

INTRODUCTION TO CONCRETE ANCHOR RODS

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## **Abstract**

Concrete anchors represent an important transition for both the design and construction of a project. Anchors are produced in two main categories: cast-in-place and post-installed. For designers, anchors are used to attach steel members to supporting concrete members. The anchors are designed using the provisions outlined in Building Code Requirements for Structural Concrete, ACI 318-11, Appendix D. These anchors are used to transmit tension and shear forces by using an individual anchor or a multiple anchor group. For contractors, anchor installation marks the transition between concrete and steel construction. Various types of anchors are produced by manufacturers, requiring contractors to be familiar with many installation methods. Careful planning and coordination is necessary to layout and place anchors into their correct location. Once anchors are installed, they must be protected from damage resulting from moving machinery and material. The purpose of this report is to introduce the basics to design concrete anchors by outlining the provisions in ACI 318-11 Appendix D and demonstrating these provisions with design examples. Anchor types, applications and common construction issues important to the structural engineer are also discussed.

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## List of Symbols

$A_{brg}$	= net bearing area of the head of a stud or anchor bolt, in. <sup>2</sup>
$A_{Nc}$	= projected concrete failure area of a single anchor or group of anchors, for calculation of strength in tension, in. <sup>2</sup>
$A_{Nco}$	= projected concrete failure area of a single anchor, for calculation of strength in tension if not limited by edge distance or spacing, in. <sup>2</sup>
$A_s$	= area of reinforcing steel, in. <sup>2</sup>
$A_{se,N}$	= effective cross-sectional area of anchor in tension, in. <sup>2</sup>
$A_{se,V}$	= effective cross-sectional area of anchor in shear, in. <sup>2</sup>
$A_{Vc}$	= projected concrete failure area of a single anchor or group of anchors, for calculation of strength in shear, in. <sup>2</sup>
$A_{Vco}$	= projected concrete failure area of a single anchor, for calculation of strength in shear, if not limited by corner influences, spacing, or member thickness, in. <sup>2</sup>
$c_{ac}$	= critical edge distance required to develop the basic strength as controlled by concrete breakout, in.
$c_{a,max}$	= maximum distance from center of an anchor shaft to the edge of concrete, in.
$c_{a,min}$	= minimum distance from center of an anchor shaft to the edge of concrete, in.
$c_{a1}$	= distance from the center of an anchor shaft to the edge of concrete in one direction, in.
$c_{a2}$	= distance from center of anchor shaft to the edge of concrete in the direction perpendicular to $c_{a1}$ , in.
$d_a$	= outside diameter of anchor or shaft diameter of headed stud, headed bolt, or hooked bolt, in.
$\sqrt{f'_c}$	= square root of specified compressive strength of concrete, psi
$f_{uta}$	= specified tensile strength of anchor steel, psi
$f_y$	= specified yield strength of reinforcement, psi
$f_{ya}$	= specified yield strength of anchor steel, psi
$h_a$	= thickness of member in which an anchor is located, measured parallel to anchor axis, in.
$h_{ef}$	= effective embedment depth of anchor, in.



$k_c$	= coefficient for basic concrete breakout strength in tension
$k_{cp}$	= coefficient for pryout strength
$l_e$	= load bearing length of anchor for shear, in.
$n$	= number of anchors
$n_t$	= number of threads per inch
$N_b$	= basic concrete breakout strength in tension of a single anchor in cracked concrete, lbs.
$N_{cb}$	= nominal concrete breakout strength in tension of a single anchor, lbs.
$N_{cbg}$	= nominal concrete breakout strength in tension of a group of anchors, lbs.
$N_n$	= nominal strength in tension, lbs.
$N_p$	= pullout strength in tension of a single anchor in cracked concrete, lbs.
$N_{pn}$	= nominal pullout strength in tension of a single anchor, lbs.
$N_{sa}$	= nominal strength of a single anchor or individual anchor in a group of anchors in tension as governed by the steel strength, lbs.
$N_{sb}$	= side-face blowout strength of a single anchor, lbs.
$N_{sbg}$	= side-face blowout strength of a group of anchors, lbs.
$N_{ua}$	= factored tensile force applied to anchor or individual anchor in a group of anchors, lbs.
$N_{ua,g}$	= total factored tensile force applied to anchor group, lbs.
$s$	= center-to-center spacing of anchors, in.
$V_b$	= basic concrete breakout strength in shear of a single anchor in cracked concrete, lbs.
$V_{cb}$	= nominal concrete breakout strength in shear of a single anchor, lbs.
$V_{cbg}$	= nominal concrete breakout strength in shear of a group of anchors, lbs.
$V_{cp}$	= nominal concrete pryout strength of a single anchor, lbs.
$V_{cpg}$	= nominal concrete pryout strength of a group of anchors, lbs.
$V_n$	= nominal shear strength, lbs.
$V_{sa}$	= nominal shear strength of a single anchor or individual anchor in a group of anchors as governed by the steel strength, lbs.
$V_{ua}$	= factored shear force applied to a single anchor or group of anchors, lbs.
$V_{ua,g}$	= total factored shear force applied to anchor group, lbs.

$\lambda_a$	= modification factor reflecting the reduced mechanical properties of lightweight concrete in certain concrete anchorage applications
$\Phi$	= strength reduction factor
$\Psi_{c,N}$	= factor used to modify tensile strength of anchors based on presence or absence of cracks in concrete
$\Psi_{c,P}$	= factor used to modify pullout strength of anchors based on presence or absence of cracks in concrete
$\Psi_{c,V}$	= factor used to modify shear strength of anchors based on presence or absence of cracks in concrete and presences or absence of supplementary reinforcement
$\Psi_{cp,N}$	= factor used to modify tensile strength of post-installed anchors intended tor use in uncracked concrete without supplementary reinforcement to account for the splitting tensile stresses due to installations
$\Psi_{ec,N}$	= factor used to modify tensile strength of anchors based on eccentricity of applied loads
$\Psi_{ec,V}$	= factor used to modify shear strength of anchors based on eccentricity of applied loads
$\Psi_{ed,V}$	= factor used to modify shear strength of anchors based on proximity to edges of concrete member
$\Psi_{h,V}$	= factor used to modify shear strength of anchor located in concrete members with $h_a < 1.5 c_{a1}$
$\Omega_o$	= amplification factor to account for overstrength of the seismic-force-resisting system determined in accordance with the legally adopted general building code

## **Acknowledgements**

I would like to thank all my professors and teachers who have guided me through my academic career. Your help and motivation has made this report possible.

## **Dedication**

This report is dedicated to my parents, Mark and Trasenda, and my brother, Regan. I would like to thank my father for introducing me to the field of construction and engineering and my mother for teaching me the importance of a strong work ethic.

## Chapter 1 - Introduction

The design of concrete anchor rods is governed by the adopted building codes of the local jurisdiction. For many jurisdictions the adopted building code is the International Building Code, IBC. Concrete anchors are covered in Section 1909 of the 2012 IBC, which then references American Concrete Institutes Building Code Requirements for Structural Concrete ACI 318-11 Appendix D for the strength design of anchors, hereinafter Appendix D.

Concrete anchors serve a unique purpose in structural design and construction, as they function as the transition between steel and concrete members. The anchors have to be able to transmit axial, shear and moment forces between the two structural members. The most common use of anchors is attaching steel columns or light poles to concrete foundations as seen in Figure 1. However, anchors can also be used in many applications such as overhead hangers or in horizontal embed plates or ledger beams. Anchors can be used individually or in groups depending on the application and load requirements. Hangers will typically use a single anchor at a specified spacing, while column attachments or embed plates use multiple anchor groups. The purpose of this report is to introduce concrete anchors and the design provisions of Appendix D, while also providing multiple design examples. The introduction to Appendix D will focus on basic cast-in-place anchors, with the provisions of post installed anchors being outside the scope of this report. This report covers common anchor types, applications, and materials in Chapter 2. The basics of the cast-in-place anchor provisions of Appendix D are outlined in Chapter 3. Common construction issues for cast-in-place anchors are discussed in Chapter 4. Design examples demonstrating the provisions of Appendix D are shown in Chapter 5. Practicing structural engineers need to be able to both understand the basic code provisions and address construction issues in the field in a timely manner.



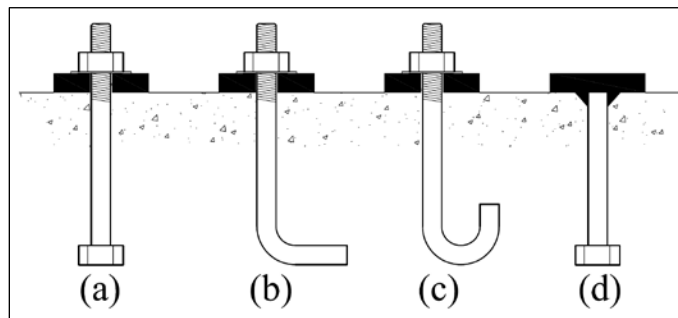
**Figure 1 - Stadium Light Pole Attachment**

## Chapter 2 - Types, Applications and Materials

Concrete anchors come in two main types: cast-in-place and post installed. Cast-in-place anchors have traditionally been the anchor of choice for structural engineers for both large and small projects. However, proprietary post installed anchors have recently become popular in the construction industry for their ease of installation. It is important to be familiar with both cast-in-place and post-installed anchors types. Even if cast-in-place anchors are chosen for a design, construction issues may demand a fast post-installed anchor design. For this reason, both anchor types are introduced in this chapter.

### Cast-In-Place Anchors

Cast-in-place anchors are a non-proprietary product that can be designed with basic steel and concrete mechanics. They are set in place along with steel reinforcement prior to the concrete placement. Anchor groups may be set using a steel or plywood template to insure proper geometry and placement. Cast-in-place anchors come in several shapes and sizes including headed hex bolt, hooked J- and L- bolt, and threaded rod with sizes ranging from 1/4" up to 4" in diameter. These types of anchors can be seen in Figure 2. The most recommended anchor rod for commercial construction, according to American Institute of Steel Construction, AISC, is a straight rod with hex head or threaded nut with minimum rod diameter of 3/4" (AISC DG1, 2010).



**Figure 2 - Cast-In-Place Anchors:**

**(a) hex head bolt; (b) L-bolt; (c) J-bolt; (d) welded headed stud**

As with other steel products, cast-in-place anchors come in a variety of material strengths. ASTM F1544 outlines three grades: Gr. 36, Gr. 55, Gr. 105. Other common material

strengths for anchors can be found in Appendix A. The most common anchor material is Gr. 36, as it is the most economical material and readily available compared to the higher strength steels (AISC, 2010). The grade of steel is commonly kept constant over the entire project with only the size of the anchor varying. This allows for the contractor to easily differentiate between anchors. However, if different grades are used on a project, ASTM F1544 Section 19 requires the anchors to be color coated to easily determine the grade of steel. The color codes are given in Table 1.

**Table 1 - Cast-in-Place Color Codes (AISC, 2010)**

Grade	Color
36	Blue or Blank
55	Yellow
105	Red

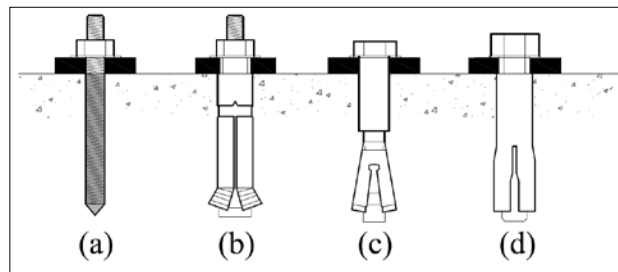
Cast-in-place anchors can be used in most anchor applications. Once cast into the concrete, these anchors form a strong and reliable mechanical bond with the surrounding concrete. Cast-in-place anchors are recommended when the loads applied require large embedment lengths and high tensile strength. Common situations requiring large cast-in-place anchors are heavy columns, bridges, or light poles with uplift forces. An example for cast-in-place anchors shown in Figure 3 is the attachment of a stadium light pole to a concrete foundation. Cast-in-place anchors are also used for smaller applications such as embed plates or wood sill plates.



