PCI's Headed Stud Design Redefined...

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The construction industry has relied on the headed-stud anchor to provide connection between concrete and multiple other structural elements for many years. In the past, the design engineer followed numerous accepted rational methods of designing for allowable strength and serviceability in accordance with accepted standard procedures.

Recent changes in the requirements in ACI 318, as well as new developments from testing completed by PCI, will impact the design engineer's practice in designing anchorages to concrete. Failure to stay current with new design procedures can result in liberal design solutions.

The following article, PCI's Headed Stud Design Redefined, is worth reading. The historical overview discusses the chronology of design methods based on various boundary conditions and load applications. The standard for designing headed stud anchor connections has been impacted by new testing results compiled by PCI. This article offers the design engineer a state-of-the-art approach to headed-stud design.

John Mercer
STRUCTURE Editorial Board

Welded headed studs have been used to connect concrete 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* [PCI 1971] since the early 1970s. Although the design concepts accounting for multiple anchors with variable spacings or those anchors close to free edges have existed for 35 years, new design concepts in the American Concrete Institute's (ACI) *Building Code Requirements for Structural Concrete* Appendix D [ACI 318 2005] have raised questions about the older PCI model, which has been used and worked successfully since 1971. The design model used by PCI is very similar to the model used by ACI Committee 349 [ACI 349 1978] in the design of anchors for nuclear structures.

In 2002, ACI Committee 318 included new provisions for designing all types of mechanical expansion anchors, headed studs, and embedded bolts that could be embedded into concrete. The codified design procedure is found in ACI 318-05 Appendix D *Anchoring to Concrete*. These new provisions present a simple physical concrete breakout model, which can easily accommodate the effects of anchor spacing in two directions and effects of edge conditions. The origins of the model date to the mid-1960s. However, the model was more fully developed in the European Kappa method of the late 1980s, and put into a "designer friendly" form with the development of the Concrete Capacity Design (CCD) model [Fuchs, Eligenhausen and Breen 1995]. Placing anchorage requirements in the ACI Code was motivated by a need to rationally design post-installed anchorages. Cast-in-place anchors, including headed studs and bolts, also were incorporated into the design provisions. The ACI 318-05 Appendix D design provisions, however, are heavily based on the behavior of post-installed anchors.

The codification of the design requirements finally provide a design engineer with the methodologies to consider many of the geometry and member influences that can affect the capacity and the overall design of an anchorage group. Prior to the ACI codification of design provisions, post-installed anchors generally had to be designed using manufacturer's catalogs/procedures and a standard factor of safety, usually 4. Visualizing the tension or shear behavior of an anchorage was not readily apparent by looking in a table of ultimate and safe working capacities. Hence, the codification of procedures greatly facilitates the general design of concrete anchors.

However, upon digging deeply into the research literature on headed studs and embedded bolts, the authors found that the database for tests on headed studs and embedded bolts loaded in shear was small compared to that on post-installed anchors. The database on tension is more substantial. 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 when the new and old procedures give different solutions.

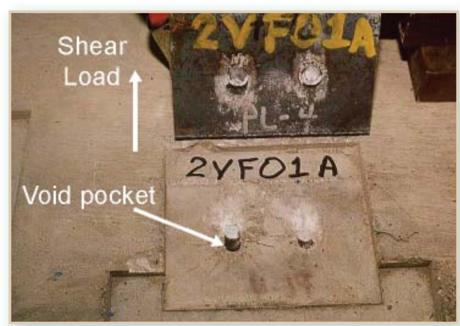

Figure 1: Typical steel failure of a headed-stud connection loaded in shear

PCI's Research Initiative

The ACI design method was calibrated using an extensive database of post-installed anchor tests. There is a potential that the codified design method is skewed toward post-installed anchors because the database is heavily dominated by post-installed expansion anchor tests. Consequently, some provisions for headed-stud connections and other cast-in anchorages may not be totally appropriate.

In the precast concrete industry, the lifeline of connecting precast elements is the embedded plate. The majority of these embedded plates are anchored with welded-headed studs. In the mid 1990s, PCI initiated a headed-stud research program to create a database for headed-stud group connections. This research program was originally in response to the proposed ACI Appendix D provisions where certain provisions within headed-stud connection design were found more liberal than the CCD approach. Because the design procedures for tension and shear loading of headed studs were based on a limited amount of research data, PCI initiated an industry sponsored research project to satisfy these points:

- Provide justification for the design procedures used in the past, which through Code implementation and adoption are now considered unconservative.
- Create a database of tests to justify accepting and conforming to the provisions of ACI 318-05 Appendix D, modifying the ACI procedures, refining the design procedures as currently published in the *PCI Design Handbook* [PCI 1999], or create a database of tests to write a new design procedure independent of ACI 318, which is permitted in the Code.

