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3.0 Anchoring Systems

3.1 Anchor principles and design

3.1.1 Definitions

Adhesive anchor is a post-installed anchor that is inserted into a drilled hole in hardened concrete, masonry or stone. Loads are transferred to the base material by the bond between the anchor and the adhesive and the adhesive and the base material.

Anchor category is an assigned rating that corresponds to a specific strength reduction factor for concrete failure modes associated with anchors in tension. The anchor category is established based on the performance of the anchor in reliability tests.

Anchor group is a group of anchors of approximately equal effective embedment and stiffness where the maximum anchor spacing is less than the critical spacing.

Anchor reinforcement is reinforcement used to transfer the full design load from the anchors into the structural member.

Anchor spacing is centerline-to-centerline distance between loaded anchors.

Attachment is the structural assembly, external to the surface of the concrete, that transmits loads to or receives loads from the anchor.

Cast-in-place anchor is traditionally a headed bolt, headed stud or hooked bolt installed before placing concrete. Additionally, cast-in-place internally threaded inserts are a form of cast-in-place anchors.

Characteristic capacity is a statistical term indicating 90 percent confidence that there is 95 percent probability of the actual strength exceeding the nominal strength. This is also called the 5% fractile capacity.

Concrete breakout is a concrete failure mode that develops a cone or edge failure of the test member due to setting of the anchor or applied loads.

Concrete splitting failure is a concrete failure mode in which the concrete fractures along a plane passing through the axis of the anchor or anchors.

Cracked concrete is condition of concrete in which the anchor is located. See Section 2.1.2.

Critical spacing is minimum required spacing between loaded anchors to achieve full capacity.

Critical edge distance is minimum required edge distance to achieve full capacity.

Cure time is the elapsed time after mixing of the adhesive material components to achieve a state of hardening of the adhesive material in the drilled hole corresponding to the design mechanical properties and resistances. After the full cure time has elapsed, loads can be applied.

Displacement controlled expansion anchor is a post-installed anchor that is set by expansion against the side of the drilled hole through movement of an internal plug in the sleeve or through movement of the sleeve over an expansion element (plug). Once set, no further expansion can occur.

Ductile steel element are anchors designed to be governed by ductile yielding of the steel. This is determined by performing tension testing on coupons machined from the finished anchors. The minimum requirements are 14% elongation and 30% reduction of area.

Expansion anchor is a post-installed anchor that is inserted into a drilled hole in hardened concrete or masonry. Loads are transferred to and from the base material by bearing, friction or both.

Edge distance is distance from centerline of anchor to the free edge of base material in which the anchor is installed.

Effective embedment depth is the overall depth through which the anchor transfers force to or from the surrounding concrete. The effective embedment depth will normally be the depth of the concrete failure surface in tension applications. For cast-in headed anchor bolts and headed studs, the effective embedment depth is measured from the bearing contact surface of the head. For expansion anchors, it is taken as the distance from surface of base material to tip of expansion element(s).

Gel time is the elapsed time after mixing of the adhesive material components to onset of significant chemical reaction as characterized by an increase in viscosity. After the gel time has elapsed, the anchors must not be disturbed.

Minimum edge distance is the spacing from the centerline of the anchor to the edge of the base material required to minimize the likelihood of splitting of the base material during anchor installation.

Minimum spacing is distance between the centerlines of adjacent loaded anchors to minimize the likelihood of splitting of the base material during anchor installation.

Minimum member thickness is minimum required thickness of member in which anchor is embedded to minimize the likelihood of splitting of the base material.

Post-installed anchor is an anchor installed in hardened concrete and masonry. Expansion, undercut, and adhesive anchors are examples of post-installed anchors.

Projected area is the area on the free surface of the concrete member that is used to represent the larger base of the assumed rectilinear failure surface.

Pryout failure is a failure mode where anchors having limited embedment depth and loaded in shear exhibit sufficient rotation to produce a pryout fracture whereby the primary fracture surface develops behind the point of load application.
Anchor Principles and Design 3.1

3.1.2 Anchors in concrete and masonry

Post-installed anchor bolts are used for a variety of construction anchoring applications including column baseplates, supporting mechanical and electric services, fixation of building facades and anchoring guardrails. Critical connections, i.e., those that are either safety-related or whose failure could result in significant financial loss, require robust anchor solutions capable of providing a verifiable and durable load path. In turn, the selection of a suitable anchor system and its incorporation in connection design requires a thorough understanding of the fundamental principles of anchoring. While a general overview is provided here, additional references can be found at the conclusion of this section.

3.1.3 Anchor working principles

Anchors designed for use in concrete and masonry develop resistance to tension loading on the basis of one or more of the following mechanisms:

- **Friction:** This is the mechanism used by most post-installed mechanical expansion anchors to resist tension loads, including the KWIK Bolt TZ, HSL-3 and HDI anchors. The frictional resistance resulting from expansion forces generated between the anchor and the wall of the drilled hole during setting of the anchor may also be supplemented by local deformation of the concrete. The frictional force is proportional to the magnitude of the expansion stresses generated by the anchor. Torque-controlled expansion anchors like the KWIK Bolt TZ and HSL-3 anchors use follow-up expansion to increase the expansion force in response to increases in tension loading beyond the service load level (preload) or to adjust for changes in the state of the base material (cracking).

