nominal pryout shear strength (lb)

n_{studs} = number of studs in the connection group

A_s = effective cross-sectional area of the stud anchor (in.^{2})

f_c' = specified compressive strength of concrete (psi)

y = center-to-center spacing of studs in direction of load

> **Author's note**: Please note the errata in the above equation for ψ_y in the *PCI Design Handbook*. The factor $\psi_y = 1.0$ when $y = 0$ and should be on the next line, rather than as presented in the handbook.

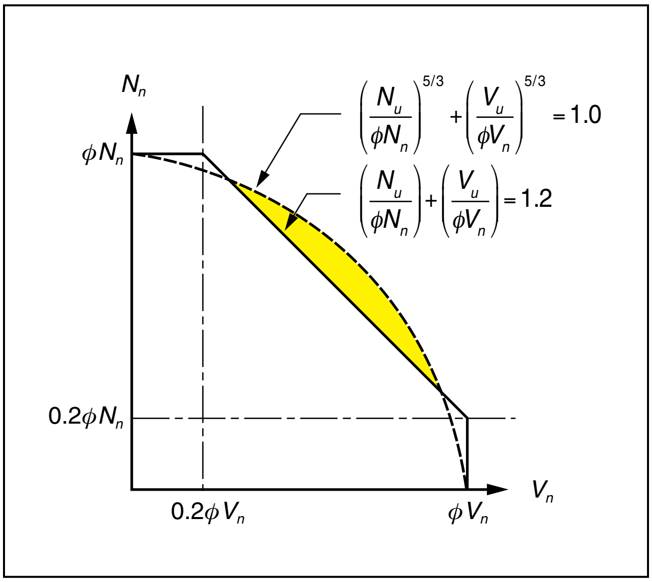


Fig. 9. The interaction of tension and shear presented in the *PCI Design Handbook*[8] Figure 6.5.8.1.

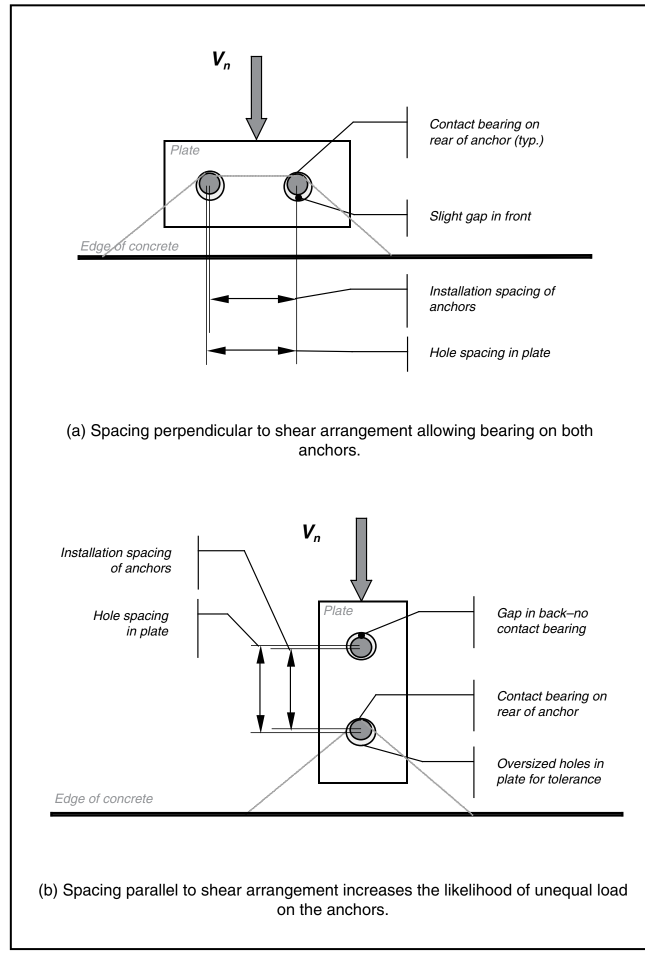


Fig. 10. Load conditions inherent with cast-in-place anchor bolts or post-installed anchorages (a) X-spacing arrangement allowing bearing on both anchors (b) Y-spacing arrangement increases the likelihood of unequal load on the anchors.

