Design of Anchorage in Reinforced Concrete Members

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ABSTRACT

This project is written in an informative manner and discusses design process, and requirements with examples for anchors and anchor groups in concrete which are used to transmit forces to concrete elements by tension, shear, or a combination of the two.

Various different anchor types, such as cast-in place anchors, post-installed anchors, adhesive anchors, expansion anchors, hooked bolts and undercut anchors are analyzed in this report. The critical effects of factored loads which the anchors and anchor groups are designed for, are determined by elastic analysis. Seismic effects which include load combinations with earthquake effects are out of this project's scope.

Discussed in this project, the background of theory, analytical analysis, and design equations provided in this project for determining the performance of a specific anchor have been calibrated from an extensive database of experimental tests which have been conducted over the last 30 years. The equations are designed with a 5% fractile failure level from these tests, meaning that 95 times out of 100, the actual strength of the anchor will exceed the nominal strength which is key in providing a more consistent level of safety for a wider range of failure modes in both cracked and uncracked concrete.

In addition, this report also provides the design process of the same anchor types aforementioned, with design equations, in The American Concrete Institute codes. Key differences and similarities are paid particular attention to and discussed in depth compared to Canadian Standards in this report.

1. Introduction

In reinforced concrete design anchor bolts are used to connect both structural and non-structural members in to concrete. This report will focus on cast-in place anchors, post-installed expansion anchors, hooked bolts, undercut anchors and adhesive anchors. Cast-in place anchors are the simplest, and strongest type of anchor and comprised of either a hex head bolt with washer. Post-installed anchors can be installed in any position of hardened concrete after a drilling operation and they are divided into two based on their principle of operation. Torque-controlled anchors are installed by applying a specific torque to the bolt head or nut with a torque wrench which causes a wedge being driven up a sleeve which expands and causes the anchor to compress against the material it is being fastened to. Displacement controlled anchors consist of an expansion sleeve and a conical expansion plug whereby the sleeve is internally threaded to accept a threaded element.

In undercut anchors, a special drilling operation helps create contact surface between the hole in the concrete and the anchor head for loads to transfer. Adhesive anchors consist of a special organic glue-like material that helps to transfer stresses. The anchoring material that binds the concrete into the bolt is an adhesive and it generally consists of epoxy, polyester, or vinyl ester resins. The main downside with adhesive anchors is that the performance in terms of load-bearing capacity depends on cleaning condition of the drilled hole. Through experiment it was found that a reduction of as much as 60% in tensile strength capacity was observed based on the condition of the hole as well as a reduction of 20% was observed if the hole in the concrete was wet.

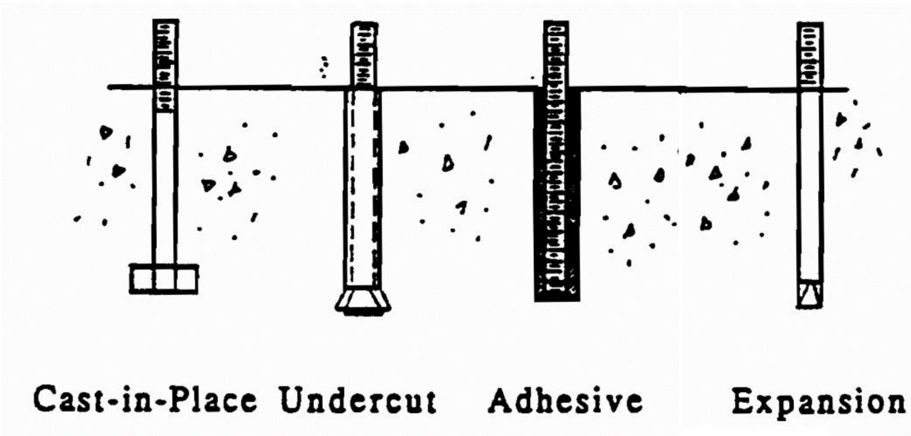


Figure 1

Anchor bolts transfer shear and tensile forces between different structural elements as seen from Figure 1. For all types of anchors, the load is transferred via mechanical interlock, the part of the anchor that is embedded in the concrete transfers the applied shear or axial load via bearing pressure at the contact zone. At failure conditions the bearing pressure can be in excess of ten times the concrete's compressive strength if a pure tension force is transferred.

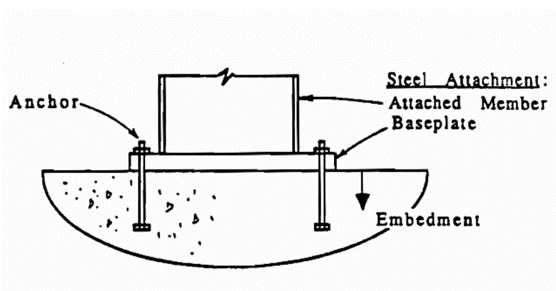


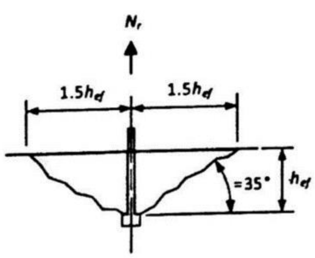
Figure 2

2. Design Procedure and Background

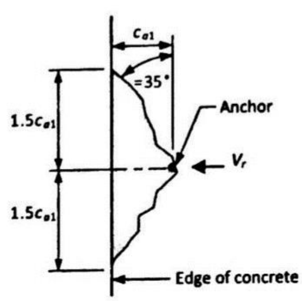
2.1. Failure Modes

When designing anchors, the capacity of the steel anchor as well as the concrete which the anchor is embedded into are the two main factors which need to be considered. The Concrete Capacity Design, CCD, method which is used in anchor design considers fracture mechanics to utilize a 35^{o} projected failure surface for the embedded portion of the anchor. In the CCD method, the capacity expressions have been determined according to extensive database of experimental tests that have been conducted for the past 30 years. A plastic approach requires the anchors to be installed in a way that promotes a brittle failure where concrete breakout occurs before steel yields which should be avoided unless a sufficient ductility is provided by the anchors that allows a redistribution of forces.