  **Keying:** Undercut anchors and, to a lesser degree, certain types of expansion anchors, rely on the interlock of the anchor with deformations in the hole wall to resist the applied tension loading. The (bearing) stresses developed in the base material at the interface with the anchor bearing surfaces can reach relatively high levels without crushing due to the triaxial nature of the state of stress. Undercut anchors like the Hilti HDA offer much greater resilience to variations in the base material conditions and represent the most robust solution for most anchoring needs.

- **Bonding (adhesion):** Adhesive anchor systems utilize the bonding mechanism that takes place between the adhesive and the anchor element, and the adhesive and the concrete, to transfer the applied load from the anchor element into the concrete. The degree of bonding available is influenced by the condition of the hole wall at the time of anchor installation. Injection anchor systems like Hilti’s HIT-HY 200 offer unparalleled flexibility and high bond resistance for a wide variety of anchoring applications.

  Hybrid anchor elements like the Hilti HIT-Z threaded rod combine the functionality of an adhesive anchor system with the working principle of a torque-controlled expansion anchor for increased reliability under adverse job-site conditions.

- **Shear resistance:** Most anchors develop resistance to shear loading via bearing of the anchor element against the hole wall close to the surface of the base material. Shear loading may cause surface spalling resulting in significant flexural stresses and secondary tension in the anchor element.

3.1.4 Anchor behavior under load

When loaded in tension to failure, anchors may exhibit one or more identifiable failure modes. These include:

- steel failure in tension
- anchor pullout or pull-through failure
- adhesive bond failure
- concrete breakout failure
- concrete splitting failure
- side-face blowout failure
Failure modes associated with anchors loaded to failure in shear may be characterized as follows:

- steel failure in shear/tension
- concrete edge breakout failure
- pryout failure

### 3.1.4.1 Prestressing of Anchors

In general, properly installed anchors do not exhibit noticeable deflection at the expected service load levels due to the application of the prescribed installation torque. External tension loading results in a reduction of the clamping force in the connection with little increase in the corresponding bolt tension force. Shear loads are resisted by a combination of bearing and friction resulting from the anchor preload forces.

At load levels beyond the clamping load, anchor deflections increase and the response of the anchor varies according to the anchor force-resisting mechanism. Expansion anchors capable of follow-up expansion show increased deflections corresponding to relative movement of the cone and expansion elements. Adhesive anchors exhibit a change in stiffness corresponding to loss of adhesion between the adhesive and the base material whereby tension resistance at increasing displacement levels is provided by friction between the uneven hole wall and the adhesive plug. In all cases, increasing stress levels in the anchor bolt/element result in increased anchor displacements.

### 3.1.4.2 Long term behavior

Following are some factors that can influence the long-term behavior of post-installed anchoring systems.

**Adhesive anchoring systems:**
- Pretensioning relaxation
- Chemical resistance/durability
- Creep
- Freeze/thaw conditions
- High temperature

**Mechanical anchoring systems:**
- Pretensioning relaxation
- Fatigue
- Concrete cracking
- Corrosion
- Creep
- Fire
- Seismic loading

### 3.1.5 Anchor design

The design of anchors is based on an assessment of the loading conditions and anchorage capacity. Strength design (SD), limit state design (LSD), and allowable stress design (ASD) methods are currently in use in North America for the design of anchors.

**Strength Design:** The Strength Design Method for anchor design has been incorporated into several codes such as IBC and ACI 318. The method assigns specific strength reduction factors to each of several possible failure modes, provides predictions for the strength associated with each failure mode, and compares the controlling strength with factored loads. The Strength Design Method is a more accurate estimate of anchor resistance as compared to the ASD approach. The Strength Design Method, as incorporated in ACI 318-14 Chapter 17, is discussed in Section 3.1.6. Strength Design is state-of-the-art and Hilti recommends its use where applicable.

**Limit State Design:** The limit state design method for anchor design is described and included in the CSA A23.3 Annex D. In principle, the method follows the strength design concept with the application of different strength reduction factors. The limit states design method generally results in a more accurate estimate of anchor resistance as compared to the ASD approach. This approach is discussed further in 3.1.7.

**Allowable loads:** Under the Allowable Stress Design Method, the allowable load, or resistance, is based on the application of a safety factor to the mean result of laboratory testing to failure, regardless of the controlling failure mode observed in the tests. The safety factor is intended to account for reasonably expected variations in loading. Adjustments for anchor spacing and edge distance are developed as individual factors based on testing of two- and four-anchor groups and single anchors near free edges. These factors are multiplied together for specific anchor layouts. This approach is discussed further in section 3.1.9. Allowable Stress Design is typically used today for masonry applications.