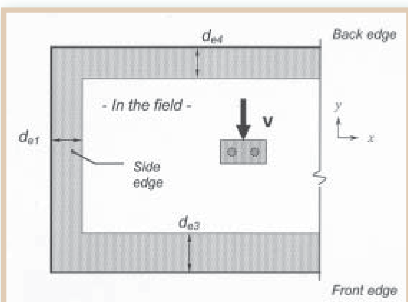

Figure 2: Definitions of edge conditions associated with shear-loaded anchorage groups

The PCI Research Program

PCI initiated this research work in two phases. Phase 1 examined headed-stud anchorages loaded only in shear, because the literature review showed almost a complete lack of headed-stud test data under this condition. Phase 2 reviewed the tension-only information from a moderately sized existing database compiled for headed-stud connections. In addition, Phase 2 included experimental studies evaluating the affects of combined tension and shear on multi-stud connections. Both of these research programs were conducted by Wiss, Janney, Elstner Associates, Inc. (WJE) in their Northbrook, Illinois structural laboratory. In total, over 400 headed-stud tests have been conducted by WJE.

The shear research program, and ensuing revisions to the *PCI Design Handbook* [PCI 1999] provisions, considered the geometric effects of the front-edge distance, corners, side-edge distance, back-edge distance, and in-the-field type connections. A new Sixth Edition of the *PCI Design Handbook* [PCI 2004] is available from PCI.

Welded, headed studs are designed to resist direct tension, shear, or in the usual case the combination of the two. The design equations given in the *PCI Design Handbook* [PCI 2004] are applicable to studs, which are welded to steel plates or other structural members, and embedded in unconfined, uncracked concrete. It is assumed that the steel plates are of sufficient thickness to prevent significant plate deformation and adequately transfer the applied load to the studs. In addition, the in–place connector strength should be taken as the minimum value based on computing both the concrete and steel characteristic capacities for an anchorage configuration.

Figure 3: Example of a front edge concrete breakout when the shear force is applied perpendicular (normal) to the edge

Results from the Research

Steel Stud Capacity

As presented in an earlier paper [Anderson and Meinheit 2001], the design ultimate tensile or shear strength governed by steel failure can be determined by using the minimum design ultimate tensile strength of the stud steel (F_{ut}), instead of the yield strength. This realization in the concrete industry may appear new; however, the steel industry has used the ultimate strength for stud shear design since the early 1970s. Although, when failure theories are reviewed, the shear failure load should be less than the tensile load. However, the shear failure shown in *Figure 1* occurs at a load dictated by the tensile ultimate strength.

Tension

As stated above, there is a substantial database of tension tests on headed studs and embedded bolts. WJE's review of data in Phase 2 of the WJE/PCI research shows that the ACI provisions accurately represent the tension behavior of headed studs. Therefore, in the new *PCI Design Handbook* provisions [PCI 2004], the design single anchor tensile strength governed by concrete failure is taken as the ACI 318-05 Appendix D provisions.

Shear

The WJE/PCI research has developed new design shear strength provisions, as governed by a concrete breakout failure. The provisions are calibrated based on experimental testing of headed-stud anchorages, which is permitted by the ACI Code. *Figure 2* shows the different edge conditions for shear, discussed further in the following sections.

Front Edge - The front edge condition represents a majority of shear-loaded connections found in engineering practice. A photograph showing a front edge concrete breakout is seen in *Figure 3*. The shear force is applied perpendicular or normal to the front edge. The 6th Edition of the *PCI Design Handbook* [PCI 2004] will have the following attributes:

- A new equation for the single stud breakout capacity is provided. Similar to ACI, PCI has adopted a design methodology where the single-stud capacity is modified by several factors to account for group anchorages.
- For a multiple-stud connection, the breakout and hence capacity is defined by the position of the rear or back row of studs. The concept of Back Edge Distance (BED) for design is introduced.
- Modification factors will be presented to account for member thickness, the influence of spacing perpendicular to the shear force for two or more studs in the back row, the influence of eccentricity, which places the anchor group into a state of torsional shear, and cracked concrete.