Concrete breakout failure mode occurs when the concrete which the anchor is embedded in cracks and fails resulting in the anchor being pulled out. This failure mode is brittle in nature and the concrete breakout cone is depicted in Figure 3 below. The concrete breakout capacity can be calculated from equation 1 seen below. Concrete pullout failure applies only to cast-in headed and hooked bolts. It occurs when the anchor does not have sufficient development length and results in the anchor being pulled out of the embedded concrete it is in. The pullout capacity of the anchor can be increased by using washers or plates which increases the bearing area. The pullout capacity of the anchor is calculated from equation 2.



a) Breakout cone for tension



Plan view

b) Breakout cone for shear

Figure 3

Steel failure mode is another form of failure and the most desirable one, since it is ductile in nature hence making it the safest. Steel failure occurs when the stresses in the anchor exceeds the ultimate stress steel can support causing it to yield and eventually rupture. According to Canadian standards where sufficient development length is provided on both sides of the breakout surface, the design strength of the anchor reinforcement may be used, as calculated from equation 3, instead of the concrete breakout strength. Anchor reinforcement comes from development length which is the minimum length required to develop the full tensile strength of the anchor. (Chalk Example) The thicker the anchor, the longer its' length has to be, the portion that is embedded in concrete in order to develop enough grip with the concrete. Side face blow out failure occurs when the lateral pressure which develops at the end which is embedded in the concrete exceeds the confining strength of the surrounding concrete. Equations 4 and 5 predict the side face blowout capacities for single and group anchors within a distance, $1.4xh_{ef}$, (effective anchor embedment depth) from a free edge.

It should be noted that these equations are not applicable to J or L bolt anchors for which there are currently no formulas representing the side face blow out capacity. Therefore, designers are encouraged to use headed anchors in close proximity to free-edges. The capacity of adhesive anchors is calculated using Equation 6 where N_{bar} represents the bond capacity of a single adhesive anchor without taking into consideration the cracking of the concrete, and edge effects. Bond capacity represents another failure type. It occurs when the bond provided by the adhesive anchor fails and causes the anchor to fail. In addition to the failure modes in tension an anchor can also fail in shear as well. The shear resistance of cast-in headed stud anchors is given by equation V_{sar}. This term represents the strength of the steel anchor in shear. Concrete breakout can also occur if the shear force in the anchor exceeds concrete's capacity which is represented in Equation 132. When an anchor is located near a corner, the designer is required to calculate the shear capacity in both orthogonal directions and use the lesser value. The final failure mode, concrete pry-out resistance. For cast-in anchors the pry-out resistance is equal to the tensile concrete breakout capacity for embedment depths less than 65 mm and twice that value for embedment depths greater than 65 mm.

3. Design Equations

$N_{sar} = A_{se,n}\,\Phi_s\, f_{uta}\, R$ x #of studs Equation 1

$N_{pr} = 8A_{brg}\,\Phi_c\, f'_c\, R \quad N_{cpr} = \Psi_{c,p}\, N_{pr}$ Equation 5 and 6

$N_{cbr} = (A_{nc}/A_{nco})\, \Psi_{ed,N}\, \Psi_{c,n}\, \Psi_{cp,n}\, N_{br}$ Equation 2

$N_{cbgr} = (A_{nc}/A_{nco})\, \Psi_{ec,n}\, \Psi_{ed,N}\, \Psi_{c,n}\, \Psi_{cp,n}\, N_{br}$ Equation 3

$N_{ar} = (A_{na}/A_{nao})\, \Psi_{ed,Na}\, \Psi_{cp,na}\, N_{bar}$ Equation 11 Adhesive Anchors

$N_{agr} = (A_{na}/A_{nao})\, \Psi_{ed,Na}\, \Psi_{ec,na}\, \Psi_{cp,na}\, N_{bar}$ Equation 12

where $A_{Nao} = (2c_{Na})^2$ and $c_{Na} = 10d_a(\tau_{uncr}/7.60)^{½}$

$$\Psi ed,n = 0.7 + 0.3\,\frac{ca,min}{1.5hef}$$

$$\Psi ec,n = \frac{1}{1+(2e'_N/3h_{ef})}$$

$\Psi_{ed,N} = 1$ for $c_{a,min} \geq 1.5h_{ef}$ if $c_{a,min} < 1.5h_{ef}$

If $c_{a,min} \geq c_{ac}$ $\Psi_{cp,N} = 1$ if $c_{a,min} < c_{ac}$ then

$$\Psi cp,n = \frac{ca,min}{ca,c} \geq \frac{1.5hef}{ca,c}$$

Equation 19

$$\Psi ec,na = \frac{1}{1 + e'n / cna} \leq 1$$

if $c_{a,min} \geq c_{na}$ $\Psi_{ed,Na} = 1.0$ if $c_{a,min} < c_{na}$

$$\Psi_{ed,na} = 0.7 + 0.3 \frac{c_{a,min}}{c_{na}}$$

if $c_{a,min} < c_{ac}$ then $\Psi_{cp,na} = c_{a,min} / c_{ac}$ If $c_{a,min} \geq c_{ac}$ $\Psi_{cp,na} = 1.0$

$$\Psi_{ed,V} = 0.7 + 0.3 \frac{c_{a2}}{1.5\, c_{a1}}$$

if $c_{a,2} \geq 1.5 c_{a1}$ $\Psi_{ed,V} = 1.0$

$$\Psi_{ec,v} = \frac{1}{1 + (2e'_v / 3 c_{a1})}$$

$$\Psi_{h,v} = \sqrt{\frac{1.5\, c_{a1}}{h_a}}$$

Equation 18

$N_{bar} = \lambda_a \Phi_c \pi h_{ef} \tau_{cr} d_a R$ Equation 10

$(N_f/N_r)^{\mu} + (V_f/V_r)^{\mu} \leq 1.0$

$N_{br} = k_c \Phi_c \lambda_a (f'_c)^{1/2} (h_{ef})^{5/3} R$ Equation 4

$N_{br} = 3.9 \Phi_c \lambda_a (f'_c)^{1/2} (h_{ef})^{5/3} R$ for $3d_a < e_n < 4.5d_a$