### 3.1.6 ACI 318 Chapter 17 Strength Design - SD (LRFD)

Strength Design of anchors is referenced in the provisions of ACI 355.2, ACI 355.4, ACI 318 -14 Chapter 17 and the ICC-ES Acceptance Criteria AC193 for mechanical anchors and AC308 for adhesive anchors. A summary of the relevant design provisions, especially as they pertain to post-installed anchors, is provided here.
3.1.6.1 Strength Design (SD) terminology

- \( A_{nc} \) = projected concrete failure area of a single anchor or group of anchors, for calculation of strength in tension, in.\(^2\) (mm\(^2\))
- \( A_{na} \) = projected influence area of a single adhesive anchor or group of adhesive anchors, for calculation of bond strength in tension, in.\(^2\) (mm\(^2\))
- \( A_{nco} \) = projected concrete failure area of a single anchor, for calculation of bond strength in tension if not limited by edge distance or spacing, in.\(^2\) (mm\(^2\))
- \( A_{se,N} \) = effective cross-sectional area of anchor in tension, in.\(^2\) (mm\(^2\))
- \( A_{se,V} \) = effective cross-sectional area of anchor in shear, in.\(^2\) (mm\(^2\))
- \( A_{se} \) = tensile stress area of threaded part, in.\(^2\) (mm\(^2\))
- \( A_{vc} \) = projected concrete failure area of a single anchor or group of anchors, for calculation of strength in shear, in.\(^2\) (mm\(^2\))
- \( A_{vco} \) = 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.\(^2\) (mm\(^2\))
- \( c \) = distance from anchor centerline to the closest free edge of concrete, in. (mm)
- \( c_{ac} \) = critical edge distance required to develop the basic strength as controlled by concrete breakout or bond of a post-installed anchor in tension in uncracked concrete without supplementary reinforcement to control splitting, in. (mm)
- \( c_{a,max} \) = maximum distance from the center of an anchor shaft to the edge of concrete, in. (mm)
- \( c_{a,min} \) = minimum distance from the center of an anchor shaft to the edge of concrete, in. (mm)
- \( c_{a1} \) = distance from the center of an anchor shaft to the edge of the concrete in one direction, in. (mm); If shear is applied to anchor, \( c_{a1} \) is taken in the direction of the applied shear; If tension is applied to the anchor, \( c_{a1} \) is the minimum edge distance, in. (mm)
- \( c_{a2} \) = distance from center of an anchor shaft to the edge of concrete in the direction perpendicular to \( c_{a1} \), in. (mm)
- \( c_{r,Na} \) = projected distance from the center of an anchor shaft on one side of the anchor required to develop the full bond strength of a single adhesive anchor, in. (mm)
- \( d \) = outside diameter of anchor or shaft diameter of headed stud, headed bolt, or hooked bolt, in. (mm)
- \( d_{bit} \) = nominal drill bit diameter, in. (mm)
- \( d_h \) = diameter of clearance hole in attachment (e.g. baseplate), in. (mm)
- \( d_{nom} \) = nominal anchor diameter, in. (mm)
- \( d_o \) = anchor outside diameter (O.D.), in. (mm)
- \( e^{'}_N \) = distance between resultant tension load on a group of anchors loaded in tension and the centroid of the group of anchors loaded in tension, in. (mm)
- \( e^{'}_V \) = distance between resultant shear load on a group of anchors loaded in shear in the same direction, and the centroid of the group of anchors loaded in shear in the same direction, in. (mm)
- \( f'_{c} \) = specified concrete compressive strength, psi (MPa)
- \( f_{ya} \) = specified bolt minimum yield strength, psi (MPa)
- \( f_{uta} \) = specified bolt minimum ultimate strength, psi (MPa)
- \( h \) = thickness of member in which an anchor is located, as measured parallel to anchor axis, in. (mm)
- \( h_{ef} \) = effective anchor embedment depth, in. (mm)
- \( h_{min} \) = minimum member thickness, in. (mm)
- \( h_0 \) = depth of full diameter hole in base material, in. (mm)
- \( k_{cr} \) = coefficient for basic concrete breakout strength in tension, cracked concrete
- \( k_{uncr} \) = coefficient for basic concrete breakout strength in tension, uncracked concrete
- \( k_{cp} \) = coefficient for pryout strength
3.1 Anchor Principles and Design