COMBINED TENSION AND SHEAR

WJE is in the process of analyzing the research data and formulating design recommendations for tension and shear as part of Phase 2 of the headed-stud research project. The design guidelines in the sixth edition of the *PCI Design Handbook* allow for anchorage capacity determination with the tension-shear interaction permitted by ACI 318-02. In the ACI provisions, a simplified tri-linear relationship for the interaction of tension and shear is permitted. In the ACI 318 commentary, the traditional five-thirds (5/3) power interaction is permitted, which is in accordance with previous editions of the *PCI Design Handbook*. Both relationships are plotted in **Fig. 9**. It is noted in the sixth edition of the *PCI Design Handbook* that both equations should be examined when determining the capacity of headed-stud anchors, as the tri-linear relationship may truncate some allowable combinations when the tension and shear magnitudes are approximately equal (between about 30 and 60 degrees, or the shaded region of Fig. 9).

As mentioned, it is anticipated that the present analysis of the interaction and the design recommendations will be incorporated into the seventh edition of the *PCI Design Handbook*.

Cast-In Anchor Bolts

The provisions for the front-edge distance d_{e3} condition are intended for use with headed-stud anchors, where the stud is welded to an attachment plate. Cast-in anchor bolts behave somewhat similarly to studs in a majority of the concrete breakout modes, but their behavior is highly dependent on the degree of fixity to the attachment plate and which of the bolts in the group are bearing on the attachment plate. Headed studs are fully welded and, hence, fixed to the plate, so double curvature in the stud displacement can develop under lateral shear loading. Likewise, a more equal distribution of the applied load to the individual studs can be assumed in a headed-stud anchor.

A cast-in anchor bolt may be placed into an oversized hole for tolerance purposes, such that some "slop" is introduced into the connection. Bolt rotation within the plate hole can occur, making full fixity difficult to achieve. In anchorages with multiple cast-in bolts, the various oversized hole dimensions coupled with the actual bolt location within the hole can give rise to conditions of uneven bearing among the bolts and, consequently, uneven loading on individual bolts in the group. **Figure 10** illustrates these issues with hole versus anchor size.

The provisions of ACI 318 Appendix D are more applicable to these anchorage situations. Moreover, post-installed anchors should be designed in accordance with the ACI provisions or those recommended by the anchor manufacturer.

Adhesive Anchors

Anchors used in concrete construction fall into two broad categories: cast-in-place anchors and post-installed anchors. With increasing demand for more flexibility in the planning and construction of concrete structures, and for repair and retrofit applications, post-installed anchors have seen increased use. One popular form of the post-installed anchor is the adhesive anchor.

An adhesive anchor is a threaded rod or a reinforcing bar that is inserted in a hole drilled in hardened concrete. The diameter of the hole is typically 15% to 25% larger than the diameter of the anchor. The annulus around the anchor is filled with an adhesive that bonds the steel anchor to the concrete, and the anchor is termed an adhesively bonded anchor.

While there are generally accepted and more recently codified procedures for the design of cast-in-place anchors, such as headed studs, comparable information is not yet available for adhesive anchors. Thus, in the interim, the designer must rely on manufacturers' recommendations to estimate the strength of these anchors.

During the past five years, research has been reported that suggests unified models for calculating tensile strength and shear strength for single adhesive anchors.[42,43] While the calculation models exist, the designer should correlate calculated capacity with manufacturers' listed capacities to assess the actual factor of safety associated with adhesive anchors.

NOTES ON NOTATION

To simplify the notation used to define the planar geometry of a headed-stud plate, the designer should notice the use of a Cartesian coordinate system of the stud layout in the *PCI Design Handbook*. Figures 4, 5, and 7 illustrated the x- and y-axes' layout used in the design equations. In general, the y-axis is oriented parallel to the applied shear force, whereas the x-axis is then oriented perpendicular to the applied shear force. Similar to a spreadsheet, it is sometimes convenient to view a multi-stud connection in terms of columns (parallel to the y-axis) and rows (parallel to the x-axis).