$N_r = A_s \Phi_s f'_y R$

$N_{pr} = 0.9 \Phi_c f'_c e_n d_a R$ Equation 7 (j-bolt) $d_a < h_{ef} < 625$ mm

$N_{sbr} = 13.3\, c_{a1} \lambda_a \Phi_c (A_{brg})^{1/2} (f'_c)^{1/2} R$ Equation 8

$$N_{sbgr} = \left(1 + \frac{s}{6(ca1)}\right) Nsbr$$

Equation 9

$V_{sar} = A_{se,v} \Phi_s f_{uta} R$

$V_{sar} = 0.6\, A_{se,v} \Phi_s f_{uta} R$ Equation 13

$V_{cbr} = (A_{vc}/A_{vco}) \Psi_{ed,v} \Psi_{c,v} \Psi_{h,v} V_{br}$ Equation 14

$V_{cbgr} = (A_{vc}/A_{vco})\ \Psi_{ec,n}\ \Psi_{ed,N}\ \Psi_{c,n}\ \Psi_{cp,n}\ V_{br}$ Equation 15

$V_{br} = 0.58\ (l_e / d_a)^{0.2}\ \Phi_c\ \lambda_a\ (f'_c)^{1/2}\ (c_{a1})^{1.5}\ R$ Lesser Equation 16

$V_{br} = 3.75\ \Phi_c\ \lambda_a\ (f'_c)^{1/2}\ (c_{a1})^{1/2}\ R$

$A_{vco} = 4.5\ (c_{a1})^{½}$

$V_r = A_s\ \Phi_s\ f'_y\ R$ R=0.85

$V_{cpr} = k_{cp}\ N_{cbr}$

$V_{cpgr} = k_{cp}\ N_{cpgr}$

3.1. Design Procedure

The design process of an anchor consists of checking individually for each failure criteria for a headed stud and then taking the lowest strength as the governing failure mode. For example, if the anchor size is given, the tensile capacity of the steel stud is calculated first according to Equation 1. In that equation, the term f_{uta} represents the specified tensile strength of the steel anchor. The term, R, is the resistance modification factor and and is rounded down, for anchors governed by ductile steel element it is 0.8 for tension and 0.7 for shear, for anchors governed by brittle steel element R is 0.7 for tension and 0.65 for shear loads. If the failure mode is governed by concrete break-out, side face blow-out, pull-out or pry-out strength the R value according to CSA is given in the table below. The area $A_{se,N}$ represents effective cross sectional area of anchor in tension.

	Condition A	Condition B
Shear Loads	1.15	1.0
Tension Loads	1.15	1.0
Category 1 Post Installed Anchors	1.15	1.0
Category 2 Post Installed Anchors	1.0	0.85
Category 3 Post Installed Anchors	0.85	0.75

Table 1

The concrete break-out for tensile resistance is represented by equation 2 and 3 for individual and group anchors respectively for uncracked concrete. The term A_{Nc}/A_{Nco} accounts for presence of adjacent anchors and/or free edges. A_{Nco} is the projected area of a 35^{o} failure plane of a single anchor measured relative to the surface of the concrete as depicted in Figure 4 A_{Nc} is a similarly projected area limited by edges and adjacent anchors. For single anchors located distant from free edges the A_{Nc}/A_{Nco} term equals to 1.0. N_{br} needs to be calculated first and it represents the factored concrete breakout resistance in tension of a single anchor in cracked concrete. N_{br} is calculated according to equation 4. The term h_{ef} is the effective anchor embedment depth. The term λ_a is a factor to account low-density concrete in certain concrete anchorage applications and k_c is a coefficient for factored concrete breakout resistance in tension. The Ψ terms in equations 2 and 3 represent modification factors for various different criterias such as tensile strength based on proximity to edges, presence of cracks etc.

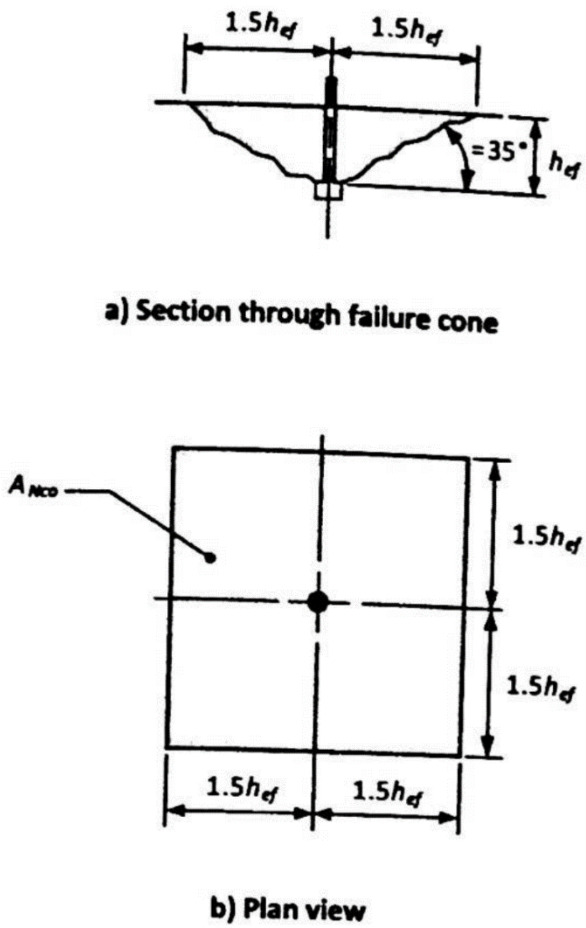


Figure 4

The pull-out capacity equations in the handbook only apply to cast-in headed and hooked bolts only. Equations 5 and 6 are used to determine the cast-in anchors pull-out capacity. For hooked-bolts as seen in Figure 5, the expression to calculate the pull-out capacity is given in equation 7.