3.1.6.1 Strength Design (SD) terminology

- \( \ell_e \) = load-bearing length of anchor for shear, in. (mm)
- \( \ell_{th} \) = anchor useable thread length, in. (mm)
- \( M_s \) = characteristic value for the bending moment corresponding to rupture, in-lb (N·m)
- \( n \) = number of anchors in a group
- \( N_a \) = nominal bond strength in tension of a single adhesive anchor, lb (kN)
- \( N_{ag} \) = nominal bond strength in tension of a group of adhesive anchors, lb (kN)
- \( N_{ao} \) = characteristic tension capacity of a single adhesive anchor in tension as limited by bond/concrete failure, lb (kN)
- \( N_b \) = basic concrete breakout strength in tension of a single anchor in cracked concrete, lb (kN)
- \( N_{ba} \) = basic bond strength in tension of a single adhesive anchor, psi (MPa)
- \( N_{cb} \) = nominal concrete breakout strength in tension of a single anchor, lb (kN)
- \( N_{cbg} \) = nominal concrete breakout strength in tension of a group of anchors, lb (kN)
- \( N_{n} \) = nominal strength in tension, lb (kN)
- \( N_{p} \) = pullout strength in tension of a single anchor in cracked concrete, lb (kN)
- \( N_{p,n,c} \) = nominal pullout strength in tension of a single post-installed mechanical anchor, lb (kN)
- \( N_{pn} \) = nominal pullout strength in tension of a single anchor, lb (kN)
- \( N_{sa} \) = nominal strength of a single or individual anchor in a group of anchors in tension as governed by the steel strength, lb (kN)
- \( N_{sb} \) = side face blowout strength of a single anchor, lb (kN)
- \( N_{sbg} \) = side face blowout strength of a group of anchors, lb (kN)
- \( N_{sa} \) = factored tensile force applied to an anchor or an individual anchor in a group of anchors, lb (kN)
- \( s \) = anchor axial spacing, in. (mm)
- \( s_{cr,Na} \) = critical adhesive anchor spacing for tension loading at which the tension capacity of each anchor is theoretically unaffected by the presence of the adjacent loaded anchor, in. (mm)
- \( S \) = elastic section modulus of anchor bolt, in.\(^3\) (mm\(^3\))
- \( t_{fix} \) = maximum thickness of attachment (e.g. baseplate) to be fastened, in. (mm)
- \( T_{\text{inst}} \) = recommended anchor installation torque, ft-lb (N·m)
- \( T_{\text{max}} \) = maximum tightening torque, ft-lb (N·m)
- \( V_b \) = basic concrete breakout strength in shear of a single anchor in cracked concrete, lb (kN)
- \( V_{cb} \) = nominal concrete breakout strength in shear of a single anchor, lb (kN)
- \( V_{cbg} \) = nominal concrete breakout strength in shear of a group of anchors, lb (kN)
- \( V_n \) = nominal strength in shear, lb (kN)
- \( V_{sa} \) = nominal shear strength of a single or individual anchor in a group of anchors as governed by the steel strength, lb (kN)
- \( V_{ua} \) = factored shear force applied to a single anchor or group of anchors, lb (kN)
- \( \phi \) = strength reduction factor
- \( \tau_{cr} \) = characteristic bond stress for cracked concrete conditions taken as the 5 percent fractile of results of tests performed and evaluated according to ACI 355.4 or ICC-ES AC308, psi (MPa)
- \( \tau_{uncr} \) = characteristic bond stress for uncracked concrete conditions taken as the 5 percent fractile of results of tests performed and evaluated according to ACI 355.4 or ICC-ES AC308, psi (MPa)
- \( \psi_{c,N} \) = 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 presence or absence of supplementary reinforcement
In most cases, elastic analysis yields satisfactory results and is recommended. It should be noted, however, that the assumption of anchor load linearly proportional to the magnitude of the applied load and the distance from the neutral axis of the group is valid only if the attachment (e.g. baseplate) is sufficiently stiff in comparison to the axial stiffness of the anchors. For additional information on elastic load distribution in typical column baseplate assemblies, the reader is referred to Blodgett, O., Design of Welded Structures, The James F. Lincoln Arc Welding Foundation, Cleveland, Ohio.

Note: Assuming a rigid base plate condition, Hilti’s PROFIS Anchor analysis and design software performs a simplified finite element analysis to establish anchor load distribution on an elastic basis.

Example of incompatibility of deformations (displacements)

3.1.6.3 General requirements for anchor strength

In accordance with general Strength Design Method principles and ACI 318-14, Section 17.3 and chapter 5, the design of anchors must satisfy the following conditions:

\[ \phi N_n \geq N_{ua} \]
\[ \phi V_n \geq V_{ua} \]

whereby \( \phi N_n \) and \( \phi V_n \) are the controlling design strengths from all applicable failure modes and \( N_{ua} \) and \( V_{ua} \) are the factored tension and shear loads resulting from the governing load combination. The load combinations given in ACI 318-14 Section 5.3 generally conform with ASCE 7-10 load combinations. For this assessment, the following potential failure modes are considered:

a) Steel strength of anchor in tension
b) Concrete breakout strength of anchor in tension
c) Pullout strength cast-in, post-installed expansion or undercut anchor in tension
d) Concrete side-face blowout strength of headed anchor in tension
e) Bond strength of adhesive anchor in tension
f) Steel strength of anchor in shear
g) Concrete breakout strength of anchor in shear
h) Concrete pryout strength of anchor in shear
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Note that as per ACI 318-14 Section 17.3.1, the strength reduction factors applicable for each failure mode must be applied prior to determining the controlling strength.

Thus, for a single anchor, the controlling strength would be determined as follows:

\[
\phi N_n = \min \{ \phi N_{sa}, \phi N_{sa}, \phi N_{sb}, \phi N_{cb} \} \\
\phi V_n = \min \{ \phi V_{sa}, \phi V_{cp}, \phi V_{cb} \}
\]

In analogy, the controlling strength for an anchor group would be determined as

\[
\phi N_n = \min \{ \phi N_{sa}, \phi N_{sa}, \phi N_{sbg}, \phi N_{cbg} \} \\
\phi V_n = \min \{ \phi V_{sa}, \phi V_{cpg}, \phi V_{cbg} \}
\]

In accordance with ACI 318-14 Section 17.2.6, for lightweight concrete conditions, the modification factor \( \lambda \) is taken as:

- 1.0 \( \lambda \) for cast-in and undercut anchor concrete failure
- 0.8 \( \lambda \) for expansion and adhesive anchor concrete failure
- 0.6 \( \lambda \) for adhesive bond failure

where \( \lambda \) is determined in accordance with Section 8.6.1 of the same document. It is permitted to use an alternate value of \( \lambda \), where tests have been performed and evaluated in accordance with ACI 355.2, ACI 355.4, or the relevant ICC-ES acceptance criteria.

3.1.6.4 Strength reduction factors

Strength reduction factors are intended to account for possible reductions in resistance due to normally expected variations in material strengths, anchor installation procedures, etc. Relevant strength reduction factors as given in ACI 318-14 Section 17.3.3 for load combinations in accordance with Section 9.2 of the same document are provided below.