**Figure 3 – Cast-In-Place Anchor Group**

## Post-Installed Anchors

Post-installed anchors are a proprietary product manufactured by several companies such as Hilti, Simpson Strong-Tie or Read Head. Post-installed anchors are available in mechanical or adhesive bonds. These anchors are installed into predrilled holes after the concrete has cured. Typical post-installed anchor types can be seen in Figure 4. With many types and manufacturers of post-installed anchors a standard testing procedure is outlined in ACI 355. The testing results in an ICC report describing all properties of the anchor. These tests have to demonstrate a predictable and acceptable failure for the anchor to qualify as a post-installed anchor in Appendix D. If the anchor qualifies it can be designed with the provisions outlined in Appendix D. Each manufacturer also has specific installation methods for each anchor they produce to insure a quality bond is created with the hardened concrete. This installation often includes using a rotary hammer to drill a hole in the hardened concrete. The hole is then cleaned with a brush and/or compressed air. It is very important to remove all the dust in the hole as the dust acts as a bond breaker between the anchor and concrete. While these methods can be cumbersome to follow, post-installed anchors do offer the flexibility to move or change the anchor group location after the foundation is poured. Installing the anchors after the pour requires detailed planning to avoid the concrete reinforcement when drilling the holes for the post-installed anchors. One advantage to post-installed anchors can be found in the construction schedule. Many fast track projects have multiple bid packages where the concrete foundation is poured before the steel superstructure design is completed. This is not possible to do with cast-in-place anchors, as the exact anchor layout may not be known at the time of the pour. However, the post-installed anchors could be installed once the steel superstructure design is completed. The decision to use a post-installed anchor over a cast-in-place anchor will need to be by both the design engineer and contractor based on performance and cost.



**Figure 4 - Post-Installed Anchors:**

**(a) Adhesive anchor; (b) undercut anchor; (c)(d) torque-controlled expansion anchors**



## **Chapter 3 - Appendix D Overview**

The design of concrete anchors has evolved from allowable stress design reference tables into very detailed and comprehensive strength design provisions. Appendix D is constructed to cover both cast-in-place and post-installed anchors, single anchors and complex layouts of multiple anchor groups, varying edge distances and a combination of tension, shear, and eccentric loadings. As a result, the designer is required to be familiar with the entire appendix to follow the multi-step process for each anchor design. This chapter is an overview of the basic concepts and variables needed to understand cast-in-place anchor design. Post-installed anchor design is has recently been added to Appendix D and is outside the scope of this chapter. However, many of the basic mechanics of cast-in-place anchor design apply to post-installed anchors. Appendix D outlines many equations, limitations and exceptions that are not repeated in this general overview.

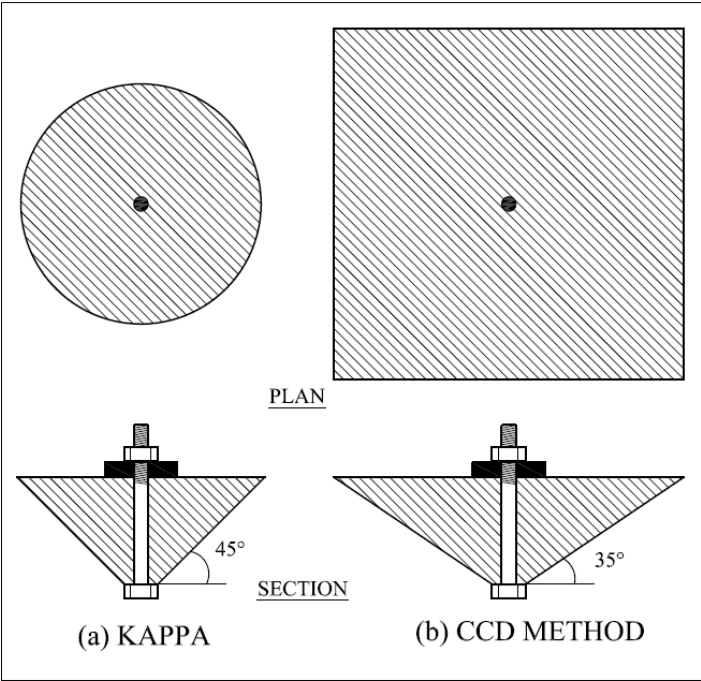
### **History of Building Codes and Philosophies**

The design of concrete anchors has long been absent in both the concrete, ACI 318, and steel, AISC Specifications, building codes. The first main document to cover the design of cast-in-place anchors was the First Edition of the PCI Design Handbook in 1971 (Anderson, 2007). Information was soon included in both the ACI 349 Appendix B and the Uniform Building Code in the form of empirical design tables. It was not until the 1990's that the ACI Committee 318 started to develop a comprehensive building code for anchors. These building codes required research on the design of both cast-in-place and post-installed anchors. Research was completed by two committees: ACI 355 Anchorage to Concrete, and ACI 349 Concrete Nuclear Structures. ACI 318 and ACI 355 attempted to finalize a new Appendix for the ACI 318-99 building code (PCA, 2008). However, the post-installed test method for evaluating the performance of post-installed anchors was not complete in time for the ACI 318-99 release. Since the entire Appendix including both cast-in-place and post-installed anchors was not completed in time to be referenced into the International Building Code, IBC 2000, only the cast-in-place portion of the research was integrated into Section 1913 of the IBC 2000.

The research discussed above was completed for both cast-in-place and post installed anchors, the ACI 318-02 included a new Appendix D titled Anchoring to Concrete. The new

appendix was referenced in the IBC 2003 Section 1913 (PCA, 2008). As the 2005, 2008, and 2011 versions of the ACI 318 were released, cast-in-place anchors saw only minor changes. However, the post-installed anchor scope has now grown to incorporate both mechanical and adhesive anchors (PCI, 2008).

The actual design philosophy and methods behind the building codes have changed many times since the 1970's. Early research in the 1980's performed at the University of Stuttgart included a 45 degree breakout cone and analyzed how capacities differed with varying edge distances, embedment lengths and group effects (Anderson, 2007). The Stuttgart research resulted in the development of the 45 degree cone Kappa method. The Kappa method was then improved on to make the calculation process more user-friendly (Fuchs, 1995). The name of this improved method is Concrete Capacity Design, CCD. With all of these different methods and philosophies to design concrete anchors, the ACI committees needed to decide upon a single method to adopt in their building code appendix. In the mid 1990's an international database of test results was compiled and the 45 degree cone method was compared to the new CCD method (Fuchs, 1995). These two design methods can be seen in Figure 5.



**Figure 5 - Design Methods**

The main difference between the two methods can be found in the concrete breakout failure mode. The 45 degree cone method uses a smaller cone than the CCD method which uses

a 35 degree cone. The CCD method also simplified how a group of anchors is analyzed by using a rectangular area for concrete breakout instead of overlapping circular cones. When comparing the two methods to the test results database, it was found that CCD method gave a better prediction for anchor strength at a given embedment. After comparing these two methods the CCD method was selected and implemented in the Appendix D of the ACI 318-02 building code (PCA, 2008).

## **General Requirements**

The first section in Appendix D outlines the general requirements for concrete anchors including the theory of design, seismic design requirements, strength reduction factors and introduces the different anchor failure modes. Appendix D is based on the theory of elasticity, meaning the attachment such as a base plate is rigid enough to transfer the loads to each anchor in the anchor group without yielding. The force in each anchor is proportional to the external load applied and the distance from the anchor to the neutral axis of the group. The required strength,  $R_u$ , is found using the LRFD load combinations in Section 9.2 of ACI 318. The factored anchor capacity,  $\Phi R_n$ , is found using Appendix D and shall be equal to or greater than the required strength.

$$\Phi R_n \geq R_u \qquad \text{Eq 3-1}$$

### *Seismic Overview*

Brittle concrete failures do not allow for the dissipation of energy in an seismic event, thus, the ACI 318 outlines specific requirements for seismic loading in Section D3.3 of Appendix D. These requirements were put in place to ensure a ductile failure mode, ideally the steel anchor, when seismic loading is applied. These requirements apply when the structure is assigned to Seismic Design Category C, D, E, or F even if the governing load combination does not include a seismic component. The requirements also apply if the seismic component is greater than 20 percent of the factored force, regardless of Seismic Design Category. When the steel anchor rods are designed to fail in a ductile manner, any attachments to the anchor should be designed not to yield.

The seismic requirements for tensile loading are found in Section D3.3.4. The requirements outline four options for the designer to insure the anchor fails in a ductile manner

or has enough strength to remain elastic during the seismic event. Two of these options result in a reduction in tensile capacity.

- a) The steel failure modes shall control the design of the anchor. The various concrete failure modes outlined in Appendix D all have a higher capacity than the steel failure mode. If the steel yields before the concrete fails, no reduction is needed for tensile loading. The anchor must be made of ductile steel per Section D.1. Having the anchor rods perform in a ductile manner as required by this option can result in the designer selecting smaller or weaker anchor. An example would be using many 3/8" diameter and Grade 36 steel anchors with deep embedment length to insure that the concrete strength exceeds the steel strength.
- b) The attachment, such as a base plate, shall develop a ductile yield mechanism to dissipate energy. If designing for the plate to yield, careful consideration should be given to the difference between specified yield strength and actual yield strength. The anchors are then designed for the maximum tension that can be transmitted to the group by the attachment. A reduction in tensile capacity must be made. This option can be complex to design, as yield lines have to be followed through the attachment. The individual anchor rod forces no longer follow the theory of elasticity as the attachment is yielding and deforming.
- c) The anchors are designed for the maximum force transmitted by a non-yielding attachment. A reduction in tensile capacity must be made. This is a special case when the attachment failure is non-yielding, such as crushing or rupture.
- d) The anchors are designed using load combinations that include seismic and the corresponding overstrength factor,  $\Omega_o$ , given in ASCE 7-10 Section 12.4.3.2. If the anchors are designed for the overstrength force, then no reduction in tensile capacity has to be made. This is an elastic option that does not allow for any energy dissipation or yielding in the anchor rods.

If option (b) or (c) is chosen, the reduction in tensile capacity is given in Section D3.3.4.4. The 0.75 strength reduction factor is due to cracked concrete and is applied to the governing concrete failure mode, but not the steel failure mode. Cracking around the anchor group will reduce the capacity as the bond is weakened between the anchor and concrete. Generally cracking cannot be avoided in an extreme cyclic event.

The seismic requirements for shear loading are found in Section D3.3.5. The requirements outline three options for the designer. These three options are similar to options ‘b’, ‘c’ and ‘d’ under the tensile requirements. The ductile failure option ‘a’ for tensile loading is not available, as anchors failing in shear generally do not dissipate large amounts of energy.

### ***Strength Reduction Factors***

The failure modes covered by Appendix D include both steel and concrete modes. Having these two different materials, along with both tension and shear forces results in many strength reduction factors being used throughout Appendix D. The factors are described in Section D.4.3 and are broken down into three main categories of failure modes: ductile steel, brittle steel, and concrete. Then, each category is further itemized by the loading causing the failure: tension or shear. Table 2 organizes the factors for clarity.