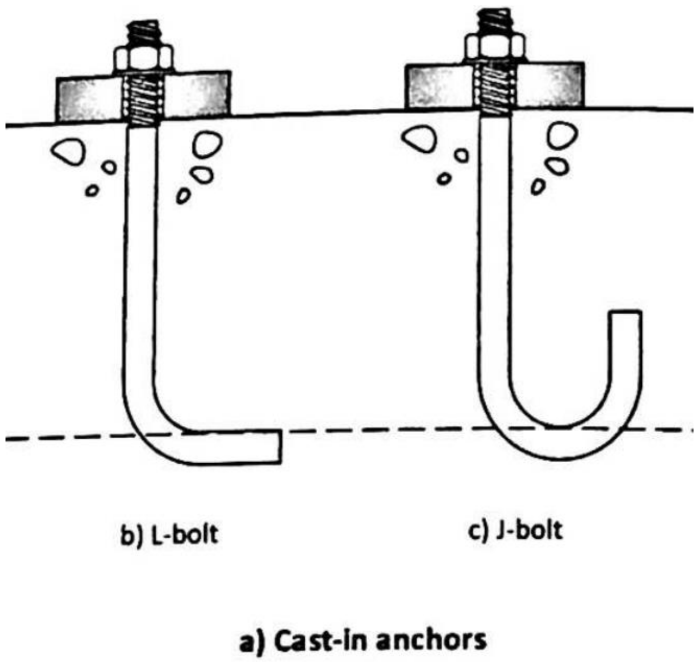


Figure 5

Side-face blowout failure must be considered when $h_{ef} \geq 2.5c_{a1}$ where c_{a1} is the distance from the center of an anchor shaft to the edge of concrete in one direction and h_{ef} is the effective embedment anchor depth. Equations 8 and 9 are used to predict the capacity for a side-face blowout. Edge distances greater than $0.4h_{ef}$ are not required to be considered for side face blowout failure mode. The group effects are considered using equation 9 when the spacing between anchors is less than $6c_{a1}$. The term s is distance between the outer anchors along the edge in a group. For J or L bolts these equations are not applicable for which there are currently no side face blowout expressions. Designers are encouraged to use headed anchors in close proximity to free edges.

For adhesive anchors, the expression for the factored resistance in tension is the same as the expression for cast-in place and mechanical post installed anchors and it is given in equations 11 and 12 both for single and group effects. A_{Nao} is the projected influence area of a single adhesive anchor and c_{Nao} is the projected distance from center of an anchor to develop the full bond strength of a single adhesive anchor. The factored bond resistance of a single adhesive anchor in tension for cracked concrete is given in Equation 10. Equations 11 and 12 represent the factored bond resistance in tension for a single adhesive anchor and a group of adhesive anchors respectively. A_{na} is the projected influence area of a single adhesive anchor or group of adhesive anchors.

Anchor reinforcement is also a viable alternative when the concrete breakout capacity cannot be sufficiently utilized to develop the required shear capacity of the anchor. Clause D7.2.9 permits the use of anchor reinforcement and detailing is shown below in Figure 9. The performance of anchor reinforcement is governed by the location and detailing of the anchor reinforcement. In addition, the anchor reinforcement needs to be sufficiently developed on both sides of the concrete failure surface and/or be properly anchored beyond the breakout section.

3.2. Steel Anchor Shear Design

Similar to anchor design for tension, Clause D.7.1.2 limits f_{uta} of the steel anchor to $1.9f_{ya}$ in order to ensure the steel anchor does not yield under service loads. The main reason for this limit is because of anchor behaviour under stress, it does not exhibit a well defined yield point. The shear resistance of cast-in headed stud anchors is represented by Equation 13. Φ_s factor represents a 20% reduction in shear capacity as required by Clause D.7.1.3 which is due to the effect of flexural stresses that develop in the anchor if the supporting grout pad fractures under the applied shear load. $A_{se,v}$ is included to represent the net cross-sectional area of either a single steel anchor or all the anchors in shear in a group.

The concrete breakout capacity in shear is represented, when the anchor is loaded in shear perpendicular to a free edge, for both single and group anchors respectively in Equations 14 and 15. Similar to design in tension, a 35^{o} breakout prism angle is assumed. First, basic concrete breakout capacity is determined according to Equation 16 and then by applying additional factors which take into account cracking in concrete, group effects, eccentricity of loadings, and edge conditions. The term A_{vc}/A_{vco} represents the ratio of the shear breakout area of a group anchor arrangement versus the full shear breakout area of a single anchor unaffected by edge distance, spacing, or section depth. The term λ_a represents additional strength reductions depending on the density of the concrete for certain anchor types and for bond resistance of adhesive anchors. The factor $\Psi_{ec,v}$ accounts for eccentric shear load application on an anchor group. The shear load is assumed to be oriented towards a free edge. If the edge distances limit the breakout area of the anchor in shear, additional reductions in the calculation of breakout strength of concrete must be taken into account. This is done using the factor $\Psi_{ed,v}$. $\Psi_{h,v}$ is a factor to take into account the fact that, reductions in shear strength for sections of limited depth is not directly proportional to the reduction in shear breakout area of the concrete. It is represented in Equation 18 and is always greater than or equal to 1.0.

The concrete pryout resistance for cast-in anchors has been found to be equal to the tensile concrete breakout capacity for embedment depths less than 65 mm and twice that value for embedment depths that are greater than 65 mm. Lastly, the interaction effects for tensile and shear need to be considered and are represented in Equation 19. The term μ in the equation varies from 1.0 to 2.0. The provisions in Clause D.8 assume μ= 5/3. According to tests a tolerance of 20% can be accommodated before interaction effects need to be considered for cases involving both tension and shear effects.