SUMMARY

The WJE/PCI headed-stud research program has produced an alternate shear design procedure that better represents the behavior of headed-stud anchors in precast concrete members. This design procedure also conforms to the ACI 318 Appendix D requirements. The sixth edition of the *PCI Design Handbook* now recognizes that there are different types of failure modes associated with headed-stud anchors depending on the type of edge condition, connection geometry, and the edge distances in relation to the connection. The front-edge breakout mode was contained in previous editions of the *PCI Design Handbook* and has been refined through WJE/PCI research. In addition, the corner-concrete breakout mode has been found to control over a greater range of edge distances, and revised connection capacity equations are presented. In the sixth edition of the *PCI Design Handbook*, the concept of a side-edge breakout was introduced, as it was part of the WJE/PCI research program. Capacity equations for a connection adjacent to a side edge are presented for the first time in this edition of the handbook.

Tension design of headed-stud anchors follows the ACI 318 Appendix D approach because the design model was found by WJE to be a good representation of headed-stud behavior in tension.

RESEARCH NEEDS

Through the course of the WJE/PCI research endeavor, attempts were made to isolate a number of variables to determine their influence. However, the nature of research is to uncover unanswered questions through extensive data analysis, newly discovered behavior, lack of appropriate or relevant tests, or new methodologies to review existing data from the literature. To this end, a number of conditions or behaviors were uncovered that could be addressed through future research. Suggested research needs include the following:

- Better definition of the transition region of corners to front edge d_{e3} and side edge d_{e1};
- Capacity of anchors in lightweight concrete; limited research has been performed to verify the assumed applicability of the λ factors from ACI 318; and
- Capacity of anchorages with large Y-spacing (spacing parallel to the shear force) to investigate shear lag influences.

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REFERENCES

1. Industry Handbook Committee. 1971. *PCI Design Handbook: Precast and Prestressed Concrete.* 1st ed. Chicago, IL: Prestressed Concrete Institute (PCI).
2. American Concrete Institute (ACI) Committee 318. 2002. *Building Code Requirements for Structural Concrete (ACI 318-02) and Commentary (ACI 318R-02)*. Farmington Hills, MI: ACI.
3. ACI Committee 318. 2005. *Building Code Requirements for Structural Concrete (ACI 318-05) and Commentary (ACI 318R-05)*. Farmington Hills, MI: ACI.
4. ACI Committee 349. 1983. Proposed Revisions to: Code Requirements for Nuclear Safety Related Concrete Structures (ACI 349-76). *Journal Proceedings.* V. 80, No. 2 (March): pp. 79–84.
5. ACI Committee 349. 1978. Addition to Commentary on Code Requirements for Nuclear Safety Related Concrete Structures (ACI 349-76). *ACI Journal*, V. 75, No. 8 (August): pp. 329–347.
6. Eligenhausen, R., and W. Fuchs. 1988. Tragverhalten von Dübelbefestigungen bei Querzug-, Schrägzug-, und Biegebeanspruchung [Load-Bearing Behavior of Anchor Fastenings under Shear, Combined Tension and Shear or Flexural Load-