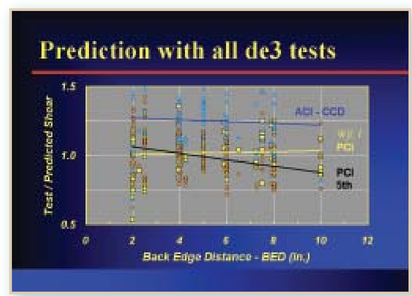

Figure 4: Front edge breakout comparison of the proposed WJE / PCI equation in the 6th Edition, PCI Design Handbook [PCI 2004] with the present ACI 318-05 Appendix D and 5th Edition, PCI Design Handbook [PCI 1999]

A comparison of the equations representing the average capacity including the affects of edge distance, spacing, and thickness of the concrete is shown in *Figure 4*. Clearly the ACI design provisions are conservative relative to the database developed from the WJE/PCI research testing. The WJE/PCI capacity equations are of course calibrated to the database and agree very well with the data.

Corners - The corner is considered to be a special case of the front-edge loaded anchorage. Shear force is applied perpendicular or normal to the front edge, but the anchorage is located

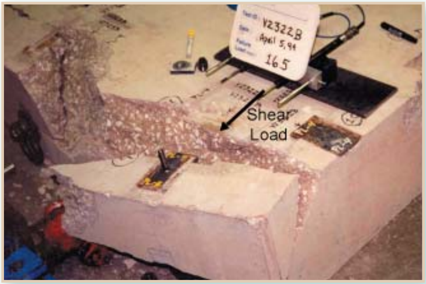

Figure 5: Corner concrete breakout when the shear force is applied perpendicular (normal) to the front edge

sufficiently close to the corner such that a different concrete breakout path occurs. A corner failure is shown in *Figure 5*. Corners will be marked by:

- The corner capacity is defined as the single-stud capacity modified by a corner factor, a thickness factor, and/or a cracked concrete factor.
- For a multiple-stud connection at a corner, the research showed the breakout capacity is defined by the cattycorner stud, or stud diagonally opposite the geometric corner. The concept of Side Edge Distance (SED) to the cattycorner (an anchor point for breakout cracking) stud is defined.
- The research found that the headed-stud corner influence might be greater than the ACI model implies. The *PCI Design Handbook* [PCI 2004] provides further guidance on this influence.

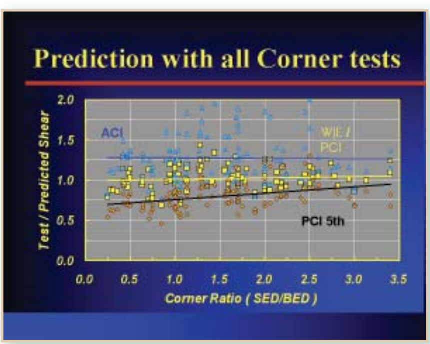

Figure 6: Corner breakout comparison of the proposed WJE / PCI equation in the 6th Edition, PCI Design Handbook [PCI 2004] with the present ACI 318-05 Appendix D and 5th Edition, PCI Design Handbook [PCI 1999]

Figure 7: Side-edge concrete breakout when the shear force is applied parallel to the free edge

Corner influences are much better modeled by the WJE/PCI equations, shown in *Figure 6,* than the new ACI provisions or the old PCI provisions, one conservative and the other unconservative, respectively.

Side Edge - Side-edge conditions exist when the shear load is parallel to the free edge, but the anchor(s) are close enough to the free edge to cause a breakout. A side-edge loaded connection results in a concrete breakout failure mode significantly different from the traditional front-edge breakout mode. The photograph in *Figure 7* shows a side edge breakout for a two stud connection. ACI 318 Appendix D's treatment of side edge conditions applies a contrived model to this case. The ACI design method fictitiously turns the anchorage 90 degrees, has the designer compute a front-edge breakout capacity, and then doubles this result for the side edge capacity. Through the WJE/PCI research, the side-edge behavior was observed and the breakout capacity has been better defined. The highlights of the *PCI Design Handbook* [PCI 2004] method for side edges include:

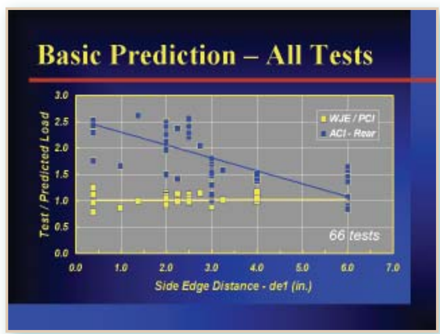

Figure 8: Side-edge breakout comparison of the proposed WJE / PCI equation in the 6th Edition, PCI Design Handbook [PCI 2004] with the present ACI 318-05 Appendix D method

- A new equation for the single-stud, side-edge breakout capacity is provided. Similar to ACI, PCI has adopted the design procedure whereby the single-stud capacity is modified by several factors to account for group anchorages.
- Modification factors account for the influence of spacing parallel and perpendicular to the shear force for the stud group, the influence of eccentricity (which places the anchor group into a state of torsional shear), and cracked concrete. The method also extrapolates the data to account for the difference in one side-edge, as found on a precast panel, and two parallel side edges, as would be found in a column.