**Table 2 - Strength Reduction Factors**

<b>Strength Governed by</b>	<b>Strength Reduction Factor, <math>\Phi</math></b>	
<b>Ductile Steel Element</b>		
Tension, $N_{sa}$	0.75	
Shear, $V_{sa}$	0.65	
<b>Brittle Steel Element</b>		
Tension, $N_{sa}$	0.65	
Shear, $V_{sa}$	0.60	
<b>Concrete</b>		
	Condition	
	A	B
Shear		
Breakout, $V_{cb}$ and $V_{cbg}$	0.75	0.70
Pryout, $V_{cp}$	0.70	0.70
Tension		
Breakout and Side-face Blowout, $N_{cb}$ , $N_{cbg}$ , $N_{sb}$ and $N_{sbg}$	0.75	0.70
Pullout, $N_{pn}$	0.70	0.70

After reviewing these reduction factors, several items stand out. The steel strength reduction factors seem low relative to the factors provided in the AISC *Steel Construction Manual*. This is because Appendix D uses the higher ultimate strength,  $f_{uta}$ , instead of the yield strength,  $f_{ya}$  for steel. Using the higher ultimate strength for steel in combination with a lower strength reduction factor yields the same factor of safety. Another observation is that shear

loading has a lower strength reduction factor than tensile loading, which usually is a result of a more volatile brittle failure in shear compared to a ductile failure in tension. However, the main reason for a lower shear strength reduction factor is that ACI expects non-uniform loading of anchors at the perimeter of anchor groups. The non-uniform loading results in the perimeter anchors failing before the interior anchors reach their full strength. For the concrete strength reduction factors, ACI provides two reinforcement conditions, one with (A) and one without (B) supplementary reinforcement. If reinforcement is provided around the anchor, a more ductile failure mode is anticipated. This results in a smaller reduction for Condition A, resulting in a larger capacity.

### **Design Requirements for Tensile Loading**

Appendix D outlines four failure modes for anchors loaded in tension: steel strength, concrete breakout strength, pullout strength, and concrete side-face blowout strength. All four of these failure modes should be checked and the lowest of the four strengths will govern the design in tension. All four are checked when only a single anchor is analyzed. However, when the designer is analyzing a group of anchors, two of the failure modes are checked for a single anchor only, while the other two are checked as an anchor group. Table 3 outlines which modes need to be checked for a specific situation.

**Table 3 - Tensile Design Checks**

<b>Failure Mode</b>	<b>Single Anchor</b>	<b>Anchor Group</b>	
		Individual Anchor in a Group	Anchors as a Group
Steel strength (D.5.1)	$\Phi N_{sa} \geq N_{ua}$	$\Phi N_{sa} \geq N_{ua,i}$	
Concrete breakout strength (D.5.2)	$\Phi N_{cb} \geq N_{ua}$		$\Phi N_{cbg} \geq N_{ua,g}$
Pullout strength (D.5.3)	$\Phi N_{pn} \geq N_{ua}$	$\Phi N_{pn} \geq N_{ua,i}$	
Concrete side-face blowout strength (D.5.4)	$\Phi N_{sb} \geq N_{ua}$		$\Phi N_{sbg} \geq N_{ua,g}$

#### ***Steel Strength of Anchor in Tension***

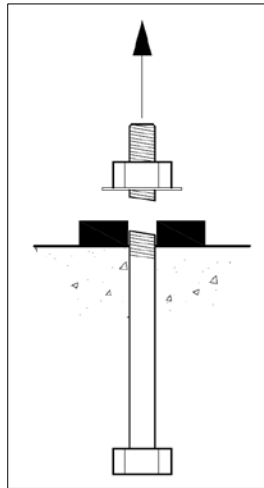
Tensile strength of a steel anchor is covered in Section D.5.1 and depends on the dimensional and material properties of the anchor. This failure can be seen in Figure 6. The nominal strength,  $N_{sa}$ , can be calculated by using Equation D-2 below:

$$N_{sa} = A_{se,N} f_{uta} \quad \text{Eq 3-2}$$

Where  $A_{se,N}$  is the effective cross-sectional area and  $f_{uta}$  is the specified tensile strength of the anchor steel. The effective cross-sectional area is used to account for the loss of area due to anchor threads. The ACI Commentary suggests using the following equation for  $A_{se,N}$ :

$$A_{se,N} = \frac{\pi}{4} \left( d_a - \frac{0.9743}{n_t} \right)^2 \quad \text{Eq 3-3}$$

Where  $n_t$  is the number of threads per inch, the  $n_t$  equation can be found at the bottom of Table 7-18 of the AISC *Steel Construction Manual*. This equation has been tabulated in the both Table 7-18 and the PCA Notes and is included in Appendix A of this report. Common material strengths for anchors are also in Appendix A.



**Figure 6 - Steel Failure  
In Tension**

### ***Concrete Breakout Strength of Anchor in Tension***

Concrete breakout strength of anchors subjected to tensile loads is outlined in Section D.5.2, which can be a very tedious and detailed calculation. This failure can be seen in Figure 7. Breakout is what is traditionally thought of as the failure mode for anchors, as it utilizes the 35 degree failure cone. Concrete breakout can only occur if the anchors do not fail in tensile yielding or pullout. Nominal concrete breakout strength,  $N_{cb}$ , is influenced by several variables as can be seen in equations D-3 and D-4:

Single anchor:

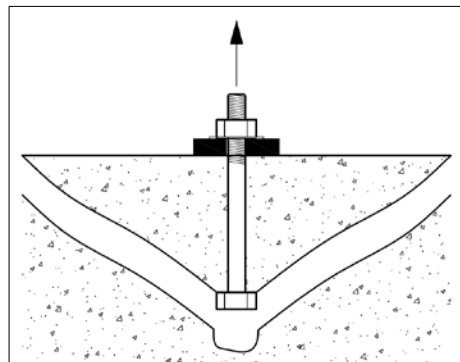
$$N_{cb} = \frac{A_{Nc}}{A_{Nco}} \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_b \quad \text{Eq 3-3}$$

Group of anchors:

$$N_{cbg} = \frac{A_{Nc}}{A_{Nco}} \Psi_{ec,N} \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_b \quad \text{Eq 3-4}$$

$N_b$  is the basic concrete breakout strength of a single anchor. This strength is then adjusted with several factors to reflect the specific design under consideration. The ratio of  $A_{Nc}$  to  $A_{Nco}$  is accounting for the increased breakout strength found when a group of anchors act together developing a larger failure cone. It also considers the overlapping of failure cones in multiple anchor groups, since each individual anchor will not be able to develop a full breakout cone.  $A_{Nc}$  is the projected concrete failure area for the specific layout being designed. It is determined from the layout geometry and embedment of the anchor group.  $A_{Nco}$  is the theoretical projected concrete failure area of a single anchor with no edge distance limitations.  $A_{Nco}$  can be understood as the perfect, uninterrupted failure area of a single anchor. The ratio of  $A_{Nc}$  to  $A_{Nco}$  then usually results in an increase factor based on the number of anchors used in a group and the layout geometry. The limitations and equations for  $A_{Nc}$  and  $A_{Nco}$  can be found in Section D.5.2.1 and Fig. RD.5.2.1 in Appendix D.

Several modification  $\Psi$  factors are also used for concrete breakout strength.  $\Psi_{ec,N}$  in Section D.5.2.4 is the modification factor for anchor groups loading eccentrically in tension and does not apply to a single anchor.  $\Psi_{ed,N}$  in Section D.5.2.5 is the modification factor for edge effects for individual or groups of anchors.  $\Psi_{c,N}$  in Section D.5.2.6 is the modification factor for uncracked concrete.  $\Psi_{cp,N}$  in Section D.5.2.7 is the modification factor to limit concrete splitting and is equal to one for cast-in-place anchor design.



**Figure 7 - Concrete Breakout Failure In Tension**



Concrete breakout strength can be a lengthy calculation if a complex layout requires consideration to all of these modification factors. To reduce the length of calculation, an alternative is outlined in Section D.5.2.9. If anchor reinforcement is developed around the anchor or anchor group, the designer can use the reinforcement strength as the concrete breakout strength. A strength reduction factor of 0.75 should be used in the design of anchor reinforcement. In most situations reinforcement is present in the concrete that could qualify as anchor reinforcement. Anchor reinforcement is shown in Fig. RD.5.2.9 in the Appendix D.

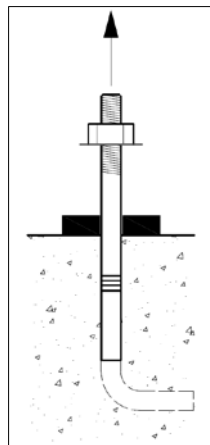
### ***Pullout Strength of Anchor in Tension***

Pullout strength of anchors subjected to tensile loading is covered in Section D.5.3.1 and depends on type of anchor used and concrete cracking. This failure can be seen in Figure 8. Pullout is calculated for an each individual anchor and has no group effects. The nominal pullout strength,  $N_{pn}$ , of a single anchor can be calculated by using equation D-13:

$$N_{pn} = \Psi_{c,P} N_p \quad \text{Eq 3-5}$$

$N_p$  is the pullout strength of the anchor and is dependent on the anchor bearing area and concrete compressive strength. The anchor bearing area of cast-in-place anchors is dependent on the type and shape of anchor used.  $\Psi_{c,P}$  is the modification factor for uncracked concrete, as the concrete is assumed to be cracked. One example of cracked concrete is for hanger applications on the tension face of a beam.

The design goal is to provide a large enough bearing area at the bottom of the anchor rod to develop the full concrete breakout cone above. If pullout strength is governing, the anchor will simply pullout of the concrete without a breakout cone.



**Figure 8 - Pullout Failure In Tension**

### ***Concrete Side-Face Blowout Strength of Anchor in Tension***

Concrete side-face blowout strength is outlined in Section D.5.4.1 and depends on embedment depth, edge distance and bearing area. This failure can be seen in Figure 9. The nominal side-face blowout strength,  $N_{sb}$ , can be calculated by using Equation D-16 and D-17:

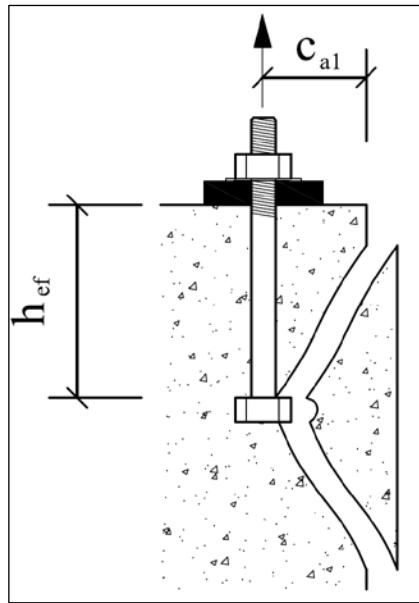
For a single anchor:

$$N_{sb} = (160c_{a1}\sqrt{A_{brg}})\lambda_a\sqrt{f'_c} \quad \text{Eq 3-6}$$

For a group of anchors:

$$N_{sbg} = \left(1 + \frac{s}{6c_{a1}}\right) N_{sb} \quad \text{Eq 3-7}$$

$A_{brg}$  is the bearing area of the embedded head and is equal to the gross area of the head, less the gross area of the anchor shaft. This limit state only applies when an anchor is close to an edge and has a deep embedment  $h_{ef} > 2.5c_{a1}$ . Otherwise, this failure mode can be ignored.



**Figure 9 - Concrete Side-Face Blowout Failure In Tension**

## Design Requirements for Shear Loading

Appendix D outlines three failure modes for shear loading of anchor rods: steel strength, concrete breakout strength, and concrete pryout strength. All three of these failure modes should be checked and the lowest of the three strengths will govern the design in shear. All three are checked when only a single anchor is analyzed. However, when the designer is analyzing a group of anchors, one of the failure modes is checked for a single anchor only, while the other two are checked as an anchor group. Table 4 outlines which modes need to be checked for specific situations.