4. Design Remarks

When designing anchors for tension and/or shear, special attention must be given to certain criterias which can directly impact the performance of an anchor. First and foremost Clause D.4.1.2 specifies when anchor group effects need to be considered. It states that whereever two or more anchors have spacing less than either $3h_{ef}$ $2c_{na}$ and $3c_{a1}$. These three terms represent failure modes, concrete breakout strength in tension, bond strength in tension, and concrete breakout in shear respectively. Due to the upper limit on tests done on anchorage, the values of f'_c used in calculations shall not exceed 70 MPa for cast-in anchors and 55 MPa for post-installed anchors. Additional testing is required for post-installed anchors used in concrete with f'_c greater than 55 MPa. Anchor design must comply Table 1 as shown below.

Failure Mode	Single Anchor	Anchor Group	
		Individual Anchor in a Group	**Anchors as a Group**
Steel Strength in Tension	$N_{sar} \geq N_{fa}$	$N_{sar} \geq N_{fa,i}$	
Concrete Breakout Strength in Tension	$N_{cbr} \geq N_{fa}$		$N_{cbgr} \geq N_{fa,g}$
Pull-out Strength in Tension	$N_{pr} \geq N_{fa}$	$N_{pr} \geq N_{fa,i}$	
Concrete Side-Face Blowout Strength in Tension	$N_{sbr} \geq N_{fa}$		$N_{sbgr} \geq N_{fa,g}$
Bond Strength of Adhesive Anchor in Tension	$N_{ar} \geq N_{fa}$		$N_{agr} \geq N_{fa,g}$
Steel Strength in Shear	$V_{sar} \geq V_{fa}$	$V_{sar} \geq V_{fa}$	
Concrete Breakout Strength in Shear	$V_{cbr} \geq V_{fa}$		$V_{cbgr} \geq V_{fa,g}$
Concrete Pry-out Strength in Shear	$V_{cpr} \geq V_{fa}$		$V_{cpr} \geq V_{fa}$

Table 2

The concrete breakout strength for both tension and shear is not required and can be ignored provided that the anchor has sufficient development length, and anchor reinforcement is provided in accordance with the standards. The factor $\Psi_{cp,n}$ represents modification for uncracked concrete. In cracked concrete it equals 1.0 however for uncracked concrete it is calculated according to Equation 19 if $c_{a,min} < c_{ac}$. For all other cases $\Psi_{cp,n}$ equals 1.0. If an additional washer or a plate is added at the head of the anchor, the projected area of the failure surface may be calculated by projecting the failure surface outward $1.5h_{ef}$ from the effective perimeter of the washer.

When calculating the pullout capacity of a cast-in anchor in uncracked concrete at service load levels, a modifiction factor of $\Psi_{c,p} = 1.4$ may be used. It should also be noted that if a second orthogonal free edge c_{a2} is within $3c_{a1}$ of the anchor, like a corner, Clause D.6.4 reduces the side face blowout strength further by up to 50% as dimension c_{a2} reduces from $3c_{a1}$ to c_{a1}. This reduction factor is represented as $(1+ c_{a2} / c_{a1}) / 4$.

When designing anchors for shear special attention must be paid when the threads in the steel anchor intercept the shear plane, the effective area of the anchor should be taken as being $0.7A_g$. Although this is not explicitly stated in Clause D.7.1.2 CSA Standards require 70% of the gross area be taken as the effective net area A_{se} for threaded bolts. It should also be noted that many post-installed anchors with expansion mechanisms have reduced cross-sectional areas. When this is the case designer should always follow the cross-sectional area data supplied by the manufacturer.

When calculating the concrete breakout in shear, where shear acts parallel to a free edge, Clause D.7.2.1(c) limits the shear force to a maximum value twice the capacity determined for shear forces acting perpendicular to the edge. However, when an anchor is located near a corner, the designer is required to evaluate the shear capacity in both orthogonal directions and limit the shear load to the lesser of the two. For anchor groups, if the anchors are spaced far enough such that their failure surfaces do not intersect then two possible failure conditions should be investigated. If the two anchors are not rigidly connected than the front anchor possesses a smaller failure surface making it the critical condition. In the case where the two anchors are rigidly connected, the shear capacity will be connected by the back anchor. When the front anchor begins to break out, the entire shear load is transferred to the back anchor. Since the back anchor will now carry the entire load, the critical failure surface will become the break-out section associated with that anchor.

Anchors' shear stiffness ratio l_e/d_a is limited to 8.0 due to the extent of the tests done. Anchors with a larger shear stiffness ratio are able to transfer the shear load over a greater depth in the concrete, providing a greater shear resistance. Also when calculating the factor $\Psi_{c,v}$ it is taken as 1.0 if cracking analysis has not been conducted and no reinforcement has provided to control the cracking. If the concrete has been determined that it will remain uncracked at service load levels, this factor $\Psi_{c,v}$ is equal to 1.4, providing a 40% increase in shear capacity of the anchor.

5. Design Examples

5.1. Example 1 Tension Capacity with No Edge Effects

What is the maximum factored tensile capacity of the single AWS D1.1 welded stud anchor shown below? Assume the stud is distant from any edges, the normal density concrete is cracked at service loads, and no supplemental reinforcement is used.