Anchor governed by a ductile steel element:

- Tension loads . . . 0.75
- Shear loads . . . . 0.65

Anchor governed by strength of a brittle steel element (non-ductile):

- Tension loads . . . 0.65
- Shear loads . . . . 0.60

Refer to Section 3.1.3 and ACI 318-14 Section 2.3 for definition of a ductile steel element.

Anchor governed by concrete breakout, side-face blowout, pullout, or pryout strength:

- Condition A
  - Shear loads . . . 0.75 . . . . 0.70
  - Tension loads
    - Cast-in headed studs, headed bolts, or hooked bolts . . . . 0.75 . . . . 0.70
    - Post-installed anchors:
      - Category 1 . . . . . 0.75 . . . . 0.65
      - Category 2 . . . . . 0.65 . . . . 0.55
      - Category 3 . . . . . 0.55 . . . . 0.45

Condition A applies where supplementary reinforcement is present, except for pullout and pryout strengths.

Condition B applies where supplementary reinforcement is not present, and for pullout and pryout strengths.

Anchor categories are determined via testing conducted in accordance with ACI 355.2 or ACI 355.4, wherein the anchor sensitivity to variations in installation parameters and in the concrete condition is investigated.

3.1.6.5 Design requirements for tensile loading

In accordance with ACI 318-14 Section 17.4.1 the nominal steel strength of an anchor in tension is determined as follows:

\[
N_{sa} = A_{se,N} f_{uta} \tag{17.4.1.2}
\]

where \( f_{uta} \leq \min [1.9 f_{yr}, 125,000 \text{ psi (860 MPa)}] \)

Nominal minimum bolt steel yield and ultimate strengths for Hilti anchor products can be found in the product specific sections of this guide.

The nominal concrete breakout strength of a single anchor loaded in tension is determined in accordance with ACI 318-14 Section 17.4.2 as follows:

\[
N_{cb} = \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_b \tag{17.4.2.1a}
\]

The nominal concrete breakout strength of anchor groups is likewise determined as follows:

\[
N_{cbg} = \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_b \tag{17.4.2.1b}
\]

where:

\[
A_{Nco} = \text{projected concrete failure area of a single anchor with an edge distance equal to or greater than } 1.5h_{ef}
\]

\[
= 9h_{ef}^2 \tag{17.4.2.1c}
\]
$A_{NC}$ = projected concrete failure area of a single anchor or group of anchors approximated as the base of the rectilinear geometrical figure that results from projecting the failure surface outward $1.5h_{ef}$ from the centerlines of the anchor, or in the case of a group of anchors, from a line through a row of adjacent anchors. $A_{NC}$ shall not exceed $nA_{Nco}$, where $n$ is the number of anchors in the group that resist tension.

$\psi_{ec,N} = \text{modification factor for anchor groups loaded by an eccentric tension force}$

$$= \frac{1}{\left(1 + \frac{2e'N}{3h_{ef}}\right)} \leq 1 \quad (17.4.2.4)$$

$\psi_{ed,N} = \text{modification factor for edge effects for single anchors or anchor groups loaded in tension}$

$$= 1 \quad \text{if} \quad c_{a,min} \geq 1.5h_{ef} \quad (17.4.2.5a)$$

$$= 0.7 + 0.3 \frac{c_{a,min}}{1.5h_{ef}} \quad \text{if} \quad c_{a,min} < 1.5h_{ef} \quad (17.4.2.5b)$$

$\psi_{c,N} = \text{Modification factor for concrete conditions (uncracked, cracked, reinforced, etc.). Ref. ACI 318-14 Section 17.4.2.6 for cast-in-place anchors. Ref. ICC-ES Evaluation Service Report for post-installed anchors}$

$\psi_{cp,N} = \text{Modification factor for splitting}$

Ref. ACI 318-14 Section 17.4.2.7 and/or the relevant ICC-ES Evaluation Service Report for post-installed mechanical anchors

$N_b = \text{basic concrete breakout strength of a single anchor in tension in cracked concrete}$

$$= k_c \lambda_a \sqrt{f'_c h_{ef}^{1.5}} \quad (17.4.2.2a)$$

Ref. ACI 318-14 Section 17.4.2.2 for permitted values of the effectiveness factor, $k_c$.

For post-installed anchors that have been tested in accordance with ACI 355.2 or ACI 355.4, specific values of the effectiveness factor (more precisely, $k_{cr}$ for cracked concrete conditions and $k_{uncr}$ for uncracked concrete conditions) are established in accordance with the provisions of that document or the relevant ICC-ES acceptance criteria. Values of $k_{cr}$ and $k_{uncr}$ for Hilti anchor products can be found in the product specific sections of this guide.

The nominal pullout strength of anchors loaded in tension is determined in accordance with ACI 318-14 Section 17.4.3 as follows:

$$N_{pn} = \psi_{c,P} N_p \quad (17.4.3.1)$$

where:

$N_p = \text{for post-installed expansion and undercut anchors, pullout strength based on the 5 percent fractile of results of tests performed and evaluated according to ACI 355.2 or the relevant ICC-ES Acceptance Criteria. It is not permissible to calculate the pullout strength in tension for such anchors}$

$\psi_{c,P} = 1.4 \quad \text{for anchors located in a region of a concrete member where analysis indicates no cracking at service load levels}$

$$= 1.0 \quad \text{where analysis indicates cracking at service load levels}$

Pullout values are based on direct tension testing of anchors in cracks as well as on the results of moving crack tests. Additional pullout values associated with seismic testing may also be provided.
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For headed anchors with deep embedment close to an edge \( (c_{a1} < 0.4h_{ef}) \), side-face blowout may control the design. In most cases, restrictions on the placement of post-installed anchors close to an edge will preclude this failure mode. For further information, see ACI 318-14 Section 17.4.4.

