ANCHORING TO CONCRETE: THE NEW ACI APPROACH

John E. Breen*, Eva-Maria Eichinger** and Werner Fuchs***
*Ferguson Structural Engineering Laboratory, The University of Texas at Austin, USA
**Institute for Structural Concrete, Technical University of Vienna, Austria
***Institute for Construction Materials, University of Stuttgart, Germany

Abstract
This paper outlines the general approach of a new appendix for design of anchoring to concrete in the American Concrete Institute (ACI) Building Code. It covers cast-in-situ anchors and mechanical post-installed anchors. ACI 318 and the ACI Technical Activities Committee have approved this proposal, and it is being published for public comment as part of the ACI 318-2001 revision. The proposed design procedures are in general harmony with provisions being developed by fib.

1. Foreword

It is a great privilege to participate in this Symposium. It is fitting that it is being held at the University of Stuttgart where Professors Gallus Rehm, Rolf Eligehausen and their coworkers, established the extensive scientific basis for modern approaches to Anchoring to Concrete.

While ACI has long had recommendations for the design of anchors when used in nuclear related structures (ACI 349) the ACI 318 Building Code has been silent on this subject. In 1970, ACI Committee 355, Anchorage to Concrete, was established to report on performance and recommend design and construction practices for anchorage to concrete. Because of trade conflicts, no design related code recommendations ever came to ACI 318. In 1989 ACI 318 Sub B formed a task force to develop anchoring to concrete design provisions. Anchor qualification provisions were left to ACI 355 and ASTM. The design approach adopted by ACI 318 stems directly from the interaction with Dr. Fuchs as a DFG post-doctoral fellow at The University of Texas in 1990-1991. His visit resulted in major integration of the European and North American test data on cast-in-place and post-installed anchors reported in comparison studies of Reference 1. Detailed discussions led to adaptation of the previously proposed Stuttgart κ method to
make it more “designer friendly,” a most important factor in subsequent adoption by ACI 318.

Now, twelve years later that challenge is virtually done, at least in its first phase. This paper describes where ACI 318 is and its general approach. But, in structural concrete, doors never truly close. While the new recommendations cover much and represent a great improvement, they are still only first steps. The search is just beginning for similar design provisions for adhesive and grouted anchors.

2. Introduction

In 1995 when setting the goals for the 2001 ACI 318 Building Code, the membership voted overwhelmingly to add specific design provisions for anchoring to concrete. This reflects the increased demand for such design guidance by code users, the considerable research and design development stimulated by ACI Committees 349 and 355, and the increased cooperation with CEB (now fib).

Main decisions in the ACI 318 approach were based on the unanimous technical advice from Committee 355. The approach had to be compatible with the load and resistance factor format of the present code. The nominal resistance expressions should be consistent with the observed accuracy of the design formulae or values from comprehensive tests. A design approach should be found that accommodates brittle failure as well as ductile failure modes. The design provisions that envision brittle failure should use load and resistance factors appropriate for brittle failure modes. The nominal resistance design formulae should account for the effects of the type of anchor, anchor material, anchor diameter, edge distance, spacing, concrete strength, embedment depth and for the effects of cracking. An alternate approach using site specific testing to determine design values should be included. The basic approach of the ACI 318 Building Code provisions is to express all possible modes of failure for the anchors, to require the use of conservative design provisions based on the 5 percent fractile, and to provide some limited spacings, edge distance minimums and minimum thicknesses for the concrete member. Then, while the user is allowed to choose any design models or design by test values that meet these general requirements, for practical use a “deemed to satisfy” procedure is included. This latter procedure for steel failures is based on the method of AISC LRFD [2] while for concrete failures, it is based on the Concrete Capacity Design (CCD) procedure [1,3] that is accurate, designer friendly and in good agreement with tests. Special provisions for seismic applications and enhanced ductility through use of supplementary reinforcement are included.