ing]. [In German and English.] *Betonwerk + Fertigteil-Technik*, V. 54, No. 2: pp. 48–56.
7. Industry Handbook Committee. 1999. *PCI Design Handbook: Precast and Prestressed Concrete*. 5th ed. MNL-120-99. Chicago, IL: PCI.
8. Industry Handbook Committee. 2004. *PCI Design Handbook: Precast and Prestressed Concrete*. 6th ed. MNL-120-04. Chicago, IL: PCI.
9. Anderson, N. S., and D. F. Meinheit. 2000. Design Criteria for Headed Stud Groups in Shear: Part 1—Steel Capacity and Back Edge Effects. *PCI Journal*, V. 45, No. 5 (September–October): pp. 46–75.
10. Anderson, N. S., and D. F. Meinheit. 2001. Steel Capacity of Headed Studs Loaded in Shear. In *Connections between Steel and Concrete: International RILEM Symposium, proceedings Pro21*, ed. R. Eligehausen, pp. 202–211. Cachan, France: RILEM Publications S.A.R.L.
11. Anderson, N. S., and D. F. Meinheit. 2005. Pryout Capacity of Cast-In Headed Stud Anchors. *PCI Journal*, V. 50, No. 2 (March–April): pp 90–112.
12. Fuchs, W., R. Eligehausen, and J. E. Breen. 1995. Concrete Capacity Design (CCD) Approach for Fastening to Concrete. *ACI Structural Journal*, V. 92, No. 1 (January–February): pp. 73–94.
13. International Conference of Building Officials (ICBO). 1979. *Uniform Building Code*. Whittier, CA: ICBO.
14. Cruz, R. D. 1987. Effect of Edge Distance on Stud Groups Loaded in Shear and Torsion. Masters thesis. Oklahoma State University, Stillwater, OK.
15. Wong, T. L. 1988. Stud Groups Loaded in Shear. Masters thesis. Oklahoma State University, Stillwater, OK.
16. Courtois, P. 1969. Industrial Research on Connections for Precast and In-Situ Concrete. In *Mechanical Fasteners for Concrete,* SP-22, pp. 123–138. Detroit, MI: ACI.
17. Industry Handbook Committee. 1978. *PCI Design Handbook: Precast and Prestressed Concrete*. 2nd ed. Chicago, IL: PCI.
18. McMackin, P. J., R. G. Slutter, and J. W. Fisher. 1973. Headed Steel Anchor under Combined Loading. *AISC Engineering Journal*, V. 10, No. 2: pp. 43–52.
19. Industry Handbook Committee. 1985. *PCI Design Handbook: Precast and Prestressed Concrete.* 3rd ed. Chicago, IL: PCI.
20. Cannon, R. W., E. G. Burdette, and R. R. Funk. 1975. *Anchorage to Concrete*. Report No. CEB 75-32. Knoxville, TN: Civil Engineering Branch, Tennessee Valley Authority.
21. Klingner, R. E., J. A. Mendonca, and J. B. Malik. 1982. Effect of Reinforcing Details on the Shear Resistance of Anchor Bolts under Reversed Cyclic Loading. *ACI Journal*, V. 79, No. 1 (January–February): pp. 3–12.
22. TRW Inc. Nelson Stud Welding Division. 1988. *Embedment Properties of Headed Studs*. Elyria, OH: TRW Inc.
23. Shaikh, A. F., and W. Yi. 1985. In Place Strength of Welded Headed Studs. *PCI Journal*, V. 30, No. 2 (March–April): pp. 56–81.
24. Ollgaard, J. G., R. G. Slutter, and J. W. Fisher. 1971. Shear Strength of Stud Connectors in Lightweight and Normal-Weight Concrete. *AISC Engineering Journal*, V. 8, No. 2: pp. 55–64.
25. Martin, L. D., and W. J. Korkosz. 1982. *Connections for Precast Prestressed Concrete Buildings, Including Earthquake Resistance*. Technical Report No. 2. Chicago, IL: PCI.
26. Industry Handbook Committee. 1992. *PCI Design Handbook: Precast and Prestressed Concrete.* 4th ed. MNL-120-92. Chicago, IL: PCI.
27. American Society for Testing and Materials (ASTM). 2004. Standard Specification for Carbon Steel Bolts and Studs. ASTM A307-04. Volume 01.08. West Conshohocken, PA: ASTM.
28. Rong, A. Y., and A. Fafitis. 1989. *Tensile and Shear Strength of Single and Group Studs*. Tempe, AZ: Civil Engineering Department, Arizona State University.
29. Comité Euro-International Du Béton (CEB). 1994. *Fastenings to Concrete and Masonry Structures*. Lausanne, Switzerland.
30. Mindess, S., and J. F. Young. 1981. *Concrete*. Englewood Cliffs, N.J.: Prentice-Hall Inc.
31. Kuhn, D. P., and A. F. Shaikh. 1997. *Pilot Study on Headed Anchor Studs: A Comparison between PCI and CCD*. Milwaukee, WI: Department of Civil Engineering, University of Wisconsin–Milwaukee.
32. ASTM. 2005. Standard Specification for Carbon Structural Steel. ASTM A36-05. V. 01.04. West Conshohocken, PA: ASTM.
33. ASTM. 2004. Standard Specification for Structural Steel Shapes. ASTM A992-04a. V. 01.04. West Conshohocken, PA: ASTM.
34. ASTM. 2003. Standard Specification for Annealed or Cold-Worked Austenitic Stainless Steel Sheet, Strip, Plate, and Flat Bar. ASTM A666-03. V. 01.03. West Conshohocken, PA: ASTM.
35. Goble, G. G. 1968. Shear Strength of Thin Flange Composite Specimens. *Engineering Journal,* V. 5, No. 2: pp. 62–65.
36. Perry, T. C., R. R. Funk, and E. G. Burdette. 1983. Effect of Plate Flexibility on Anchor Loads. *Symposium on Anchorage to Concrete.* Farmington Hills, MI: ACI.
37. American Welding Society (AWS). 2004. *Structural Welding Code—Steel*. AWS D1.1/D1.1M. 20th ed. Miami, FL: AWS.
38. ASTM. 2003. Standard Specification for Steel Bars, Carbon, Cold-Finished, Standard Quality. ASTM A108-03. V. 01.05. West Conshohocken, PA: ASTM.
39. AWS. 2002. *Structural Welding Code—Steel*. AWS D1.1/D1.1M. 18th ed. Miami, FL: AWS.
40. Chambers, H. A. 2001. Principles and Practices of Stud Welding. *PCI Journal*, V. 46, No. 5 (September–October): pp. 46–58.
41. Zhao, G. 1994. *Tragverhalten von randfernen Kopfbolzenverankerungen bei Betonbruch* [Load-Carrying Behavior of Headed Stud Anchors in Concrete Breakout Away from an Edge]. [In German.] Report 1994/1. Stuttgart, Germany: Institut für Werkstoffe im Bauwesen, Universität of Stuttgart.
42. Cook, R. A., J. Kunz, W. Fuchs, and R. C. Konz. 1998. Behavior and Design of Single Adhesive Anchors under Tensile Load in Uncracked Concrete. *ACI Structural Journal,* V. 95, No. 1 (January): pp. 9–26.
43. Bickel, T. S., and A. F. Shaikh. 2002. Shear Strength of Adhesive Anchors. *PCI Journal*, V. 47, No. 5 (September–October): pp. 92–101.