Assume $f'_c = 30$ MPa $\Phi_c = 0.65$ $\Phi_s = 0.85$ $h_{ef} = 125$ mm $f_{ya} = 344$ MPa $f_{uta} = 414$ MPa

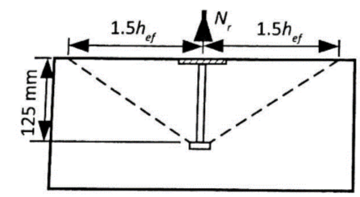


Figure 6

Shank diameter = $d_a = 15.7$ mm

Head diameter $d_{head} = 25.4$ mm

R = 0.8 (ductile steel loaded in tension)

$A_{se,n} = (\pi/4)\, d^2 = (\pi/4)\, 15.7^2 = 193\ \text{mm}^2$

$N_{sar} = A_{se,n}\, \Phi_s\, f_{uta}\, R = 193 x 0.85 x 414 x 0.8 = 54333$ N

Check concrete's breakout capacity R=1.0

$N_{br} = k_c\, \Phi_c\, \lambda_a\, (f'_c)^{1/2}\, (h_{ef})^{5/3}\, R$ $\lambda_a = 1.0$ R = 1.0 $k_c = 10$ for cast-in headed studs

$N_{br} = 10 x 0.65 x 1 x (30)^{½} x (125)^{5/3} x 0.8 = 49755$ N

$N_{cbr} = (A_{nc}/A_{nco})\, \Psi_{ed,N}\, \Psi_{c,n}\, \Psi_{cp,n}\, N_{br}$

$A_{nc} = A_{nco} = 9(h_{ef})^2 = 9(125)^2 = 140625\ \text{mm}^2$ (the anchor is distant from all edges)

$\Psi_{ed,N} = 1.0$ (the anchor is distant from all edges)

$\Psi_{c,n} = 1.0$ (cracked concrete)

$\Psi_{cp,n} = 1.0$ (not applicable, taken as 1.0 for cast-in anchors)

$N_{cbr} = 1x1x1x1x49755 = 49755$ N

Check concrete's pullout capacity

$N_{pr} = 8A_{brg}\, \Phi_c\, f'_c\, R$

$A_{brg} = (\pi/4)\, (d^2_{head} - d^2_a) = (\pi/4)\, (25.4^2 - 15.7^2) = 313\ \text{mm}^2$

R = 1.0 (for pullout capacity always assume condition B)

N_{pr} = 8x313x0.65x30x1 = 48843 kN

For cracked concrete $\Psi_{c,p} = 1.0$

$N_{cpr} = \Psi_{c,p} N_{pr}$ = 1.0x48843 = 48843 N

Least capacity governs therefore maximum factored tensile capacity: **N_{cpr} = 48.8 kN**

5.2. Example 2 Shear Capacity with No Edge Effects

Determine the factored shear capacity of the 19 mm (¾") threaded Grade A ASTM A307 anchor shown. Assume uncracked 30 MPa concrete with f_{ya} = 250 MPa, f_{uta}= 414 MPa, and normal density concrete.

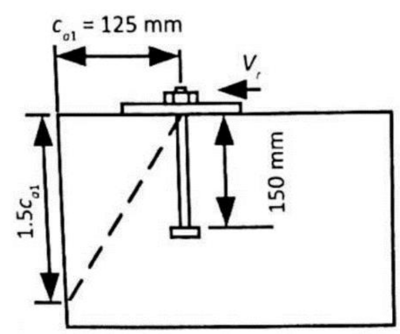


Figure 7

$f_{uta} / f_{ya} = 1.65 < 1.9$ passes the check

$V_{sar} = 0.6A_{se,v} \Phi_s f_{uta} R$

R = 0.75 for ductile steel anchor loaded in shear

$A_{se,v} = 0.7(A_g) = 0.7 (\pi d^2{}_a) = 0.7 (\pi x19^2) = 198\ mm^2$

V_{sar} = 0.6x198x0.85x414x0.75 = 31.354 kN

Check the concrete's breakout capacity in shear

$V_{cbr} = (A_{vc}/A_{vco}) \Psi_{ed,v} \Psi_{c,v} \Psi_{h,v} V_{br}$

Where $V_{br} = 0.58 (l_e / d_a)^{0.2} \Phi_c \lambda_a (f'_c)^{1/2} (c_{a1})^{1.5} R$

l_e / d_a = 150/19 = 7.89 < 8 therefore it is good!

R = 1.0 for shear load without supplementary reinforcement, i.e. condition B

λ_a = 1.0 for normal density concrete

C_{a1} = 125 mm as seen from the figure

$V_{br} = 0.58x(7.89)^{0.2}x0.65x1x(30)^{½} x(125)^{1.5} x1 = 19012$ N

$\Psi_{ed,v} = 1.0$ (no edge effects considered)

$\Psi_{c,v} = 1.4$ assume concrete is uncracked at service loads

$$\Psi h,v = \sqrt{\frac{1.5\, c_{a1}}{h_a}}$$

$\Psi_{h,v} = ((1.5x125)/200)^{½} = 0.98$ but $\Psi_{h,v} \geq 1.0$ therefore use 1.0

$V_{cbr} = 1.0x1.0x1.4x1.0x19.0 = 26.6$ kN

Check the pryout capacity

$V_{cpr} = k_{cp} N_{cbr}$ where $k_{cp} = 2.0$ for $h_{ef} \geq 65$ mm pryout capacity is taken as twice the tensile breakout capacity

$N_{cbr} = (A_{nc}/A_{nco})\, \Psi_{ed,N}\, \Psi_{c,n}\, \Psi_{cp,n}\, N_{br}$

$A_{Nc} = (125+1.5h_{ef})x3h_{ef} = (125+(1.5x150))x3x150 = 157500$ mm^2

$A_{nco} = 9h^2_{ef} = 9x150^2 = 202500$ mm^2

$$\Psi ed,n = 0.7 + 0.3 \frac{ca,min}{1.5hef}$$ if $c_{a,min} < 1.5h_{ef}$

$\Psi_{ed,N} = 0.7 + 0.3(125/(1.5x150)) = 0.867$

$\Psi_{c,n} = 1.25$ for uncracked concrete

$\Psi_{cp,n} = 1.0$ for cast-in-place anchor

$N_{br} = k_c\, \Phi_c\, \lambda_a\, (f'_c)^{1/2}\, (h_{ef})^{5/3}\, R$ $R = 1.0$ $k_c = 10$ for cast-in headed studs

$N_{br} = 10x0.65x1x(30)^{½} x (150)^{1.5} x 1.0 = 65404$ N

$N_{cbr} = (157500/202500)(0.867)x1.25x1.0x65404 = 55100$ N

$V_{cpr} = k_{cp} N_{cbr} = 2x55100 = 110200$ N $= 110.2$ kN

Least capacity governs therefore maximum factored shear capacity: $\mathbf{V_{cbr} = 26.6\ kN}$

5.3. Example 3 Group of Headed Studs in Tension Near an Edge

Design a group of four AWS D1.1 Type B welded headed studs spaced 150 mm center to center each way and eccentrically loaded with a factored load, $N_f = 45$ kN. Assume $f'_c = 30$ MPa $\Phi_c = 0.65$ $\Phi_s = 0.85$ and ductile steel $f_{uta} = 414$ MPa $f_{uta} / f_{ya} < 1.9$.