The ACI 318 Building Code provisions are applicable in scope to cast-in-place headed studs and headed or hooked bolts as well as a variety of post-installed anchors such as expansion anchors and undercut anchors [See Fig. 1]. Committee 318 plans to include provisions for adhesive anchors in a future code revision. A key element in the design philosophy is that the post-installed anchors must be prequalified by acceptance testing.
using performance standards developed by ASTM or ACI 355 (ACI 355.2-00) [4]. This testing differentiates between post-installed anchors according to their installation sensitivity, behavior in oversize holes, low and high strength concrete, or with partial torque or expansion in cracked or uncracked concrete. These standards are similar to European EOTA (ETAG) requirements but adapted to American certification procedures. The anchors are placed in one of three categories according to their performance in tests. The ACI Code gives reduced resistance factors ($\phi$) for the poorer performing categories. Designers may specify allowable categories to be used according to their safety requirements. The ACI 318 Appendix passed all voting procedures of Committee 318 and was approved by the ACI Technical Activities Committee pending final approval of the reference testing standard. A version for cast-in-place anchors that does not require such acceptance testing was adopted and included in the International Building Code 2000 [9]. An almost identical version has been adopted by ACI 349B for cast-in and post-installed anchors for nuclear-related structures. Since the Nuclear Regulatory Commission prescribes test procedures for fastening acceptance, the ACI 349B-1999 version did not have to wait for completion of the reference acceptance testing standard. Thus, ACI has now replaced the traditional 45º cone approach of ACI 349B with the new CCD procedures. The overall approach has also been adopted by the Fastening to Concrete committee of NEHRP (National Earthquake Hazard Reduction Program) for inclusion in NEHRP 2000 [10].

![Diagram of anchors](image)

Figure 1 – Types of anchors

The proposed Appendix provides design requirements for structural anchors used to transmit structural loads from attachments into concrete members by means of tension, shear, or a combination of tension and shear. Several failure types of fasteners can be differentiated [See Figs. 2 and 3]. Strength design of structural anchors is based on the computation or test evaluation of the steel tensile and shear strengths of the anchor and the attachment, the concrete breakout tensile and shear strengths, the tensile pullout strength of the anchor, the side-face blowout strength, the concrete pryout strength and required edge distances, spacing and member thickness to preclude splitting failure. The minimum of these strengths is taken as the nominal strength of the anchor for each load condition. Regardless of the mode which governs for a given anchor at a given embedment depth, the suitability of post-installed anchors for use in concrete must be demonstrated by the prequalification tests of ACI 355.2 [4].
Many possible design approaches exist and the user is always permitted to “design by test” as long as sufficient data are available to verify the model. If test results are used these must be evaluated on an equivalent statistical basis to that used to select the values for the concrete breakout method in the “deemed to satisfy” provisions. The basic capacity shall not be taken greater than the 5 percent fractile.

When the failure of an anchor group is due to breakage of the concrete, the behavior is brittle and there is limited redistribution of the forces between the highly stressed and less stressed anchors. In such a case the theory of elasticity must be used for determining the force on the anchor, assuming the attachment that distributes loads to the anchors is sufficiently stiff. The forces in the anchors are considered to be proportional to the external load and its distance from the neutral axis of the anchor group.

Figure 2 — Failure modes for fasteners under tensile loading
If an anchor failure is governed by ductile failure of the anchor steel, significant redistribution of anchor forces may occur. In such a case, analysis assuming the theory of elasticity will be conservative. A non-linear analysis, using theory of plasticity, is allowed for the determination of the ultimate loading conditions of ductile anchor groups.

The levels of safety defined by the combinations of load factors and resistance factors \((\phi)\) are appropriate for structural applications. The designer may use lower levels of safety in design for non-structural applications and may wish to use more demanding safety levels for particularly sensitive structural connections. The safety levels are not intended for handling and construction conditions. The \(\phi\) factors proposed for use with the current load factors given in the 1995 ACI Code Section 9.2 are given in Table 1.

Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided or where pullout or pryout strength governs.

Higher \(\phi\) factors are given for anchors that have supplementary reinforcement in the direction of the load to increase overall ductility, i.e. Condition A [See Fig. 4].
Table 1: $\phi$ factors

<table>
<thead>
<tr>
<th>Condition A</th>
<th>Condition B</th>
</tr>
</thead>
<tbody>
<tr>
<td>i) Shear Loads</td>
<td>0.85</td>
</tr>
<tr>
<td>ii) Tension Loads</td>
<td>0.85</td>
</tr>
</tbody>
</table>

Cast-in headed studs, headed bolts, or hooked bolts
Post-installed anchors with category as determined from ACI 355.2