**Table 4 - Shear Design Checks**

Failure Mode	Single Anchor	Anchor Group	
		Individual Anchor in a Group	Anchors as a Group
Steel strength (D.6.1)	$\Phi V_{sa} \geq V_{ua}$	$\Phi V_{sa} \geq V_{ua,i}$	
Concrete breakout strength (D.6.2)	$\Phi V_{cb} \geq V_{ua}$		$\Phi V_{cbg} \geq V_{ua,g}$
Concrete pryout strength (D.6.3)	$\Phi V_{cp} \geq V_{ua}$		$\Phi V_{cpg} \geq V_{ua,g}$

### *Steel Strength of Anchor in Shear*

Steel strength of anchors subjected to shear loading is covered in Section D.6.1 and depends on the anchor steel strength and cross-sectional dimensions. This failure can be seen in Figure 10. The nominal strength,  $V_{sa}$ , can be calculated using Equation D-28 and D-29:

For welded cast-in headed stud:

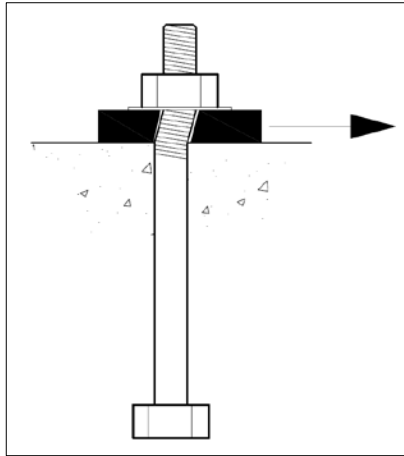
$$V_{sa} = A_{se,V} f_{uta} \quad \text{Eq 3-8}$$

For cast-in headed bolt and hooked bolts:

$$V_{sa} = 0.6 A_{se,V} f_{uta} \quad \text{Eq 3-9}$$

$A_{se,V}$  is the effective cross-sectional of the anchor in shear and  $f_{uta}$  is the steel ultimate strength. The effective area, found in Appendix A of this report, is used to account for the bolt threads in the shear plane. As can be seen in the equations above, the welded cast-in headed studs have a higher shear capacity than the headed or hooked bolts. This is due to the greater fixity provided by the weld between the studs and the base plate, compared to a bolted connection.

When built up grout pads are used in column base plate design the shear strength of the anchor is reduced by 20% by applying a 0.8 reduction factor per Section D.6.1.3. The built up grout pad produces a moment arm for the base plate shear force. This reduction is made to account for the flexural stresses induced when the grout cracks and is no longer supporting the anchor.



**Figure 10 - Steel Failure In Shear**

### ***Concrete Breakout Strength of Anchor in Shear***

Concrete breakout strength of an anchor subjected to shear loading is outlined in Section D.6.2.1 and depends on many of the same variables used for concrete breakout strength under tensile loading. However, one main difference is shear can be induced in two directions: perpendicular to the free edge or parallel to the free edge. This failure can be seen in Figure 11. To account for direction, the nominal concrete breakout strength,  $V_{cb}$ , is double when the shear is parallel to an edge.  $V_{cb}$  can be calculated using the equations:

For shear force perpendicular to the edge on a single anchor:

$$V_{cb} = \frac{A_{vc}}{A_{vco}} \Psi_{ed,v} \Psi_{c,v} \Psi_{h,v} V_b \quad \text{Eq 3-10}$$

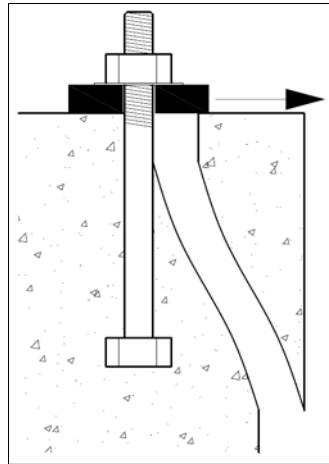
For shear force perpendicular to the edge on a group of anchors:

$$V_{cbg} = \frac{A_{vc}}{A_{vco}} \Psi_{ec,v} \Psi_{ed,v} \Psi_{c,v} \Psi_{h,v} V_b \quad \text{Eq 3-11}$$

$V_b$  is the basic concrete breakout strength of a single anchor. As with tension, the basic concrete breakout strength is adjusted to reflect the specific design under consideration. The ratio of  $A_{vc}$  to  $A_{vco}$  is performing a similar role in shear design as it accounts for the geometry of

multiple anchor groups. The limitations and equations for  $A_{Vc}$  and  $A_{Vco}$  can be found in Section D.6.2.1 and Fig. RD.6.2.1 in the Appendix D.

The modification factors  $\Psi_{ec,V}$  in Section D.6.2.4,  $\Psi_{ed,V}$  in Section D.6.2.5,  $\Psi_{c,V}$  in Section D.6.2.6, and  $\Psi_{cp,V}$  in section D.6.2.7 are applied similar for shear loading as with tensile loading.  $\Psi_{h,V}$  in Section D.6.2.8 is a modification factor to account for a concrete support member having a shallow depth. This factor was not used for tensile concrete breakout. Also similar to tensile loading, an alternative design is outlined in Section D.6.2.9. If anchor reinforcement is developed around the anchor or anchor group, the designer can use the reinforcement strength as the concrete breakout strength. Typical anchor reinforcement for shear is shown in Fig. RD.6.2.9 in Appendix D. An example of how to design shear anchor reinforcement is shown in Chapter 5 Example 5 of this report.



**Figure 11 - Concrete Breakout Failure In Shear**

### ***Concrete Pryout Strength of Anchor in Shear***

Concrete pryout strength of an anchor subjected to shear loading is covered in Section D.6.3. It depends directly on the concrete breakout strength found for tensile loading,  $N_{cb}$ . This failure can be seen in Figure 12. The nominal pryout strength,  $V_{cp}$ , can be calculated by using Equations D-40 and D-41:

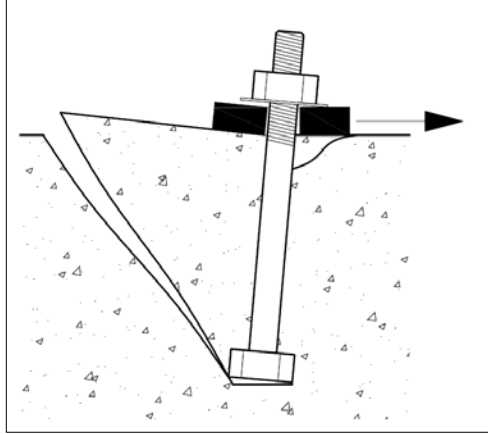
For a single anchor:

$$V_{cp} = k_{cp}N_{cb} \quad \text{Eq 3-12}$$

For a group of anchors:

$$V_{cpg} = k_{cp}N_{cbg} \quad \text{Eq 3-13}$$

$N_{cp}$  and  $N_{cpg}$  are equal to  $N_{cb}$  and  $N_{cbg}$  in Section D.5.2. The  $k_{cp}$  is a multiplier that depends on the embedment length, the longer the embedment length the greater the capacity. Pryout strength may govern when a large diameter anchor has shallow embedment.



**Figure 12 - Concrete Pryout Strength In Shear**

### **Interaction of Tensile and Shear Forces**

The interaction of tensile and shear forces is outlined in Section D.7 and depends on the percent stressed for each type of load. If the shear applied,  $V_{ua}$ , is less than 20% of the capacity,  $\Phi V_n$ , then the full tensile capacity,  $\Phi N_n$ , can be used. Similarly, if the tension applied,  $N_{ua}$ , is less than 20% of the capacity,  $\Phi N_n$ , then the full shear capacity,  $\Phi V_n$ , can be used. In both of these cases interaction between tension and shear is ignored.

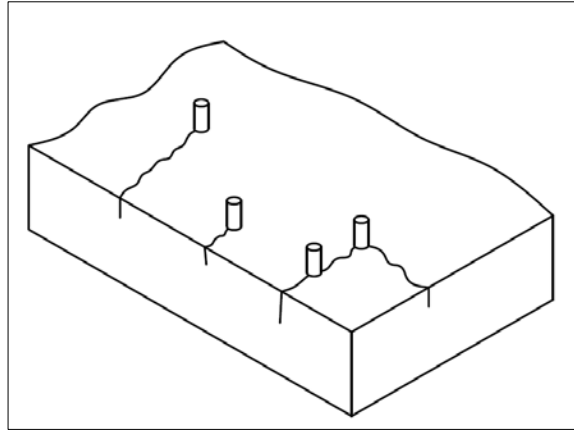
However if both the tensile and shear percent stressed values are greater than 20%, interaction between the two must be considered. When applicable, the interaction design criteria will govern the anchor design. This interaction is calculated by using Equation D-42:

$$\frac{N_{ua}}{\Phi N_n} + \frac{V_{ua}}{\Phi V_n} \leq 1.2 \quad \text{Eq 3-14}$$

### **Requirements to Preclude Splitting Failure**

Requirements to preclude splitting failure are covered in Section D.8. This failure can be seen in Figure 13. Requirements include a minimum center to center spacing of  $4d_a$  and recommends using concrete cover requirements of Section 7.7. A concrete cover of 1-1/2 inches is recommended for all cast-in anchors (PCA, 2008). If these minimums are not satisfied, a reduction is outlined in Section D.8.4 for the reduced strength of the anchor. This could be the

case when a larger diameter anchor is used in a closely spaced multiple anchor group. Splitting failure is a greater concern for the installation of post-installed anchors into hardened concrete than it is for cast-in-place anchors.



**Figure 13 - Splitting Failure**

### **International Building Code Topics**

The 2012 International Building Code (IBC) lists several modifications to ACI 318 in Section 1905. For Appendix D, the designer should delete Section D.3.3.4 through D.3.3.7 and D.4.2.2 and replace with the modified sections shown in the IBC. Appendix D Section D.3.3.4 through D.3.3.7 describes the requirements for seismic loading in tension and shear. The ACI 318-11 requirements for seismic design were updated and reformatted to include more design options for the engineer. The IBC 2012 modifications delete this update and replace it with the requirements from the ACI 318-08. This could be due to the IBC not agreeing with the new seismic requirements or not having enough time to review them before the printing of the 2012 IBC. The second modification is to Section D.4.2.2. This section outlines the anchor diameter and embedment depth limitations for the concrete breakout strength design equations. ACI 318-11 limits only the anchor diameter to 4", placing no limit on the anchor embedment depth. This is an update to the ACI 318-08 which limited both parameters: 2" diameter and 25" embedment depth. Again by not having enough time to review the changes, the IBC 2012 deletes the new limitations used in ACI318-11 and replaces them with the old limitations from ACI318-08.

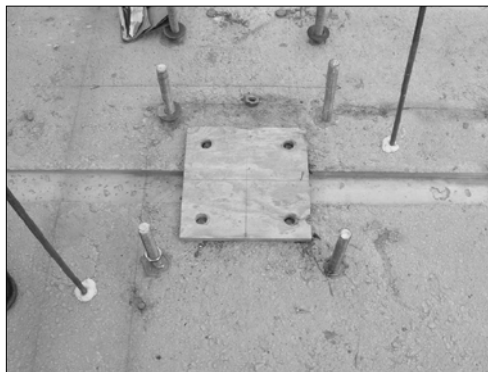
Concrete anchors also require a special inspection per IBC 2012 Section 1705 and Table 1705.3. This periodic special inspection is required if the cast-in-place anchors are designed using the strength design provisions from Appendix D (IBC, 2012).

## Chapter 4 - Constructability Issues and Solutions

Several construction issues can arise in the field for concrete anchor rods. As the engineer of record, being able to handle these issues in a timely manner is critical for the project schedule. Understanding common issues and their solutions will save the engineer and contractor time and money. Some of the most common problems are misplaced anchors, bent anchors, and anchors of incorrect length. When any of these construction issues arise, the structural engineer should be notified immediately. Any error in construction of the transition to steel construction can have a compounding effect as the same anchor detail is repeated throughout the project. Concrete tolerances for placement of embedded items can be found in *Specification for Tolerances for Concrete Construction and Materials* (ACI 117-10, 2010), while steel tolerances for placement of anchor rods can be found in *AISC Code of Standard Practice* (AISC 303-10, 2010). Both of these documents should be reviewed for the installation of anchor rods. However, after reviewing both documents the AISC tolerances are stricter than the ACI tolerances. Since both list specific tolerances for the same item, it is recommended the designer use the tighter tolerances in the project specifications (AISC DG1, 2010). This chapter discusses the most common anchor problems that can arise in the field.

### Misplaced Anchors

Anchor rods are typically used to connect steel attachments to concrete members. This attachment usually is a steel base plate with holes predrilled in a specific geometry. Since the same geometry of anchor rods is typically repeated for multiple columns, it is common to use a template to insure proper consistent anchor spacing. An example template can be seen in Figure 14. These templates should be firmly fastened to the formwork during the concrete pour.