NOTATION

A_s = effective cross-sectional area of a stud anchor (in.^{2})

A_{se} = effective cross-sectional area of a stud anchor (in.^{2}), ACI 318 Appendix D notation

d_b = diameter of the anchor

d_{e1} = side-edge distance normal to the shear load application direction, parallel to the x-axis, taken from the center of an anchor shaft to the side concrete edge (in.)

d_{e2} = side-edge distance normal to the shear load application direction, parallel to the x-axis, taken from the center of an anchor shaft to the side concrete edge (in.); also, the side-edge distance opposite d_{e1}

d_{e3} = front-edge distance parallel to the shear load application direction and y-axis, taken from the center of a front-anchor shaft to the front concrete edge (in.)

d_{e4} = back- or rear-edge distance parallel to the shear load application direction and y-axis, taken from the center of a back anchor shaft to the rear concrete edge (in.)

d_0 = shaft diameter of a headed stud (in.)

f_c' = specified compressive strength of concrete (psi)

F_{ut}, f_{ut} = specified ultimate tensile strength of anchor steel in tension (psi)

F_{vy} = shear yield strength of anchor steel (psi)

F_y, f_y = specified yield strength of anchor steel in tension (psi)

h = thickness of a concrete member in which the anchors are embedded, measured parallel to the anchor axis (in.)

h_{ef} = effective headed-stud embedment depth taken as the length under the head to the concrete surface (in.)

n = number of anchors in a connection or group

N_{cb} = nominal concrete breakout strength in tension of a single anchor (lb), ACI 318 Appendix D notation

N_u = concrete tensile breakout capacity for a single stud

t_p = thickness of the attachment plate (in.)

V_n = nominal shear strength (lb)

V_s, V_{steel} = nominal shear strength of a single headed stud or group of headed studs governed by steel strength

x = center-to-center spacing of stud anchors in the x direction of the Cartesian plane (in.)

X = out-to-out X-row spacing = Σx

y = center-to-center spacing of stud anchors in the y direction of the Cartesian plane (in.)

Y = out-to-out Y-row spacing = Σy

λ = concrete density factor
= 1.0 for normalweight concrete
= 0.85 for sand lightweight concrete
= 0.75 for all lightweight concrete

κ = one-sided population limit (fractile) factor for a normal distribution

μ = coefficient of friction

ϕ = strength reduction factor

SI EQUIVALENTS

1 in. = 25.4 mm
1 ft = 0.3048 m
1 lb = 4.448 N
1 kip = 4.448 kN
1 psi = 6.895 kPa
1 ksi = 6.895 MPa
1 yd^3 = 0.7646 m^3