- **Category 1** (Low sensitivity to installation and high reliability) | 0.85 | 0.75 |
- **Category 2** (Medium sensitivity to installation and medium reliability) | 0.75 | 0.65 |
- **Category 3** (High sensitivity to installation and lower reliability) | 0.65 | 0.55 |

Figure 4 – Influence of reinforcement on the load-displacement behavior of headed anchors loaded in shear (from Ref.5)
For a post-installed anchor to be acceptable in seismic loading situations, the system must be proven to have adequate ductility. The anchor must demonstrate the capacity to undergo large displacements through several cycles as specified in the seismic simulation of the ACI 355.2 prequalification tests. If the anchor cannot meet these requirements or if substantially reduced design loads are being applied which assume substantial ductility in the structure, then the attachment must yield at a load well below the anchor capacity.

3. Steel Based Resistance

For the calculation of steel failure, an approach based on the AISC LRFD [1] approach was “deemed to satisfy” (See Reference 6). In case of steel failure the shear and tensile strength of an anchor are evaluated based on the properties of the anchor material and the dimensions of the anchor. Values based on the 5 percent fractile of test results may also be used.

4. Concrete Based Resistance

The basic design concrete capacities for any anchor or group of anchors must be based on design models which result in predictions of strength in substantial agreement with results of comprehensive tests and which account for the size effect. They are to be based on the 5 percent fractile of the basic individual anchor capacity, with modifications made for the number of anchors, the effects of close spacing of anchors, proximity to edges, depth of the concrete member, eccentric loading of anchor groups, and presence or absence of cracking. Limits on edge distances and anchor spacing in the design models shall be consistent with the tests that have verified the model.

The “deemed to satisfy” design method used for the calculation of the concrete breakout capacities under tensile or shear loading was developed from the Concrete Capacity Design (CCD) Method [1,3], which was an adaptation of the $\kappa$ method [7,8] and is considered to be accurate, relatively easy to apply, and can be extended to irregular layouts. For single anchors, it assumes a breakout prism angle of about 35 degrees [Figs. 5, 6]. Both the CCD and the $\kappa$ methods include fracture mechanics theory, which indicates that in the case of brittle concrete failure the failure load increases at a rate less than the increase in the available surface and that the nominal stress at failure (peak load divided by failure area) decreases with increasing member size. The method predicts the load-bearing capacity of an anchor or group of anchors by using one basic equation for a single anchor in cracked concrete, and multiplying by factors which account for the number of anchors, edge distance, spacing, eccentricity and absence of cracking [1,6].

A very important attribute of the CCD approach is that it is reasonably “transparent” and hence designer friendly. Rather than working with the complex intersection of 45° cones as previously required by the ACI 349B approach, the CCD method when applied to groups uses values of $A_v/A_{vo}$ or $A_o/A_{vo}$ that are based on projected areas of
quadrilaterals. These areas are illustrated in Figure 7 for tension loads and in Figure 8 for shear loads.

The critical edge distance for headed studs, headed bolts, expansion fasteners, and undercut fasteners is $1.5h_{ef}$.

\[
A_{V_0} = 2 \times 1.5h_{ef} \times 2 \times 1.5h_{ef} \\
= 3h_{ef} \times 3h_{ef} \\
= 9h_{ef}^2
\]

Figure 5 – CCD concrete cone breakout model for tensile loading

The critical edge distance for heading studs, headed bolts, expansion fasteners, and undercut fasteners is $1.5c_1$.

\[
A_{V_0} = 2 \times 1.5c_1 \times 1.5c_1 \\
= 3c_1 \times 1.5c_1 = 4.5c_1^2
\]

Figure 6 – CCD concrete cone breakout model for shear loading
AN = (c1 + 1.5hef) (2 x 1.5hef)  
if c1 < 1.5hef

AN = (c1 + s1 + 1.5hef) (2 x 1.5hef)  
if c1 < 1.5hef and s1 < 3hef

AN = (c1 + s1 + 1.5hef) (c2 + s2 + 1.5hef)  
if c1 and c2 < 1.5hef 
and s1 and s2 < 3hef

Figure 7 – Projected areas for single anchors and groups of anchors for tension loads

5. Other Design Concepts

A comparison with the extensive test database indicated that the CCD method gave good results over the full range of applications [Figs. 9, 10]. While the ACI 349-85 procedure had very much the equivalent accuracy in some ranges, it was very unconservative in other ranges, particularly with group effects, and the geometry of intersecting circles was much more complex in group applications [Fig. 11].