**Figure 14 - Misplaced Anchors (Fisher, 2012)**



For a steel base plate, Table 14-2 in the AISC *Steel Construction Manual* recommends maximum sizes for oversized holes depending on the bolt size. If followed, these recommended hole sizes allow for easier column erection and setting. This table is reproduced in Table 5. Note that the washer sizes listed are usually custom fabricated. This increase in cost for larger custom washers may be offset by the faster installation and fewer fit-up problems for larger diameter holes. Even with these tight tolerances for anchor rods, a lack of planning in the field can lead to anchors missing the base plate holes or even having the wrong geometry. An example of misplaced anchors can be seen in Figure 14. This construction issue can be handled several ways and each should be considered for the specific situation.

If the problem is realized early, the solution can be to alter the steel base plate to fit the layout by fabricating a new plate or drilling larger holes in the original plate. Both of these solutions can be quite costly in the field, as the base plates are typically shop welded to the base of steel columns before arriving on site. If the problem is not realized before the steel is delivered to the site, modifying the anchor rods instead of the base plate may be more economical. If the cast-in-place anchor rods are not in the correct location, the contractor may choose to cut off the rods and use a post-installed anchor. Typically an epoxy anchor is used for this situation, by drilling new holes into the concrete member (Fisher, 2012). It is important to note that epoxy anchors may require a certified installer if the anchor is subjected to sustained tensile forces, such as hanger supports (ACI 318-11, 2011).

**Table 5 - Maximum Sizes for Anchor Rod Holes**

Anchor Rod Diameter	Max. Hole Diameter	Min Washer Size	Min. Washer Thickness
3/4	1 1/3	2	1/4
7/8	1 4/7	2 1/2	1/3
1	1 4/5	3	3/8
1 1/4	2	3	1/2
1 1/2	2 1/3	3 1/2	1/2
1 3/4	2 3/4	4	5/8
2	3 1/4	5	3/4
2 1/2	3 3/4	5 1/2	7/8

## Bent Anchors

Once the anchor rods are cast into the finished concrete, the transition to steel construction begins. During this time materials are moved, slabs are cleaned and new contractors begin working on the site. All of this movement of material and workers can result in anchor rods being struck by machinery and bent over in place. One example shown in Figure 15 is when clearing snow off a slab, the covered anchors were bent by a bent by a snow blade. Bending anchors can result in untimely delays as the concrete sub-contractor may have to return to the project to perform repair work on the cast-in-place anchors. There are several ways to fix the issue depending on the grade of steel used in anchor design. If the anchor is made of Grade 36 steel, the anchors are allowed to be cold bent back into place as long as the bend is less than 45 degrees (ASTM F1554, 2011). If needed the anchors may be heated according to ASTM F1554 to assist in bending large diameter anchors. However, if high strength steel is used the anchors should be replaced by a post-installed anchor. Not all cast-in-place applications can be replaced by post-installed anchors since post-installed anchors typically have lower capacities. Protecting or marking anchor locations and improving site awareness is the easiest way to prevent this issue from occurring (Fisher, 2012).

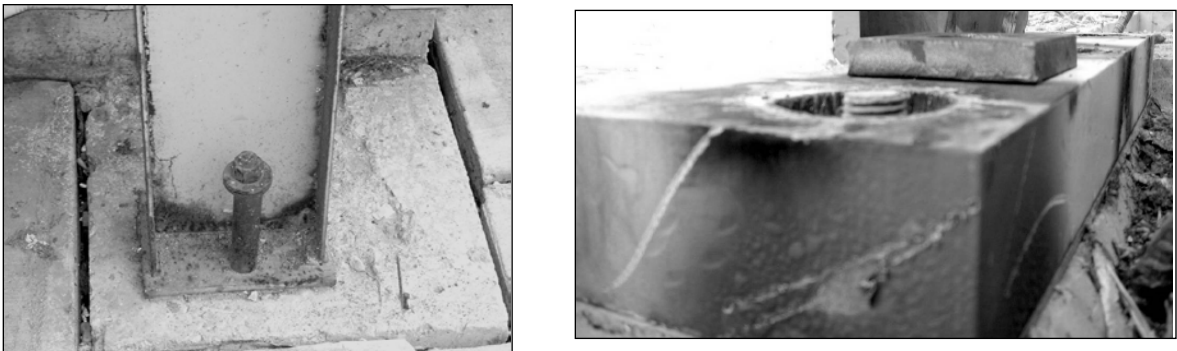


**Figure 15 - Bent Anchor Rods (AISC DG1, 2010)**

## Long or Short Anchors

When anchor rods are installed by using a wood or metal template, the anchor projection out of the template can be fixed by using threaded nuts to firmly attach them in place. However if threaded nuts are not used to support anchors, large variances can be seen in anchor elevation. One example is when small cast-in-place anchors are placed into the top of a foundation wall for a wood sill plate. The anchors are not installed using a template, as they are spaced along the top of the wall. If they are not fastened to the formwork they may settle into the wet concrete.

Anchors may be too long or short once the concrete is poured. If the threaded nut is not fully engaged the designer may choose to use a fraction of the original anchor strength. Other solutions for extremely short anchor rods include welding on a threaded rod, using a coupling nut, or replacing the rods with post-installed anchors. The AISC recommends an extra three inches or more of thread beyond what the detail requires to compensate for some variation in projection (AISC DG1, 2010). However, having an excessive amount of threaded area exposed above the base plate can also be a problem, as it may reflect improper embedment into the concrete member. Examples of both long and short anchor rods can be seen in Figure 16.



**Figure 16 - Long and Short Anchor Rods (Fisher, 2012)**

## Chapter 5 - Cast-in-Place Anchor Design Examples

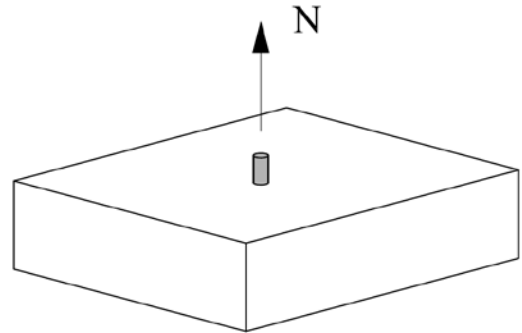
In this chapter several examples are shown to demonstrate the provisions of ACI 318-11 Appendix D and IBC 2012. The examples are completed step-by-step with figures, discussion and corresponding references. The examples are described below:

- 1. Single anchor subjected to tensile loading** – This example demonstrates how to calculate the tensile capacity for an anchor.
- 2. Single anchor subjected to shear loading** – This example demonstrates how to calculate the shear capacity for an anchor.
- 3. Four anchor group base plate subjected to tensile and shear loading** – This column base plate example demonstrates how group effects are considered. The example also shows how a free edge can decrease the anchor group capacity. Tension and shear failure modes are checked individually along with the interaction.
- 4. Four anchor group base plate subjected to tensile and shear loading** – This example demonstrates the effects of a decreased edge distance placed on the Example 3 anchor group.
- 5. Anchor reinforcement design** – This example demonstrates how to design anchor reinforcement for the anchor group used in Example 4.

## Example 1

### Given

- Anchor: (1) 5/8" Diameter Hex Head
- Material: Grade 36
- Embedment,  $h_{ef}$ : 4 inches
- Concrete Compressive Strength,  $f'_c$ : 4,000 psi
- No Supplemental Reinforcement
- Concrete Cracking
- No Seismic Forces



### Find

Determine the ultimate tensile capacity of the cast-in-place anchor.

### Solution

<u>Calculations and Discussion</u>	<u>References</u>
<p><u>Step 1 – Steel Strength in Tension</u></p> $N_{sa} = A_{se,N} f_{uta}$ <p>The effective cross-sectional area of an anchor in tension, <math>A_{se,N}</math>, can be found using the equation given in the commentary of Section RD.5.1.2 as shown below. The area is also tabulated in the AISC Steel Manual Table 7-18.</p> $A_{se,N} = \frac{\pi}{4} \left( d_a - \frac{0.9743}{n_t} \right)^2 = \frac{\pi}{4} \left( 5/8 - \frac{0.9743}{11} \right)^2 = 0.226 \text{ in}^2$ $f_{uta} = 58 \text{ ksi}$ $N_{sa} = (0.226 \text{ in}^2)(58 \text{ ksi}) = 13.11 \text{ kips}$ <p>Grade 36 steel is considered ductile steel. The strength reduction factor for ductile steel failure is:</p> $\Phi = 0.75$ $\Phi N_{sa} = 0.75(13.11 \text{ kips}) = 9.83 \text{ kips}$	<p>D.5.1</p> <p>Eq. D-2</p>  <p>AISC Table 7-18</p> <p>ASTM F1554</p>  <p>D4.3</p>

<u>Calculations and Discussion</u>	<u>References</u>
<u>Step 2 – Concrete Breakout Strength in Tension</u>	D.5.2
$N_{cb} = \frac{A_{NC}}{A_{Nco}} \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_b$	Eq. D-3
For this example there are no free edges around the single anchor. Thus, $A_{NC}$ and $A_{Nco}$ are equal.	
$A_{NC} = A_{Nco} = 9h_{ef}^2 = 9(4)^2 = 144 \text{ in}^2$	Eq. D-5
$\Psi_{ed,N} = 1.0 \text{ since } c_{a,min} \geq 1.5h_{ef} \text{ with no free edge}$	D.5.2.5
$\Psi_{c,N} = 1.0 \text{ when concrete cracking is expected}$	D.5.2.6
$\Psi_{cp,N} = 1.0 \text{ for cast-in-place anchors}$	D.5.2.7
$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5} \text{ for a single anchor with } h_{ef} < 11 \text{ in}$	Eq. D-6
$k_c = 24 \text{ for cast-in-place anchors}$	D5.2.2
$\lambda_a = 1.0 \text{ for normal weight concrete}$	
$f'_c = 4000 \text{ psi}$	
$h_{ef} = 4 \text{ inches}$	
$N_b = (24)(1.0)\sqrt{4000 \text{ psi}}(4 \text{ inches})^{1.5} = 12,143 \text{ lbs}$	
$N_{cb} = \frac{144 \text{ in}^2}{144 \text{ in}^2} (1.0)(1.0)(1.0)(12,143 \text{ lbs}) = 12,143 \text{ lbs}$	Eq. D-3
$\Phi = 0.70 \text{ for concrete breakout under Condition B}$	D.4.3
$\Phi N_{cb} = 0.70(12,143 \text{ lbs}) = 8500 \text{ lbs} = 8.50 \text{ kips}$	
<u>Step 3 – Pullout Strength in Tension</u>	D.5.3
$N_{pn} = \Psi_{c,P} N_p$	Eq. D-13
$\Psi_{c,P} = 1.0 \text{ when concrete cracking is expected}$	D.5.3.6
$N_p = 8A_{brg} f'_c \text{ for a headed bolt}$	Eq. D-14
The bearing area of heads and nuts, $A_{brg}$ , is not found in Appendix D. However, the values are tabulated in the PCA Notes and reproduced in Appendix A.	PCA Notes Table 34-2
$A_{brg} = 0.454 \text{ in}^2 \text{ for a } 5/8'' \text{ diameter hex head}$	
$f'_c = 4,000 \text{ psi}$	