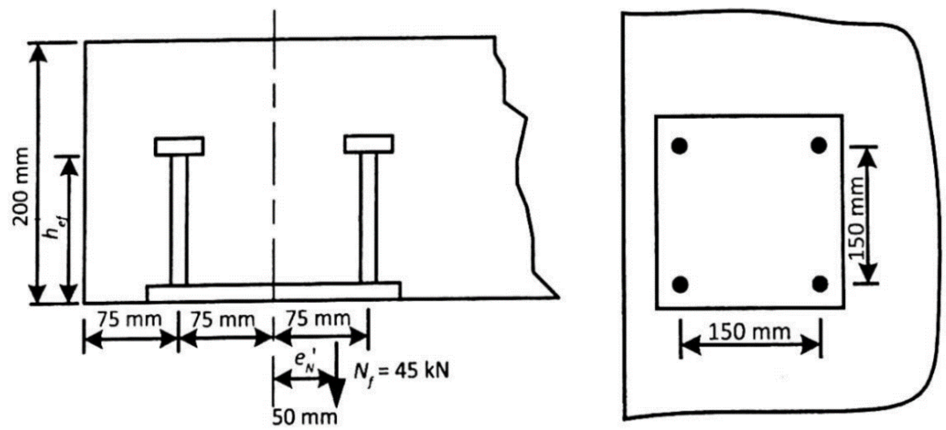


Figure 8

As seen from the above figure, the load is applied eccentrically. Due to this eccentricity, in order to find the load in each anchor, sum moments about the left anchor and solve for the force in right anchor. Then divide this number by 2 since there are two interior studs for conservative design to get N_{sar}.

Right Anchor=45x(75+50)/(75+75) = 37.5 kN $\quad$ N_{sar} =37.5/2 = 18.7 kN

N_{sar} = 18700 = $A_{se,n}\, \Phi_s\, f_{uta}\, R$ = 0.85x0.8x414x$A_{se,n}$ $\quad$ $A_{se,n}$ = 66.4 mm^2

From Table 3 below select an anchor size keeping in mind that the anchor will also need to resist the pullout force as well. ½” diameter stud with $A_{se,n}$ = 127 mm^2 is selected.

Provided: $N_{sar} = A_{se,n}\, \Phi_s\, f_{uta}\, R$ x#of studs = 127x0.85x414x0.8x4 = 143012 N > N_f

$N_{cbgr} = (A_{nc}/A_{nco})\, \Psi_{ec,n}\, \Psi_{ed,N}\, \Psi_{c,n}\, \Psi_{cp,n}\, N_{br}$

$A_{nc} = (75+150+1.5h_{ef}) \times (1.5h_{ef} + 150 + 1.5h_{ef}) = (75+150+172)x(172 + 150+ 172)$

$=196118 \text{ mm}^2$

$A_{nco} = 9h^2_{ef} = 9(115)^2 = 119205 \text{ mm}^2$ (failure surface without edge effects)

4x119205 = 476280 > 196118 mm^2 Passes check for group effect

Calculate the coefficient for eccentricity

$$\Psi ec, n = \frac{1}{1+(2e'_N/3h_{ef})}$$

$\Psi_{ec,n} =$ 1/(1+(2(50) / 3x115)) = 0.775

$$\Psi ed,n = 0.7 + 0.3\frac{ca,min}{1.5hef}$$

$\Psi_{ed,n}$ = 0.7 + 0.3x(75/1.5x115) = 0.83

$\Psi_{c,n}$ = 1.0 for for cracked concrete

$\Psi_{cp,n}$ = 1.0 for cast-in anchors

$N_{br} = k_c\, \Phi_c\, \lambda_a\, (f'_c)^{1/2}\, (h_{ef})^{3/2}\, R$ R = 1.0 k_c = 10 for cast-in headed studs

$N_{br} = 10x0.65x1x(30)^{½} \times (115)^{1.5} \times 1.0 = 43.9$ kN

$N_{cbgr} = (A_{nc}/A_{nco})\, \Psi_{ec,n}\, \Psi_{ed,N}\, \Psi_{c,n}\, \Psi_{cp,n}\, N_{br}$

N_{cbgr} = (119118/119025) (0.775x0.83x1.0x1.0)43.9 = 46.4 kN > N_f = 45 kN

Check the pull-out capacity N_{pr} for the two most heavily loaded studs.

$A_{brg} = 380 \text{ mm}^2$ for ½" diameter studs with a 1" diameter head

$N_{pr} = 8A_{brg}\, \Phi_c\, f'_c\, R$

N_{pr} = 8x380x0.65x30x1.0 = 59280 N > 18700 N passes the check!

Least capacity governs therefore maximum factored tensile capacity:

N_{cbgr} = 46.4 kN > N_f = 45 kN which is sufficient to support the factored eccentric load.

