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CONSIDERATION OF BREAKOUT FAILURE MODES IN DESIGN OF ATTACHMENTS TO CONCRETE STRUCTURES

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ABSTRACT

Steel embedment plates are often needed in safety-related nuclear facilities to provide a means of attaching equipment, piping, and other nonstructural components to concrete walls. To accommodate large tensile forces on the plates, these plates may be anchored to the reinforced concrete structure using groups of conventional deformed reinforcing bar anchors (DRAs) or deformed wire anchors (DWAs). It is generally believed that specifying bars at a length greater than or equal to the code-required development length will provide adequate strength to transfer tensile forces to the concrete. However, this procedure neglects the potential for concrete breakout failure modes. This paper discusses the considerations of concrete breakout failure modes for groups of straight DRAs and DWAs. A discussion of the relevance of current breakout failure equations is included as well as suggestions for experimental testing and analytical modelling that can help to further understand this failure mode.

INTRODUCTION

Embedment plates commonly used in nuclear power plants consist of an attachment plate with a number of DWAs or DRAs welded to the plate. These anchors are embedded in the reinforced concrete structure as a means to transfer forces from the plate to the concrete. Figure 1 provides a schematic view of this type of anchorage assembly. Shear and tensile loads may be applied to the attachment plate. For the purposes of this study, only tensile forces are being considered.

There are a number of possible tensile failure mechanisms for this anchorage assembly: 1) Pullout (failure of the bond between the anchors and the concrete), 2) Steel/weld rupture, 3) Concrete breakout failure; however, breakout failure modes for straight, group anchor assemblies are rarely considered. Breakout failure is a concrete tensile failure that typically results in cracking at a 35º angle from the base of the anchor to produce a cone. This type of failure can significantly limit the pull-out strength of the system. The applicability and conservatism of the existing breakout failure equations for straight DRAs and DWAs has not been verified experimentally. This paper explores the applicability of concrete breakout failures for straight DRA and DWA assemblies and propose an experimental test matrix.

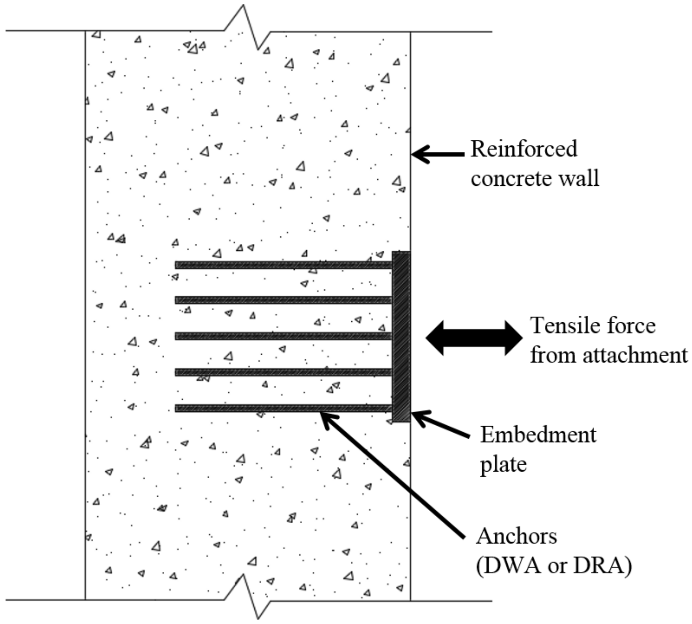


Figure 1 - Schematic section of anchorage assembly.

CODE-PRESCRIBED FAILURE MODES

Appendix B of ACI 349 (2001): Code Requirements for Nuclear Safety Related Concrete Structures provides design equations for anchoring to concrete by safely transferring shear and tensile demands from attachments to the concrete. The design requirements are applicable to both cast-in anchors and post-installed anchors. Only tensile loading is being evaluated in this study. Equation 1 calculates the nominal steel strength of the anchor group. In the equation, n is the number of anchors, A_{se} is the effective cross-sectional area of the anchor, and f_{ut} is the specified tensile strength of the steel anchor. For an embedment plate with many anchors, this failure mode is unlikely to govern due to the high strength and ductility of the steel anchors.

$$N_s = \mathrm{n}A_{se}f_{ut} \tag{1}$$

There is also the possibility of bond (pullout) failure of the anchors. This is prevented by ensuring that the bar has adequate development length. The development length can be calculated from Section 12.2 of ACI 349 (2001).