The comparison of the ACI procedure and the WJE/PCI procedure are shown in *Figure 8*. It is noted that the ACI provisions are very conservative for small side-edge distances.

Back Edge - For this type of loading, the shear force is applied perpendicular (normal) to the back edge and is directed away from the free edge. Some pilot testing of single studs or stud groups, close to the back edge, implied a concrete breakout could occur in the presence of tension through eccentric shear. However, under a condition of pure shear, testing has found that the back-edge distance has no influence on the headed-stud connection capacity. At very small edge distances, the failure was consistently one of steel, as shown in *Figure 9*.

Figure 9: Steel stud failure at a back edge distance of 2½ in. (63.5 mm) or 4d

In-the-Field - When a headed-stud anchorage is sufficiently away from all edges, termed "in-the-field" of the member, the anchorage capacity is consistently governed by the steel stud(s), provided certain conditions exist with the stud geometry. If the stud is short and stocky, a pryout failure can occur; that is, if $h_{ef}/d < 4.5$, where h_{ef} is the effective embedment depth of the stud. *Figures 10* and *11* illustrate the schematic behavior of pryout and a photograph of an actual pryout test failure mode, respectively.

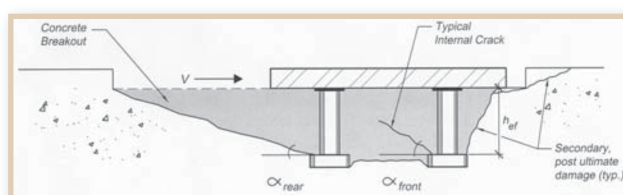

Figure 10: Sketch of a concrete pryout failure (courtesy of PCI Journal)

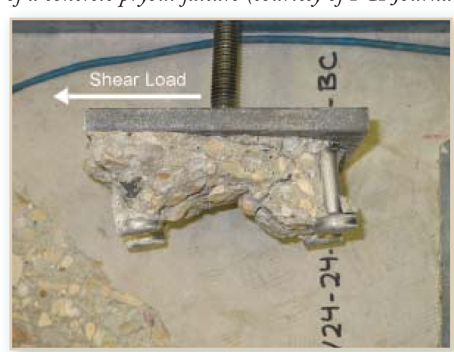

Figure 11: Photograph of pryout failure for a four-stud group

A new pryout capacity equation for lightweight and normal concrete is given in the new *PCI Design Handbook* [PCI 2004] provisions. As shown in a recent paper [Anderson and Meinheit 2005], the pryout equation in the *Design Handbook* is more appropriate than what is presently in ACI 318 Appendix D.

Combined Force Action - Phase 2 of the WJE/PCI research explored combined tension and shear loading on a stud group. A test representing an embedded steel column corbel with headed studs is shown in *Figure 12*. ACI 318-05 prescribes a tri-linear interaction equation, allowing the designer to neglect shear or tension influences when the applied shear or tension force is less than 20 percent of the capacity. The ACI 318-05 Commentary also suggests the use of an elliptical type interaction based on exponents of 5/3, which has been used in previous editions of the *PCI Design Handbook*. Both interaction equations are shown in *Figure 13* and can be used in the *PCI Design Handbook* [PCI 2004] procedures, because PCI considers both appropriate to use for design.

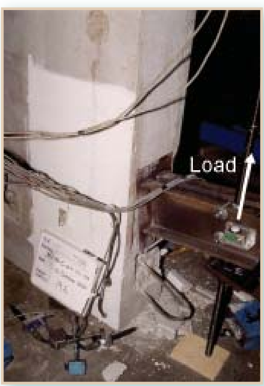

Figure 12: Photograph of combined tension and shear failure on an embedded column corbel plate

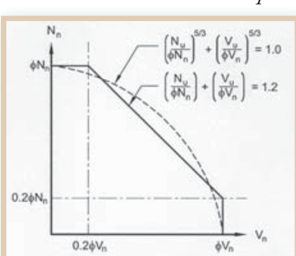

Figure 13: Combined tension and shear relationships permitted by ACI 318-05 Appendix D (plot from PCI Design Handbook [2004])

A Final Thought

The design provisions presented in the 6th Edition of the *PCI Design Handbook* [PCI 2004] represent a lower bound value based on breakout 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, accounting for reinforcement development beyond postulated failure planes. In most precast connection design, there is reinforcement surrounding the embedded plate studs. This extra measure of ductility exists but is currently not recognized by the new *PCI Design Handbook*.

More and better things are still to come as the behavior of anchoring to concrete unfolds…▪

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