The nominal bond strength in tension of a single adhesive anchor loaded in tension is determined in accordance with ACI 318-14 Section 17.4.5 as follows:

\[
N_a = \frac{A_{Na}}{A_{Nao}} \psi_{ed,Na} \psi_{cp,Na} N_{ba} \quad (17.4.5.1a)
\]

The nominal bond strength of anchor groups is likewise determined as follows:

\[
N_{ag} = \frac{A_{Na}}{A_{Nao}} \psi_{ec,Na} \psi_{ed,Na} \psi_{cp,Na} N_{ba} \quad (17.4.5.1b)
\]

where:

\[
A_{Nao} = \text{projected influence area of a single adhesive anchor with an edge distance equal to or greater than } c_{Na} = (2c_{Na})^2 \quad (17.4.5.1c)
\]

\[
c_{Na} = 10d_a \sqrt{\frac{\tau_{\text{uncr}}}{1100}} \quad (17.4.5.1d)
\]

\[
A_{Na} = \text{projected influence area of a single adhesive anchor or group of adhesive anchors approximated as a rectilinear area that projects outward a distance } c_{Na} \text{ from the centerline of the adhesive anchor, or in the case of a group of adhesive anchors, from a line through a row of adjacent adhesive anchors. } A_{Na} \text{ shall not exceed } nA_{Nao}, \text{ where } n \text{ is the number of adhesive anchors in the group that resist tension loads}
\]

\[
\psi_{ec,Na} = \text{modification factor for anchor groups loaded by an eccentric tension force}
\]

\[
= \frac{1}{1 + \frac{e'}{c_{Na}}} \leq 1.0 \quad (17.4.5.3)
\]

\[
\psi_{ed,Na} = \text{modification factor for edge effects for single adhesive anchors or adhesive anchor groups loaded in tension}
\]

\[
= 1.0 \text{ if } c_{a1} \geq c_{Na} \quad (17.4.5.4a)
\]

\[
= 0.7 + 0.3 \frac{c_{a1}}{c_{Na}} \text{ if } c_{a1} < c_{Na} \quad (17.4.5.4b)
\]

\[
\psi_{cp,Na} = \text{modification factor for splitting. Ref. ACI 318-14 Section 17.4.5.5 and/or the relevant ICC-ES Evaluation Service Report for post-installed adhesive anchors}
\]

\[
N_{ba} = \text{basic bond strength of a single adhesive anchor in tension in cracked concrete}
\]

\[
= \lambda \frac{\tau_{cr}}{n d_a h_{ef}} \quad (17.4.5.2)
\]

Where analysis indicates no cracking at service load levels, it is permitted to use \( \tau_{\text{uncr}} \) in place of \( \tau_{cr} \).

3.1.6.6 Design requirements for shear loading

In accordance with ACI 318-14 Section 17.5.1, the nominal steel strength for headed stud anchors in shear is determined as follows:

\[
V_{sa} = A_{se,y} f_{uta} \quad (17.5.1.2a)
\]

For cast-in headed bolt and hooked bolt anchors and for post-installed anchors where sleeves do not extend through the shear plane:

\[
V_{sa} = 0.6A_{se,y} f_{uta} \quad (17.5.1.2b)
\]

Where \( f_{uta} \leq \min \{1.9 f_y, 125,000 \text{ psi (860 MPa)}\} \)

For other post-installed anchors where sleeves extend through the shear plane, \( V_{sa} \) is based on the results of tests performed and evaluated according to ACI 355.2 or the relevant ICC-ES Acceptance Criteria. Alternatively, Eq. (17.5.1.2b) is permitted to be used.

In accordance with ACI 318-14 Section 17.5.1.3, the nominal shear strength of anchors used with built-up grout pads must be multiplied by a 0.80 factor.

The nominal concrete breakout strength of a single anchor loaded in shear is determined in accordance with ACI 318-14 Section 17.5.2 as follows:

\[
V_{cb} = A_{vc} \psi_{ed,V} \psi_{c,V} N_{cb} \quad (17.5.2.1a)
\]

The concrete breakout strength of anchor groups is likewise determined as follows:

\[
V_{cbg} = A_{vcg} \psi_{ed,V} \psi_{c,V} N_{cbg} \quad (17.5.2.1b)
\]

where:

\[
A_{vc} = \text{projected area for a single anchor in a deep member with a distance from edges equal to or greater than } 1.5c_{a1} \text{ in the direction perpendicular to the shear force. It is permitted to evaluate } A_{vc} \text{ as the base of a half pyramid with a side length parallel to the edge of } 3c_{a1} \text{ and a depth of } 1.5c_{a1}
\]

\[
= 4.5(c_{a1})^2 \quad (17.5.2.1c)
\]
\( A_{Vc} = \) projected area of the failure surface on the side of the concrete member at its edge for a single anchor or a group of anchors.