Breakout Failure Mode

Equation 2 (from Section B.5.2.2 of ACI 349) evaluates the nominal concrete breakout strength for the group of anchors. It was developed from the Concrete Capacity Design (CCD) Method (Fuchs et al. 1995; Eligehausen and Balogh, 1995). This breakout equation presumes a breakout prism angle of 35° as shown in Figure 2. In the equation, A_N is the projected area of the surface of the group breakout cone and A_{No} is the projected area of the failure surface of a single anchor, which can be estimated using Equation 3. h_{ef} is the effective anchor embedment depth. ψ_1 is a modification factor for concentric loading, ψ_2 is a modification factor for edge effects, and ψ_3 is a modification factor for cracking. N_b is the basic concrete breakout strength as given in Equation 4. In the equation, f'_c is the compressive strength of the concrete. *k* is a calibration factor based on the 5% fractile of experimental results. This 5% fractile value means that there is a 90% confidence that there is a 95% probability of actual strength exceeding nominal strength. The 5% fractile for cast-in anchors and post-installed (adhesive) anchors is 24 and 17, respectively. Note that these values for *k* are based primarily on headed stud/headed anchor bolt tests and adhesive anchor tests. The median (best estimate) value is 40 for cast in place anchors and 35 for post-installed fasteners (Fuchs et al. 1995). The applicability of these k factors for straight group anchor assemblies is not yet known.