Anchor Diameter (in)	Gross Area of anchor (in^2)	Effective Area of Anchor $A_{se,N}$ (in^2)	Bearing Area of Hex Nut A_{brg} (in^2)
0.25	0.049	0.032	0.117
0.375	0.11	0.078	0.164
0.5	0.196	0.142	0.291
0.625	0.307	0.226	0.454
0.75	0.442	0.334	0.654
0.875	0.601	0.462	0.891
1	0.785	0.606	1.163
1.125	0.994	0.763	1.472
1.25	1.227	0.969	1.817
1.375	1.485	1.16	2.199
1.5	1.767	1.41	2.617
1.75	2.405	1.9	4.144
2	3.142	2.5	5.316

Table 3

Anchor Diameter (mm)	Gross Area of anchor (mm^2)	Effective Area of Anchor $A_{se,N}$ (mm^2)	Bearing Area of Hex Nut A_{brg} (mm^2)
6.35	32	21	108
9.525	71	50	193
12.7	127	92	301
15.875	198	146	433
19.05	285	215	588
22.225	388	298	766
25.4	507	391	968
28.575	641	492	1194
31.75	792	625	1443
34.935	959	748	1715
38.1	1140	910	2012
44.45	1552	1226	2674
50.8	2027	1613	3430

Table 4

6. Anchor Design in ACI Standard

Similar to the Canadian Codes, the design process of an anchor in ACI Standards consists of checking individually for each failure criteria for a headed stud and then taking the lowest strength as the governing failure mode. Similar to the Canadian Code, ACI Standards use the Concrete Capacity Design, CCD, method which considers fracture mechanics to utilize a 35° projected failure surface for the embedded portion of the anchor. This failure plane can be seen below in Figure 9**.** The design formulas in the two codes appear similar to each other in that both use elastic analysis in determining critical effects of factored loads in anchor and anchor groups.

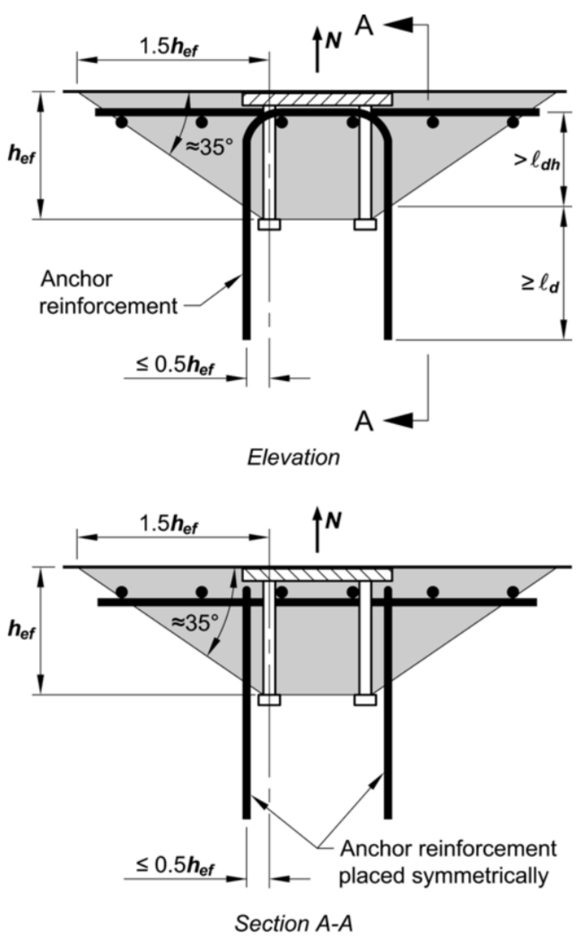


Figure 9

Specifically, for example when calculating the design strength of the anchor the only difference is that in the Canadian Codes, an appropriate resistance factor Φ_s is used and also the resistance modification factor, R, is added in accordance with Clause D.5.3. When calculating the basic concrete breakout strength and the pull-out capacity of a single anchor in tension in cracked concrete the only additions in the Canadian Codes are the Φ_c factor for concrete compressive strength and the resistance modification factor, R. The factors used in ACI Standards, for taking group effects into account in concrete breakout strength, do not change and they remain the same. However for cast-in headed studs and headed bolts with 11 in $\leq h_{ef} \leq$ 25 in. The equation is given as $N_{br} = 16 \lambda_a (f'_c)^{1/2} (h_{ef})^{5/3}$.

The concrete side-face blowout resistance of a headed anchor in tension is also similar in the ACI Standards except for the exclusion of R and a different coefficient number in the beginning of the equation. This coefficient is different because units used in the American Standards are empirical whereas the Canadian Code is written in metric. The equation in ACI is given as $N_{sbr} = 160\, c_{a1}\, \lambda_a (A_{brg})^{1/2} (f'_c)^{½}$. The factored bond strength of adhesive anchors given in ACI Standards is very similar to the Canadian Codes. The only difference is the exclusion of the Φ_c factor and R. Also C_{Na} is calculated with different numbers in the formula due to the differences in the units used in the equation.

Design requirements in shear for anchors in ACI standards is again similar to Canadian Codes. The maximum factored resistance of an anchor in shear, V_{sar}, and concrete breakout strength of anchor in shear for both standards is exactly the same with the only differences appearing due to the differences in units used for ACI and Φ_c factor and R. In addition, the concrete pry-out resistance of an anchor in shear in ACI Standards is exactly same as the Canadian Codes.

7. Conclusion

When designing anchors, both the strength capacity of steel and concrete the anchor is embedded into must be considered. The Concrete Capacity Design method currently used in both ACI and Canadian Standards utilizes a 35^{o} projected failure surface for the embedded portion of the anchor. The expressions used in this method have been calibrated to the extensive database of experimental tests that have been conducted in the last 30 years. The CCD Method addresses a variety of anchor types, and design conditions, while providing a more consistent level of safety for a wider range of anchorage failure modes for both cracked and uncracked concrete.

This project discussed various different types of anchors, their design, and different failure modes that can occur under different loadings. Anchors transfer tensile and shear loads via a mechanism called mechanical interlock. The contact points in-between the anchor and concrete transfers the shear and tensile load via bearing pressure. At failure, this bearing pressure can be in excess of ten times the concrete's compressive strength.

8. References

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