Calculations and Discussion	References
<p> <math display="block">N_p = 8(0.454 \text{ in}^2)(4,000 \text{ psi}) = 14,528 \text{ lbs}</math> <math display="block">N_{pn} = (1.0)(14,528 \text{ lbs}) = 14,528 \text{ lbs}</math> <math display="block">\Phi = 0.70 \text{ for pullout under Condition B}</math> <math display="block">\Phi N_{pn} = 0.70(14,528 \text{ lbs}) = 10,170 \text{ lbs} = 10.17 \text{ kips}</math> </p>	<p>D.4.3</p>
<p> <u>Step 4 – Concrete Side-Face Blowout in Tension</u>  Concrete side-face blowout only applies when:  <math display="block">h_{ef} &gt; 2.5c_{a1}</math> However, with no free edge near the single anchor, <math>c_{a1} = \infty</math>  <math display="block">4 \text{ inches} &lt; \infty</math> Thus, concrete side-face blowout does not apply. </p>	<p>D.5.4</p>
<p> <u>Tensile Summary and Governing Case</u>  Steel Strength = 9.83 kips  Concrete Breakout Strength = <b>8.50 kips ←</b>  Pullout Strength = 10.17 kips  Concrete Side-Face Blowout Strength = N/A </p>	
<p> <u>Step 5 – Splitting Failure</u>  The single anchor is not close to a free edge or any other anchor. Thus, the requirements to preclude splitting failure are met.   This example demonstrated how to find the tensile capacity of a single anchor. Concrete breakout strength governs the design because the anchor is not near any free edge and has a shallow 4” embedment. </p>	<p>D.8</p>



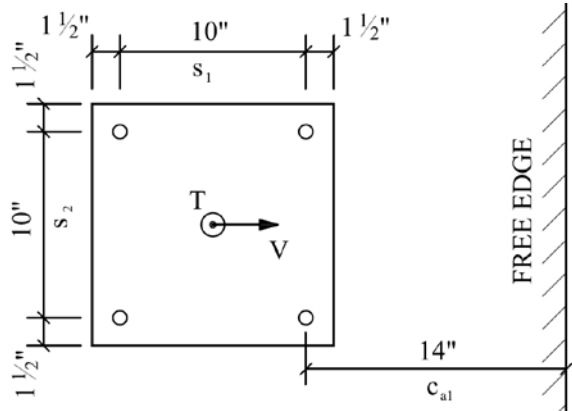




### Example 3

#### Given

- Anchor: (4) 3/4" Diameter Hex Head
- Material: Grade 36
- Embedment,  $h_{ef}$ : 12 inches
- Concrete Support:  $f'_c$ : 4,000 psi,  $\infty$  thickness
- No Supplemental Reinforcement
- Concrete Cracking
- No Seismic Forces
- Strength Level Forces: Tension = 40 kips,  
Shear = 10 kips



#### Find

Determine the ultimate capacity of the cast-in-place anchor. Check tension, shear, and interaction forces.

#### Solution

<u>Calculations and Discussion</u>	<u>References</u>
<p>Since the anchor group is concentrically loaded, each anchor is subjected to the same tension and shear forces.</p> <p><u>Step 1 – Steel Strength in Tension</u></p> $N_{sa} = A_{se,N} f_{uta}$ <p>The effective cross-sectional area of an anchor in tension, <math>A_{se,N}</math>, can be found using the equation given in the commentary of Section RD.5.1.2 as show below. The area is also tabulated in the AISC Steel Manual Table 7-18.</p> $A_{se,N} = \frac{\pi}{4} \left( d_a - \frac{0.9743}{n_t} \right)^2 = \frac{\pi}{4} \left( \frac{3}{4} - \frac{0.9743}{10} \right)^2 = 0.334 \text{ in}^2$ $f_{uta} = 58 \text{ ksi}$ $N_{sa} = (0.334 \text{ in}^2)(58 \text{ ksi}) = 19.37 \text{ kips}$ <p><math>\Phi = 0.75</math> for ductile steel</p> $\Phi N_{sa} = 0.75(19.37 \text{ kips}) = 14.53 \text{ kips for a single anchor}$ $= (4)(14.53 \text{ kips}) = 58.12 \text{ kips for the anchor group}$	<p>D.5.1</p> <p>Eq. D-2</p> <p>AISC Table 7-18</p> <p>ASTM F1554</p> <p>D4.3</p>

<u>Calculations and Discussion</u>	<u>References</u>
<u>Step 2 – Concrete Breakout Strength in Tension</u>	D.5.2
$N_{cbg} = \frac{A_{Nc}}{A_{Nco}} \Psi_{ec,N} \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_b$	Eq. D-4
<p>For this example <math>A_{Nc}</math> and <math>A_{Nco}</math> will not be equal, as the multi anchor group will have a larger breakout area than a single anchor. There is also an edge distance of 14 inches is less than <math>1.5h_{ef}</math>, meaning the full failure cone cannot be developed.</p>	
$A_{Nc} = (c_{a1} + s_1 + 1.5h_{ef})(1.5h_{ef} + s_2 + 1.5h_{ef})$ $= (14 \text{ in} + 10 \text{ in} + 1.5(12 \text{ in}))(1.5(12 \text{ in}) + 10 \text{ in} + 1.5(12 \text{ in}))$ $= 1,932 \text{ in}^2$	Fig. RD.5.2.1
$A_{Nco} = 9h_{ef}^2 = 9(12 \text{ in})^2 = 1,296 \text{ in}^2$	Eq. D-5
<p><math>\Psi_{ec,N} = 1.0</math> when the tensile force is applied at the centroid of the anchor group.</p>	D.5.2.4
<p>Since, <math>c_{a1} \leq 1.5h_{ef}</math> edge effects have to be considered</p>	
$\Psi_{ed,N} = 0.7 + 0.3 \left( \frac{c_{a \text{ min}}}{1.5h_{ef}} \right) = 0.7 + 0.3 \left( \frac{14 \text{ in}}{1.5(12 \text{ in})} \right) = 0.933$	D.5.2.5
<p><math>\Psi_{c,N} = 1.0</math> when concrete cracking is expected</p>	D.5.2.6
<p><math>\Psi_{cp,N} = 1.0</math> for cast-in-place anchors</p>	D.5.2.7
$N_b = 16\lambda_a \sqrt{f'_c} h_{ef}^{5/3} \text{ for } 11 \text{ in} \leq h_{ef} \leq 25 \text{ in}$ $\lambda_a = 1.0 \text{ for normal weight concrete}$ $f'_c = 4000 \text{ psi}$ $h_{ef} = 4 \text{ inches}$	Eq. D-7
$N_b = (16)(1.0)\sqrt{4000 \text{ psi}}(4 \text{ in})^{5/3} = 63,648 \text{ lbs}$	
$N_{cbg} = \frac{1,932 \text{ in}^2}{1,296 \text{ in}^2} (1.0)(0.933)(1.0)(1.0)(63,648 \text{ lbs}) = 88,557 \text{ lbs}$	
<p><math>\Phi = 0.70</math> for concrete breakout under Condition B</p>	D.4.3
$\Phi N_{cb} = 0.70(88,557 \text{ lbs}) = 61,990 \text{ lbs} = 61.99 \text{ kips for the group}$	

<u>Calculations and Discussion</u>	<u>References</u>								
<p><u>Step 3 – Pullout Strength in Tension</u></p> $N_{pn} = \Psi_{c,P} N_p$ <p><math>\Psi_{c,P} = 1.0</math> when concrete cracking is expected</p> $N_p = 8A_{brg} f'_c \text{ for a headed bolt}$ <p>The bearing area of heads and nuts,, is not found in Appendix D. However the values are tabulated in the PCA Notes and reproduced in Appendix A.</p> $A_{brg} = 0.654 \text{ in}^2$ $f'_c = 4,000 \text{ psi}$ $N_p = 8(0.654 \text{ in}^2)(4,000 \text{ psi}) = 20,928 \text{ lbs}$ $N_{pn} = 1.0(20,928 \text{ lbs}) = 20,928 \text{ lbs}$ <p><math>\Phi = 0.70</math> for pullout under Condition B</p> $\Phi N_{pn} = 0.70(20,928 \text{ lbs}) = 14,650 \text{ lbs} = 14.65 \text{ kips for a single anchor}$ $= (4)(14.65 \text{ kips}) = 58.60 \text{ kips for the anchor group}$	<p>D.5.3</p> <p>Eq. D-13</p> <p>D.5.3.6</p> <p>Eq. D-14</p> <p>PCA Notes Table 34-2</p>								
<p><u>Step 4 – Concrete Side-Face Blowout in Tension</u></p> <p>Concrete side-face blowout only applies when:</p> $h_{ef} > 2.5c_{a1}$ <p>However,</p> $12 \text{ in} < 2.5(14 \text{ in})$ $12 \text{ in} < 35 \text{ in}$ <p>Concrete Side-Face Blowout does not apply for this example.</p>	<p>D.4.3</p>								
<p><u>Tensile Summary and Governing Case</u></p> <table> <tr> <td>Steel Strength =</td> <td><b>58.12 kips ←</b></td> </tr> <tr> <td>Concrete Breakout Strength =</td> <td>62.00 kips</td> </tr> <tr> <td>Pullout Strength =</td> <td>58.60 kips</td> </tr> <tr> <td>Concrete Side-Face Blowout Strength =</td> <td>N/A</td> </tr> </table>	Steel Strength =	<b>58.12 kips ←</b>	Concrete Breakout Strength =	62.00 kips	Pullout Strength =	58.60 kips	Concrete Side-Face Blowout Strength =	N/A	<p>D.5.4</p>
Steel Strength =	<b>58.12 kips ←</b>								
Concrete Breakout Strength =	62.00 kips								
Pullout Strength =	58.60 kips								
Concrete Side-Face Blowout Strength =	N/A								

<u>Calculations and Discussion</u>	<u>References</u>
<p><u>Step 5 – Steel Strength in Shear</u></p> $V_{sa} = 0.6A_{se,V}f_{uta}$ <p>The effective cross-sectional area of an anchor in shear, <math>A_{se,V}</math>, can be found using the equation given in the commentary of Section RD.6.1.2 as show below. The area is also tabulated in the AISC Steel Manual Table 7-18.</p> $A_{se,V} = \frac{\pi}{4} \left( d_a - \frac{0.9743}{n_t} \right)^2 = \frac{\pi}{4} \left( 3/4 - \frac{0.9743}{11} \right)^2 = 0.334 \text{ in}^2$ $f_{uta} = 58 \text{ ksi}$ $V_{sa} = 0.6(0.334 \text{ in}^2)(58 \text{ ksi}) = 11.62 \text{ kips}$ <p>Grade 36 steel is considered ductile steel. The strength reduction factor for ductile steel failure is:</p> $\Phi = 0.65$ $\Phi V_{sa} = 0.65(11.62 \text{ kips}) = 7.55 \text{ kips for a single anchor}$ $= (4)(7.55 \text{ kips}) = 30.21 \text{ kips for the anchor group}$	<p>D.5.1</p> <p>Eq. D-29</p> <p>AISC Table 7-18</p>
<p><u>Step 6 – Concrete Breakout Strength in Shear</u></p> $V_{cbg} = \frac{A_{Vc}}{A_{Vco}} \Psi_{ec,V} \Psi_{ed,V} \Psi_{c,V} \Psi_{h,V} V_b$ <p>The equation for <math>A_{Vc}</math> changes for each specific case, as can be seen in the commentary for Section RD.6.2. For this example there are two anchors located along the free edge with no depth limitation.</p> $A_{Vc} = (\text{width})(\text{depth}) = (2(1.5c_{a1}) + s_1)(1.5c_{a1})$ $= (2(1.5(14 \text{ in}) + 10 \text{ in}))(1.5(14 \text{ in}))$ $= 1,092 \text{ in}^2$ $A_{Vco} = 4.5c_{a1}^2 = 4.5(14 \text{ in})^2 = 882 \text{ in}^2$ <p><math>\Psi_{ec,V} = 1.0</math> when the shear force is applied at the centroid of the anchor group.</p> <p><math>\Psi_{ed,V} = 1.0</math> for only a single free edge</p> <p><math>\Psi_{c,V} = 1.0</math> when concrete cracking is expected</p> <p><math>\Psi_{h,V} = 1.0</math> when <math>h_a &gt; 1.5c_{a1}</math> meaning the bottom surface is below the failure cone.</p>	<p>D.6.2</p> <p>Eq. D-31</p> <p>Fig. RD.6.2.1(b)</p> <p>Fig. RD.6.2.1(a)</p> <p>D.6.2.5</p> <p>D.6.2.6</p> <p>D.6.2.7</p> <p>D.6.2.8</p>