$$N_{cbg} = \frac{A_N}{A_{No}} \psi_1 \psi_2 \psi_3 N_b \tag{2}$$

$$A_{No} = 9 h_{ef}^{\ 2} \tag{3}$$

$$N_b = k \sqrt{f_c'} h_{ef}^{\ 1.5} \tag{4}$$

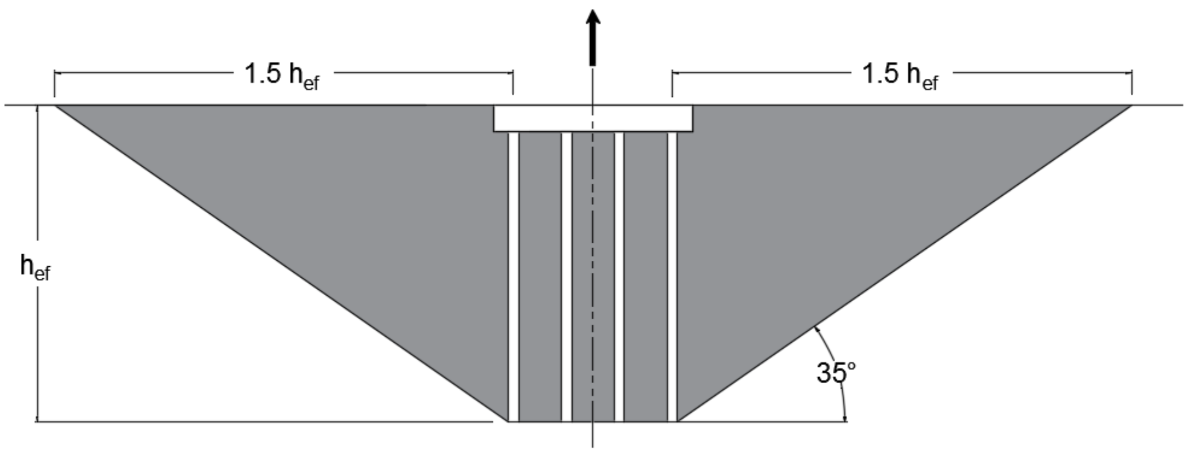


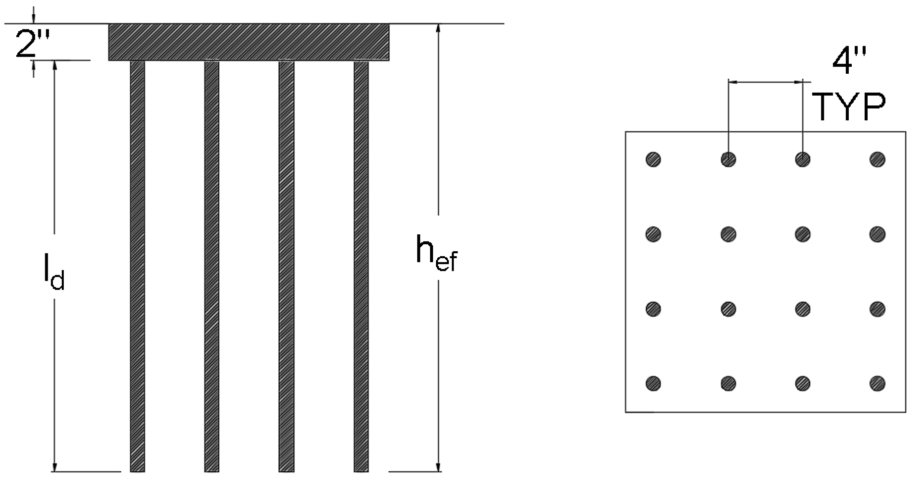
Figure 2 - Schematic section of breakout cone for tension.

CASE STUDY EXAMPLE

A case study example will be used to compare the capacities of each failure mode for a group anchorage assembly using the code provisions in Appendix B of ACI 349. Figure 3 shows an embedment plate with (16) #6 DRAs oriented in a 4x4 configuration. The concrete compressive strength is 5000 psi and the steel tensile strength, f_y = 65 ksi. The bars are embedded at the development length of the bar, l_d, per ACI 349, which is 16.5 inches long. It is assumed that the plate is sufficiently rigid to transfer axial forces uniformly between all anchors. Table 1 provides calculations for expected capacities for concrete breakout and steel failure modes. The concrete breakout capacity was calculated assuming ψ_3=1.25 (uncracked concrete). The *k* factor is varied, as shown in the table.

Based on the provisions for anchorage to concrete, concrete breakout capacity is the controlling failure mode for this specimen whether the *k* factor is assumed to be cast in place or post installed, and 5% fractile or median values. Even when using the median (best estimate) values of *k* which are unconservative for design, the calculated strengths are less than the steel failure capacity of 459 kips.

This finding shows that concrete breakout may be a relevant failure mode for this type of configuration. In order to determine the appropriate procedure to calculate concrete breakout strength, recommendations for testing and modelling are provided in the following section.



(a) (b)

Figure 3 - Case study specimen: (a) Section view. (b) Plan view.

Table 1 - Expected capacity failures for case study specimen.

(a) Concrete Breakout

$A_n = (1.5\ h_{ef} + 3s + 1.5\ h_{ef})^2$	4556 in^2
$A_{no} = 9\ h_{ef}^{\ 2}$	3080 in^2
$n\ A_{no}$	49280 in^2
$N_b / k = \sqrt{f'_c}\ h_{ef}^{\ 1.5}$	5.6 kips
$N_{cbg} / k = A_n/A_{no}\ \psi_1\ \psi_2\ \psi_3\ N_b$	10.4 kips
Design strength, N_{cbg} (5% fractile: cast in place, *k*=24)	**250 kips**
Expected strength, N_{cbg} (Best estimate: cast in place, *k*=40)	**416 kips**
Design strength, N_{cbg} (5% fractile: post-installed, *k*=17)	**177 kips**
Expected strength, N_{cbg} (Best estimate: post-installed, *k*=35)	**364 kips**

(b) Steel Yielding

$N_s = n\ A_{se}\ F_y$	**459 kips**

FUTURE NEEDS

Experimental testing must be conducted in order to validate the applicability of the breakout design equations for this type of group anchor assembly. Test results can be used to provide an indication of the k-factor value that is appropriate and conservative for DRAs and DWAs. In addition, the angle of the breakout cone and the location of where it initiates can be determined. It is currently unclear if the cracks initiate at the full depth of the anchors or if it is at a slightly shorter distance (i.e.: 80-90% of the bar depth).