However, recognizing that widely accepted procedures such as the earlier ACI 349-85 model as well as the PCI model can give satisfactory results in certain ranges, the proposed ACI 318 Appendix allows any “design models which result in substantial agreement with results of comprehensive tests” to be used. This generalized wording
allows previous procedures like the ACI 349 or PCI techniques to be used in applicable ranges if desired.

\[ A_v = 2 \times 1.5c_1 \times h \]

if \( h < 1.5c_1 \)

\[ A_v = 1.5c_1 (1.5c_1 + c_2) \]

if \( c_2 < 1.5c_1 \)

\[ A_v = (2 \times 1.5c_1 + s_1) \times h \]

if \( h < 1.5c_1 \) and \( s_1 < 3c_1 \)

\[ A_v = 2 \times 1.5c_1 \times h \]

if \( h < 1.5c_1 \)

Note: One assumption of the distribution of forces indicates that half the shear would be critical on front anchor and its projected area.

Note: Another assumption of the distribution of forces that applies only where anchors are rigidly connected to the attachment indicates that the total shear would be critical on the rear anchor and its projected area.

**Figure 8 – Projected areas for single anchors and groups of anchors for shear loads**

Typical cast-in-place headed studs, headed anchor bolts and hooked anchors have been tested and have proven to behave predictably, so calculated pullout values are acceptable.
Post-installed anchors do not have predictable pullout failure loads, therefore they must be tested. The pullout strength of headed studs or headed anchor bolts can be increased by provision of confining reinforcement such as closely spaced spirals throughout the head region. This increase can be demonstrated by tests.

The tensile and shear capacity can be increased by provision of supplementary reinforcement with resisting components in the direction of the applied force [See Fig. 4] [5].

Figure 9 – Mean and design CCD equations for anchors in uncracked concrete compared to test data for a) post-installed anchors and b) headed anchors
Equation vs. Shear Test Results
for single anchors in deep uncracked members
(European Tests)

Figure 10 – Mean and design CCD shear equations for uncracked concrete compared to test data

Figure 11 – Comparison of ACI 349-85 and CCD design equations for anchor groups
Interaction of tensile and shear loads is considered in the design using an interaction expression which results in predictions of strength in substantial agreement with results of comprehensive tests [See Ref. 6 and Fig. 12].

The values used for the tension part of the interaction equation shall be the smallest of the anchor steel strength, concrete breakout strength, sideface blowout strength, or pullout strength. For the shear part, the smaller of the steel strength, the concrete pryout strength or the concrete breakout strength shall be used.

\[
\phi N_n = \frac{0.2 \phi N_n}{0.2 \phi V_u} + \frac{\phi V_u}{V_u} = \frac{1}{1}
\]

Figure 12 – Shear and tensile load interaction equation

6. Status

In North America, a task force of ACI 318 Subcommittee B developed and refined the current proposals that have been approved by ACI Committee 318. ACI 355 recently completed the post-installed anchor acceptance test standard [4], but it has been subject to procedural and legal challenges by one anchor manufacturer. Assuming resolution of this challenge, comprehensive design provisions will be in ACI 318-2001. Even if the challenge provides further delay in adoption of the provisions for post-installed fasteners, the new ACI 349B expressions will be widely used and cast-in anchors will be governed by the recently adopted IBC 2000 provisions. These are identical to the ACI 318 provisions given herein but are limited to cast-in applications.

The proposed new Appendix to the ACI 318 Building Code is a very important step in harmonizing several existing design procedures. The user is allowed to choose any design models or design by test values that meet the general requirements. A design procedure based on the CCD design method is “deemed to satisfy.” The harmonization with the CEB task force leads to the hope that future fib recommendations and Eurocodes will be in close agreement with the new ACI approach.
7. References


4. ACI Committee 355, ACI Provisional Standard 355.2-00, Evaluating the Performance of Post-Installed Mechanical Anchors in Concrete, American Concrete Institute, Detroit, MI.


6. ACI Committee 318, Proposed Changes to Building Code Requirements for Structural Concrete, scheduled to be published in June 2001 edition of Concrete International.