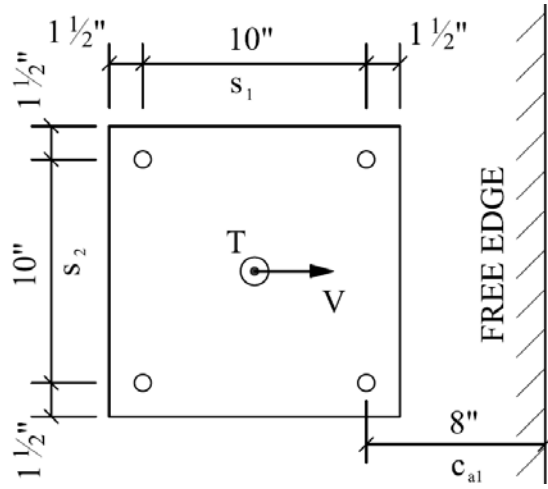
<u>Calculations and Discussion</u>	<u>References</u>
$V_b = \left(7 \left(\frac{l_e}{d_a}\right)^2 \sqrt{d_a}\right) \lambda_a \sqrt{f'_c} c_{a1}^{1.5}$	Eq. D-33
$l_e = \text{smaller of: } h_{ef} = 12 \text{ in} \quad \text{or} \quad 8d_a = 8 \left(\frac{3}{4} \text{ in}\right) = 6 \text{ in}$	D.6.2.2
$= 6 \text{ in}$	
$d_a = \frac{3}{4} \text{ in}$	
$\lambda_a = 1.0 \text{ for normal weight concrete}$	
$f'_c = 4000 \text{ psi}$	
$c_{a1} = 14 \text{ in}$	
$V_b = \left(7 \left(\frac{6 \text{ in}}{\frac{3}{4} \text{ in}}\right)^{0.2} \sqrt{\frac{3}{4} \text{ in}}\right) (1.0) \sqrt{4,000 \text{ psi}} (14 \text{ in})^{1.5} = 30,442 \text{ lbs}$	
$V_{cbg} = \frac{1,092 \text{ in}^2}{882 \text{ in}^2} (1.0)(1.0)(1.0)(1.0)(30,442 \text{ lbs}) = 37,690 \text{ lbs}$	
$\Phi = 0.70 \text{ for concrete breakout under Condition B}$	D.4.3
$\Phi V_{cbg} = 0.70(37,690 \text{ lbs}) = 26,380 \text{ lbs} = 26.38 \text{ kips for the group}$	
<p><u>Step 3 – Concrete Pryout Strength in Shear</u></p>	D.6.3
$V_{cp} = k_{cp} N_{cp} \text{ for a group of anchors}$	Eq. D-41
$k_{cp} = 2.0 \text{ for } h_{ef} \geq 2.5 \text{ in}$	
$N_{cp} = N_{cb} \text{ from tension calculations} = 88,557 \text{ lbs}$	
$V_{cp} = (2.0)(88,557 \text{ lbs}) = 177,114 \text{ lbs}$	
$\Phi = 0.70 \text{ for concrete pryout under Condition B}$	D.4.3
$\Phi V_{cp} = (0.70)(177,114 \text{ lbs}) = 123,980 \text{ lbs}$	
$= 123.98 \text{ kips for the anchor group}$	
<p><u>Shear Summary and Governing Case</u></p>	
<p>Steel Strength = 30.21 kips</p>	
<p>Concrete Breakout Strength = <b>26.38 kips ←</b></p>	
<p>Pullout Strength = 123.98 kips</p>	



## Example 4

### Given

- Anchor: (4) 3/4" Diameter Hex Head
- Material: Grade 36
- Embedment,  $h_{ef}$ : 12 inches
- Concrete Support:  $f'_c$ : 4,000 psi,  $\infty$  thickness
- No Supplemental Reinforcement
- Concrete Cracking
- No Seismic Forces
- Strength Level Forces: Tension = 40 kips,  
Shear = 10 kips



### Find

Find the ultimate capacity of the cast-in-place anchor. Check tension, shear, and interaction forces.

### Solution

<u>Calculations and Discussion</u>	<u>References</u>
<p>Example 4 has the exact parameters as Example 3 except for the edge distance <math>c_{al}</math>. As a result, only Step 2 and...are impacted. Shown below</p> <p><u>Step 1 – Steel Strength in Tension</u></p> <p><math>\Phi N_{sa} = 58.12</math> kips for the anchor group. No change from Example 3.</p>	D.5.1
<p><u>Step 2 – Concrete Breakout Strength in Tension</u></p> $N_{cbg} = \frac{A_{Nc}}{A_{Nco}} \Psi_{ec,N} \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_b$ <p>There is an edge distance of 8 inches which is less than <math>1.5h_{ef}</math>, meaning the full failure cone cannot be developed.</p> $A_{Nc} = (c_{al} + s_1 + 1.5h_{ef})(1.5h_{ef} + s_2 + 1.5h_{ef})$ $= (8 \text{ in} + 10 \text{ in} + 1.5(12 \text{ in}))(1.5(12 \text{ in}) + 10 \text{ in} + 1.5(12 \text{ in}))$ $= 1,656 \text{ in}^2$ $A_{Nco} = 9h_{ef}^2 = 9(12 \text{ in})^2 = 1,296 \text{ in}^2$	D.5.2 Eq. D-4  Fig. RD.5.2.1  Eq. D-5



<u>Calculations and Discussion</u>	<u>References</u>
<p><math>\Psi_{ec,N} = 1.0</math> when the tensile force is applied at the centroid of the anchor group.</p>	D.5.2.4
<p>Since, <math>c_{a1} \leq 1.5h_{ef}</math> edge effects have to be considered</p>	D.5.2.5
$\Psi_{ed,N} = 0.7 + 0.3 \left( \frac{c_{a \min}}{1.5h_{ef}} \right) = 0.7 + 0.3 \left( \frac{8 \text{ in}}{1.5(12 \text{ in})} \right) = 0.833$	
<p><math>\Psi_{c,N} = 1.0</math> when concrete cracking is expected</p>	D.5.2.6
<p><math>\Psi_{cp,N} = 1.0</math> for cast-in-place anchors</p>	D.5.2.7
<p><math>N_b = 16\lambda_a \sqrt{f'_c} h_{ef}^{5/3}</math> for <math>11 \text{ in} \leq h_{ef} \leq 25 \text{ in}</math></p> <p style="margin-left: 40px;"><math>\lambda_a = 1.0</math> for normal weight concrete</p> <p style="margin-left: 40px;"><math>f'_c = 4000 \text{ psi}</math></p> <p style="margin-left: 40px;"><math>h_{ef} = 4 \text{ inches}</math></p>	Eq. D-7
$N_b = (16)(1.0)\sqrt{4000 \text{ psi}}(4 \text{ in})^{5/3} = 63,648 \text{ lbs}$	
$N_{cbg} = \frac{1,656 \text{ in}^2}{1,296 \text{ in}^2} (1.0)(0.833)(1.0)(1.0)(63,648 \text{ lbs}) = 67,773 \text{ lbs}$	
<p><math>\Phi = 0.70</math> for concrete breakout under Condition B</p>	D.4.3
$\Phi N_{cb} = 0.70(67,773 \text{ lbs}) = 47,441 \text{ lbs}$ <p style="margin-left: 40px;">= 47.44 kips for the anchor group</p>	
<p><u>Step 3 – Pullout Strength in Tension</u></p>	D.5.3
<p><math>\Phi N_{pn} = 58.60 \text{ kips}</math> for the anchor group. No change from Example 3.</p>	
<p><u>Step 4 – Concrete Side-Face Blowout in Tension</u></p>	D.5.4
<p>Concrete side-face blowout only applies when:</p> <p><math>h_{ef} &gt; 2.5c_{a1}</math></p> <p>However,</p> <p><math>12 \text{ in} &lt; 2.5(8 \text{ in})</math></p> <p><math>12 \text{ in} &lt; 20 \text{ in}</math></p> <p>Concrete Side-Face Blowout does not apply for this example.</p>	

<u>Calculations and Discussion</u>	<u>References</u>
<u>Tensile Summary and Governing Case</u>	
Steel Strength =	58.12 kips
Concrete Breakout Strength =	<b>47.44 kips ←</b>
Pullout Strength =	58.60 kips
Concrete Side-Face Blowout Strength =	N/A
<u>Step 5 – Steel Strength in Shear</u>	D.5.1
$\Phi V_{sa} = 30.21$ kips for the anchor group. No change from Example 3.	
<u>Step 6 – Concrete Breakout Strength in Shear</u>	D.6.2
$V_{cbg} = \frac{A_{Vc}}{A_{Vco}} \Psi_{ec,V} \Psi_{ed,V} \Psi_{c,V} \Psi_{h,V} V_b$	Eq. D-31
The equation for $A_{Vc}$ changes for each specific case, as can be seen in the commentary for Section RD.6.2. For this example there are two anchors located along the free edge with no depth limitation.	
$A_{Vc} = (width)(depth) = (2(1.5c_{a1}) + s_1)(1.5c_{a1})$	Fig.
$= (2(1.5(8in) + 10 in))(1.5(8 in))$	RD.6.2.1(b)
$= 408 in^2$	
$A_{Vco} = 4.5c_{a1}^2 = 4.5(8 in)^2 = 288 in^2$	Fig.
$\Psi_{ec,V} = 1.0$ when the shear force is applied at the centroid of the anchor group.	RD.6.2.1(a)
$\Psi_{ed,V} = 1.0$ for only a single free edge	D.6.2.5
$\Psi_{c,V} = 1.0$ when concrete cracking is expected	D.6.2.6
$\Psi_{h,V} = 1.0$ when $h_a > 1.5c_{a1}$ meaning the bottom surface is below the failure cone.	D.6.2.7
	D.6.2.8

<u>Calculations and Discussion</u>	<u>References</u>
$V_b = \left(7 \left(\frac{l_e}{d_a}\right)^2 \sqrt{d_a}\right) \lambda_a \sqrt{f'_c} c_{a1}^{1.5}$	Eq. D-33
$l_e = \text{smaller of: } h_{ef} = 12 \text{ in} \quad \text{or} \quad 8d_a = 8 \left(\frac{3}{4} \text{ in}\right) = 6 \text{ in}$	D.6.2.2
$= 6 \text{ in}$	
$d_a = \frac{3}{4} \text{ in}$	
$\lambda_a = 1.0 \text{ for normal weight concrete}$	
$f'_c = 4000 \text{ psi}$	
$c_{a1} = 8 \text{ in}$	
$V_b = \left(7 \left(\frac{6 \text{ in}}{\frac{3}{4} \text{ in}}\right)^{0.2} \sqrt{\frac{3}{4} \text{ in}}\right) (1.0) \sqrt{4,000 \text{ psi}} (8 \text{ in})^{1.5} = 13,150 \text{ lbs}$	
$V_{cbg} = \frac{408 \text{ in}^2}{288 \text{ in}^2} (1.0)(1.0)(1.0)(1.0)(13,150 \text{ lbs}) = 18,629 \text{ lbs}$	
$\Phi = 0.70 \text{ for concrete breakout under Condition B}$	D.4.3
$\Phi V_{cbb} = 0.70(18,629 \text{ lbs}) = 13,040 \text{ lbs}$	
$= 13.04 \text{ kips for the anchor group}$	
<p><u>Step 3 – Concrete Pryout Strength in Shear</u></p>	D.6.3
$V_{cpg} = k_{cp} N_{cpg} \text{ for a single anchor}$	Eq. D-41
$k_{cp} = 2.0 \text{ for } h_{ef} \geq 2.5 \text{ in}$	
$N_{cpg} = N_{cbg} \text{ from tension calculations} = 67,773 \text{ lbs}$	
$V_{cp} = (2.0)(67,773 \text{ lbs}) = 135,546 \text{ lbs}$	
$\Phi = 0.70 \text{ for concrete pryout under Condition B}$	D.4.3
$\Phi V_{cp} = (0.70)(135,546 \text{ lbs}) = 94,882 \text{ lbs}$	
$= 94.88 \text{ kips for the anchor group}$	
<p><u>Shear Summary and Governing Case</u></p>	
Steel Strength =	30.21 kips
Concrete Breakout Strength =	<b>13.04 kips</b> ←
Pryout Strength =	94.88 kips