Due to the time and cost of large-scale experimental tests, both full-scale and half-scale tests are proposed. If the behavior between the two types of tests are comparable, then more half-scale tests can be conducted. A proposed preliminary test matrix is shown in Table 2. Tests 1-4 study DRAs, while 5-8 experiment on DWAs. Half-scale tests are shown in Tests 1-2 and 5-6. With subsequent tests, additional configurations could be studied (2x2, 3x3, 5x5, etc.) as well as bar sizes and spacing. In additional, bent bars may be tested as well.

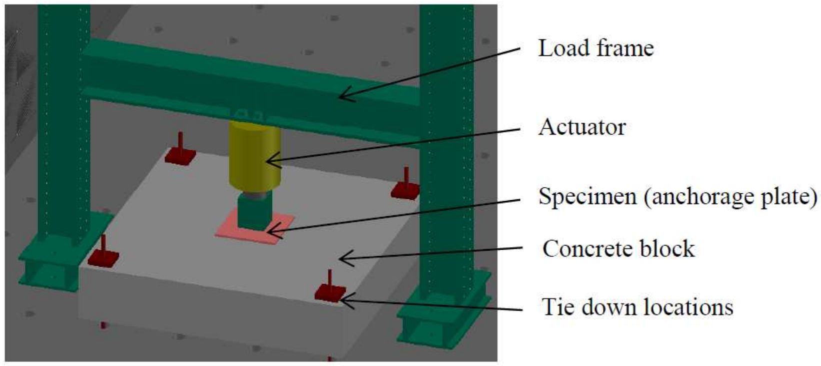


Figure 3 - Experimental test set-up.

Table 2 - Proposed test matrix for group anchors

Test #	Configuration	Bar type	Bar size	Spacing (in)
1	4x4	DRA	#3	3
2	4x4	DRA	#3	3
3	4x4	DRA	#6	6
4	4x4	DRA	#6	6
5	4x4	DWA	#3	3
6	4x4	DWA	#3	3
7	4x4	DWA	#6	6
8	4x4	DWA	#6	6

The bond behavior between the anchor and the concrete should also be verified experimentally. Chapter 3 of the CEB-FIP model code (2010) provides a method for calculating the relationship between local bond stress and slip. Depending on the presumed bond condition type (good or other), the bond stress-slip behavior can vary significantly. Experimental testing can aid in better understanding this relationship. Single bar pullout tests can be conducted, which would be tested in the same manner as the group anchor tests. Bars shorter than, equal to and greater than the code-prescribed development length of the bar should be tested.

Analytical modelling can also be used to conduct a more expansive parametric study. Experimental results can be used to calibrate three-dimensional finite element method (FEM) models. The bond stress-slip of the experimentally tested single bar anchors can be input into the cohesive model to simulate bond behavior. ABAQUS (2016), a commercially available FEM software, contains a built-in surface-based cohesive interaction model to capture this bond behavior.

CONCLUSIONS

In the absence of experimental data, it has been argued that if the reinforcing bars are fully developed (i.e., provided with development length) in group anchorage assemblies, then no further design checks are needed. However, the issue of whether concrete breakout failure modes should be considered in the design of such connections has been raised. The ACI 349 code is silent about the necessity of checking embed plates equipped with fully developed welded reinforcing bars for concrete breakout. A simple design example using the anchorage equations from the ACI 349 provisions shows that there is the potential for concrete breakout failures to occur. Work is currently underway by the authors to experimentally and analytically understand this breakout failure mode more closely. The results of such findings will be expressed in a subsequent paper.

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