It is permitted to evaluate \( A_{Vc} \) as the base of a truncated half pyramid projected on the side face of the member where the top of the half pyramid is given by the axis of the anchor row selected as critical. The value of \( c_{a1} \) must be taken as the distance from the edge to this axis. \( A_{Vc} \) must not exceed \( n A_{Vco} \), where \( n \) is the number of anchors in the group.

\( \psi_{ec,V} = \) modification factor for anchor groups loaded eccentrically in shear

\[
\frac{1}{1 + \frac{2e_{V}}{3c_{a1}}} \leq 1.0 \quad (17.5.2.5)
\]

\( \psi_{ed,V} = \) modification factor for edge effect for a single anchor or group of anchors loaded in shear computed using the smaller value of \( c_{a2} \)

\[
1.0 \text{ if } c_{a2} \geq 1.5c_{a1} \quad (17.5.2.6a)
\]

\[
0.7 + 0.3 \frac{c_{a2}}{1.5c_{a1}} \text{ if } c_{a2} < 1.5c_{a1} \quad (17.5.2.6b)
\]

\( \psi_{c,V} = \) modification factor for concrete conditions (uncracked, cracked, reinforced, etc.). Ref. ACI 318-14 Section 17.5.2.7 for permitted values of this factor

\( \psi_{h,V} = \) modification factor for anchor located in a member where \( h_{a} \leq 1.5c_{a1} \) (thin member)

\[
\sqrt{\frac{1.5c_{a2}}{h_{a}}} \quad (17.5.2.8)
\]

\( V_{b} = \) basic concrete breakout strength in shear of a single anchor in cracked concrete, determined as the smaller of (17.5.2.2a) and (17.5.2.2b). Ref. ACI 318-14 Section 17.5.2.2 for limiting values of \( \ell_{e} \):

\[
7 \left( \frac{e_{V}}{d_{a}} \right)^{0.5} \lambda_{h} \sqrt{f'_{c}} (c_{a1})^{1.5} \quad (17.5.2.2a)
\]

\[
9\lambda_{h} \sqrt{f'_{c}} (c_{a1})^{1.5} \quad (17.5.2.2b)
\]

\( \) for cast-in headed studs, headed bolts, or hooked bolt, Ref. ACI 318-14 Section 17.5.2.3

The nominal pryout strength of a single anchor is determined in accordance with ACI 318-14 Section 17.5.3 as follows:

\[
V = k_{cp} N_{cp} \quad (17.5.3.1a)
\]

For cast-in, expansion, and undercut anchors, \( N_{cp} \) must be taken as \( N_{cb} \) (17.4.2.1a). For adhesive anchors, \( N_{cp} \) must be the lesser of \( N_{ad} \) (17.4.5.1a) and \( N_{cb} \) (17.4.2.1a).

Likewise, for anchor groups, the pryout strength is determined as follows:

\[
V_{cpg} = k_{cp} N_{cpg} \quad (17.5.3.1b)
\]

For cast-in, expansion, and undercut anchors, \( N_{cpg} \) must be taken as \( N_{cbg} \) (17.4.2.1b). For adhesive anchors, \( N_{cpg} \) must be the lesser of \( N_{adg} \) (17.4.5.1b) and \( N_{cbg} \) (17.4.2.1b).

In Eq. (17.5.3.1a) and (17.5.3.1b):

\[
k_{cp} = 1.0 \text{ for } h_{a} < 2.5 \text{ in.}
\]

\[
k_{cp} = 2.0 \text{ for } h_{a} \geq 2.5 \text{ in.}
\]

### 3.1.6.7 Interaction – Strength Design

Where anchors are loaded simultaneously in tension and shear, interaction must be considered. In accordance with ACI 318-14 Section 17.6, interaction may be checked as follows:

If \( V_{ua} \leq 0.2 \Phi V_{n} \) \( \Phi N_{n} \geq N_{ua} \)

If \( N_{ua} \leq 0.2 \Phi N_{n} \) \( \Phi V_{n} \geq V_{ua} \)

If \( N_{ua} > 0.2 \Phi N_{n} \)

\[
\left[ \frac{N_{ua}}{\Phi N_{n}} \right] + \left[ \frac{V_{ua}}{\Phi V_{n}} \right] \leq 1.2 \quad (17.6.3)
\]

Alternatively, ACI 318-14 Section 17.6 permits the use of an interaction expression of the form:

\[
\left[ \frac{N_{ua}}{\Phi N_{n}} \right]^{\alpha} + \left[ \frac{V_{ua}}{\Phi V_{n}} \right]^{\alpha} \leq 1.0
\]

Where \( \alpha \) varies from 1 to 2. The current trilinear recommendation is a simplification of the expression where \( \alpha = 5/3 \).
3.1 Anchor Principles and Design

3.1.6.8 Required edge distances, anchor spacing and member thickness – Strength Design

Refer to ACI 318 -14 Section 17.7, ACI 355.2, ACI 355.4, or the relevant ICC-ES acceptance criteria for the geometry requirements for cast-in-place and post-installed anchors.