## Example 5

**Given**

Anchor: (4) 3/4" Diameter Hex Head

Material: Grade 36

Embedment,  $h_{ef}$ : 12 inches

Concrete Support:  $f'_c$ : 4,000 psi,  
∞ thickness

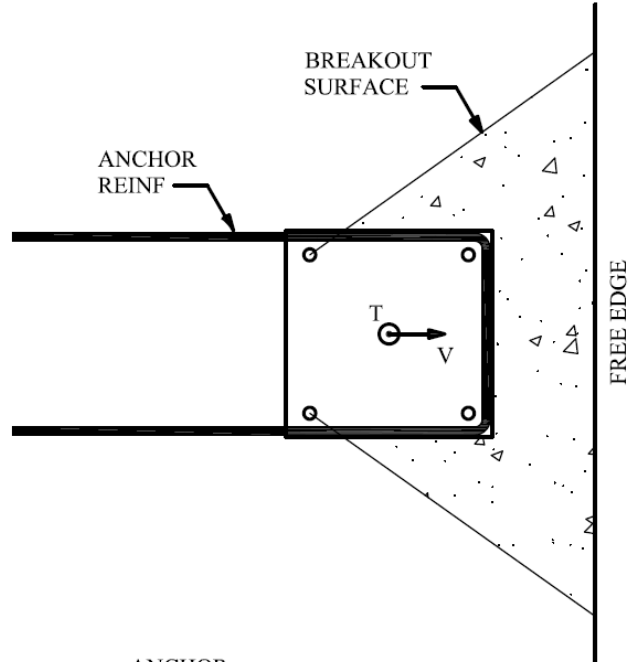
Concrete Cracking

No Seismic Forces

Strength Level Forces:

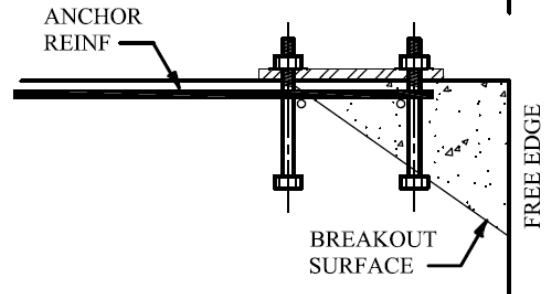
Tension = 40 kips,

Shear = 10 kips



**Find**

Find the shear anchor reinforcement needed to make Example 4 adequate.



**Solution**

<u>Calculations and Discussion</u>	<u>References</u>
<p>Example 4 failed under the interaction of tension and shear forces. By adding shear anchor reinforcement around the anchor group, the governing concrete breakout strength can be increased to the anchor reinforcement strength.</p>	
<p><u>Step 1 – Steel Strength in Tension</u></p> <p><math>\Phi N_{sa} = 58.12</math> kips for the anchor group. No change from Example 4.</p>	D.5.1
<p><u>Step 2 – Concrete Breakout Strength in Tension</u></p> <p><math>\Phi N_{cb} = 47.44</math> kips for the anchor group. No change from Example 4.</p>	D.5.2

<u>Calculations and Discussion</u>	<u>References</u>
<p><u>Step 3 – Pullout Strength in Tension</u></p> <p><math>\Phi N_{pn} = 58.60</math> kips for the anchor group. No change from Example 4.</p>	D.5.3
<p><u>Step 4 – Concrete Side-Face Blowout in Tension</u></p> <p>Concrete Side-Face Blowout does not apply for this example, similar to Example 4.</p>	D.5.4
<p><u>Tensile Summary and Governing Case</u></p> <p>Steel Strength = 58.12 kips  Concrete Breakout Strength = <b>47.44 kips ←</b>  Pullout Strength = 58.60 kips  Concrete Side-Face Blowout Strength = N/A</p>	
<p><u>Determine <math>\Phi V_n</math> Required</u></p> $\frac{N_{ua}}{\Phi N_n} + \frac{V_{ua}}{\Phi V_n} \leq 1.2$ $\frac{40 \text{ kips}}{47.44 \text{ kips}} + \frac{10 \text{ kips}}{\Phi V_n} \leq 1.2$ <p><math>\Phi V_n = 28.02 \text{ kips} \approx 28 \text{ kips}</math></p> <p>The governing shear failure mode must be greater than 28 kips to make the interaction equation less than 1.2.</p>	Eq. D-42
<p><u>Step 5 – Steel Strength in Shear</u></p> <p><math>\Phi V_{sa} = 30.21</math> kips for the anchor group. No change from Example 4.</p>	D.6.1
<p><u>Step 6 – Concrete Breakout Strength in Shear</u></p> <p>If using shear anchor reinforcement this failure mode does not apply per Section D.6.2.9. Instead, the design strength of the reinforcement is permitted to be used by determining <math>\Phi V_n</math>. The lost <math>\Phi V_n</math> allowed is 28 kips.</p>	D.6.2 D.6.2.9

<u>Calculations and Discussion</u>	<u>References</u>						
$V_{ua} \leq \Phi V_n$ $V_{ua} = \Phi A_s f_y$ <p>28 kips = 0.75(<math>A_s</math>)(60 ksi) solving for <math>A_s</math></p> $A_s = 0.63 \text{ in}^2$ <p>Section D.6.2.9 Commentary recommends stirrups or hairpins and a maximum bar size of #5. Use #4 for this example.</p> <p>Use (2)#4 stirrups = (2 stirrups)(2 legs)(0.2 in<sup>2</sup>) = 0.8 in<sup>2</sup></p> $\Phi V_n = (0.75)(0.8 \text{ in}^2)(60 \text{ ksi}) = 36.00 \text{ kips for the stirrups}$	D.6.2.9						
<p><u>Step 3 – Concrete Pryout Strength in Shear</u></p> $\Phi V_{cp} = 94.88 \text{ kips for the anchor group. No change from Example 4.}$	D.6.3						
<p><u>Shear Summary and Governing Case</u></p> <table style="width: 100%; border: none;"> <tr> <td style="width: 60%;">Steel Strength =</td> <td style="text-align: right;"><b>30.21 kips ←</b></td> </tr> <tr> <td>Concrete Breakout Strength =</td> <td style="text-align: right;">36.00 kips</td> </tr> <tr> <td>Pullout Strength =</td> <td style="text-align: right;">94.88 kips</td> </tr> </table>	Steel Strength =	<b>30.21 kips ←</b>	Concrete Breakout Strength =	36.00 kips	Pullout Strength =	94.88 kips	
Steel Strength =	<b>30.21 kips ←</b>						
Concrete Breakout Strength =	36.00 kips						
Pullout Strength =	94.88 kips						
<p><u>Check Interaction Equation</u></p> $\frac{N_{ua}}{\Phi N_n} + \frac{V_{ua}}{\Phi V_n} \leq 1.2$ $\frac{40 \text{ kips}}{47.44 \text{ kips}} + \frac{10 \text{ kips}}{30.21 \text{ kips}} \leq 1.2$ $0.843 + 0.332 \leq 1.2$ $1.18 \leq 1.2 \text{ ok}$	D.7 Eq. D-42						
<p>By adding (2) #4 stirrups around the anchor group, the concrete breakout strength was replaced by the reinforcement strength. The steel strength is now governing case. This resulted in the anchor group being 98% stressed compared to the 135% stressed from Example 4. The designer is encouraged to add anchor reinforcement to the design, instead of redesigning the entire anchor group.</p>							

## Chapter 6 - Conclusions

Concrete anchors mark an important transition between the design and construction of steel and concrete. Anchors are used in many applications to transmit a variety of loads between the structural members. Concrete anchor design and construction is an important part of a projects success. The reader should now be familiar with anchor types, materials, applications, design provisions, and construction issues.

The reader is encouraged to further review the Appendix D design provisions. Since Appendix D is an all-encompassing document for every anchor situation there are many exceptions and limitations. Mechanical post-installed anchors have similar failure modes to cast-in-place anchors, while adhesive anchors introduce epoxy bond as a failure mode. These post-installed anchor capacities are highly dependent on the manufacture's specifications and testing.

It is also suggested the designer become familiar with available computer software to design concrete anchors. If a project requires several different anchor layouts with varying loads, designing anchors by hand using Appendix D becomes highly inefficient. There are several design software programs available such as RISABase, Hilti PROFIS Anchor and Simpson's Anchor Designer. It is important to be familiar with Appendix D and its provisions before using any of these design programs.

Concrete anchors are constantly evolving in the structural engineering industry. The Appendix D design provisions will continue to be improved and updated with each code cycle along with the modifications in the International Building Code. Manufactures will develop new anchor types and materials to fit engineer and contractor needs. Concrete anchors will continue to be a major point of discussion for both the structural engineer and contractor.



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## Appendix A - Reference Tables

**Table 6 - Anchor Dimensional Properties (PCA, 2008)**

Anchor Diameter ( $d_a$ ) (in)	Gross Area of Anchor ( $\text{in}^2$ )	Effective Area of Anchor ( $A_{se,N}, A_{se,V}$ ) ( $\text{in}^2$ )	Bearing Area of Heads and Nuts ( $A_{brg}$ ) ( $\text{in}^2$ )			
			Square	Heavy Square	Hex	Heavy Hex
1/4	0.49	0.032	0.142	0.201	0.117	0.167
3/8	0.11	0.078	0.280	0.362	0.164	0.299
1/2	0.196	0.142	0.464	0.569	0.291	0.467
5/8	0.307	0.226	0.693	0.822	0.454	0.671
3/4	0.442	0.334	0.824	1.121	0.654	0.911
7/8	0.601	0.462	1.121	1.465	0.891	1.188
1	0.785	0.606	1.465	1.855	1.163	1.501
1 1/8	0.994	0.763	1.854	2.291	1.472	1.851
1 1/4	1.227	0.969	2.228	2.773	1.817	2.237
1 3/8	1.485	1.16	2.769	3.300	2.199	2.659
1 1/2	1.767	1.41	3.295	3.873	2.617	3.118
1 3/4	2.405	1.90	--	--	--	4.144
2	3.142	2.50	--	--	--	5.316

**Table 7 - Anchor Material Properties (PCA, 2008)**

Material Specification	Grade or Type	Diameter (in)	Tensile Strength for Design $f_{ut}$ (ksi)	Tensile Strength Minimum (ksi)	Yield Strength Minimum		Elongation Minimum		Reduction of Area Minimum (%)
					ksi	Method	%	Length	
AWS D1.1	B	1/2 to 1	60	60	50	0.20%	20	2"	50
ASTM A307	A	$\leq 4$	60	60	--	--	18	2"	--
	C	$\leq 4$	58	58-80	36	--	23	2"	--
ASTM A354	BC	$\leq 4$	125	125	109	0.20%	16	2"	50
	BD	$\leq 4$	125	150	130	0.20%	14	2"	40
ASTM A449	1	$\leq 1$	120	120	92	0.20%	14	4D	35
		1 to 1-1/2	105	105	81	0.20%	14	4D	35
		$> 1-1/2$	90	90	58	0.20%	14	4D	35
ASTM F1554	36	$\leq 2$	58	58-80	36	0.20%	23	2"	40
	55	$\leq 2$	75	75-95	55	0.20%	21	2"	30
	105	$\leq 2$	125	125-150	105	0.20%	15	2"	45

**Table 8 - Anchor Threads Per Inch (AISC, 2010)**

Bolt Diameter	Threads Per Inch, $n_t$
0.250	20
0.375	16
0.500	13
0.625	11
0.750	10
0.875	9
1.000	8
1.125	7
1.250	7
1.375	6
1.500	6
1.750	5
2.000	4.5