3.1.6.9 Bolt bending – Strength Design

An additional check for shear load resulting from stand-off conditions can be performed when calculating nominal shear strengths.

\[ V_s^M = \frac{\alpha M \cdot M_s \cdot \ell}{l} \]

whereby:

- \( \alpha_M \) = adjustment of bending moment associated with rotational restraint, where \( 1 \leq \alpha_M \leq 2 \)
- \( M_s \) = resultant flexural resistance of single anchor
  \[ M_s = M_s^o \left( 1 - N_{ua} / N_{sa} \right) \]
- \( M_s^o \) = characteristic flexural resistance of single anchor
  \[ M_s^o = 1.2 \cdot S \cdot f_{u,min} \]
- \( f_{u,min} \) = minimum nominal ultimate tensile strength of anchor element
- \( S \) = elastic section modulus of anchor bolt at concrete surface (a uniform cross section is assumed)
  \[ S = (n \cdot d^2) / 32 \]
- \( \ell \) = internal lever arm adjusted for spalling of the concrete surface as follows:
  \[ \ell = z + (n \cdot d_o) \]
- \( z \) = distance from center of base plate to surface of concrete (standoff distance)
- \( d_o \) = anchor outside diameter at concrete surface
- \( n \) = 0, for loading with clamping at the concrete surface as provided by a nut and washer assembly (required for mechanical anchors)
  \[ n = 0.5, \text{ for loading without clamping at the concrete surface, e.g., adhesive anchor without nut and washer at concrete surface} \]

Note that stand-off installations of post-installed mechanical anchors require a nut and bearing washer at the concrete surface as shown below for proper anchor function and to properly resist compression loads.

Determination of bolt bending – Strength Design

3.1.7 CSA A23.3 Annex D Limit State Design

Limit State Design of anchors is referenced in the provisions of CSA A23.3-14 Annex D, which cover headed studs and bolts, hooked bolts and post-installed anchors that meet the assessment of ACI 355.2 and ACI 355.4. Furthermore, the suitability of post-installed anchors for use in concrete must be demonstrated by the ACI 355.2 and ACI 355.4 prequalification tests. A summary of the relevant design provisions, especially as they pertain to post-installed anchors, is provided here.

3.1.7.1 Load Distribution

The provisions of CSA A23.3-14 Annex D and ACI 318-14 Chapter 17 are based on identical assumptions. Refer to Section 3.1.6.2 for more details.

3.1.7.2 General Requirements for Anchor Strength

In accordance with CSA A23.3-14 Annex D, the design of anchors must satisfy the following conditions:

\[ N_r \geq N_f \]
\[ V_r \geq V_f \]

whereby \( N_r \) and \( V_r \) are the lowest design resistances determined from all applicable failure modes in tension and shear, respectively, and \( N_f \) and \( V_f \) are the factored tension and shear loads resulting from the governing load combination. For this assessment, identical failure modes as described in Section 3.1.6.3 must be considered.

Thus, for a single anchor, the controlling resistance would be determined as follows:

\[ N_r = \min \{|N_{sar}, N_{cbr}, N_{pr}, N_{ar}| \} \]
\[ V_r = \min \{|V_{sar}, V_{cbr}, V_{cpr}| \} \]
In analogy, the controlling resistance for an anchor group would be determined as
\[ N_r = \min | N_{sar}, N_{cbr}, N_{pr}, N_{agr} | \]
\[ V_r = \min | V_{sar}, V_{cbr}, V_{cpr}, V_{cpr} | \]

In accordance with CSA A23.3-14 Clause D.4.6, all requirements for anchor axial tension and shear resistance shall apply to normal-density concrete. When low-density aggregate concrete is used, \( N_r \) and \( V_r \) shall be modified by multiplying all values of \( f'_{c} \) affecting \( N_r \) and \( V_r \) by \( \lambda \). Ref. CSA A23.3-14 Clause D.8.6.5 for more details.

### 3.1.7.3 Strength Reduction Factors

Strength reduction factors are intended to account for possible reductions in resistance due to normally expected variations in material strengths, anchor installation procedures, etc. Relevant strength reduction factors as given in CSA A23.3-14 Clauses 8.4 and D.5.3 are provided below.

Material resistance factor for concrete tensile strength:
\( \phi = 0.65 \)

Material resistance factor for reinforcing bars and embedded steel anchors: \( \phi_s = 0.85 \)

Resistance modification factors, \( R \), as specified in Clauses D.6 and D.7 must be as follows:

**Anchor governed by a ductile steel element:**
- Tension loads: 0.80
- Shear loads: 0.75

**Anchor governed by strength of a brittle steel element:**
- Tension loads: 0.70
- Shear loads: 0.65

**Anchor governed by concrete breakout, blowout, pullout, or pryout strength:**

<table>
<thead>
<tr>
<th>Condition</th>
<th>Anchor 1</th>
<th>Anchor 2</th>
<th>Anchor 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>A.1</td>
<td>1.15</td>
<td>1.00</td>
<td>0.85</td>
</tr>
<tr>
<td>A.2</td>
<td>1.00</td>
<td>0.85</td>
<td>0.70</td>
</tr>
</tbody>
</table>

Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member.

Condition B applies where such supplementary reinforcement is not provided or where pullout or pryout strength governs.

### 3.1.7.4 Design Requirements for Tensile Loading

The factored resistance of an anchor in tension as governed by the steel, \( N_{sar} \), shall be evaluated by calculations based on the properties of the anchor material and the physical dimensions of the anchor. In accordance with CSA A23.3-14 Clause D.6.1, the factored resistance of an anchor or anchor group in tension shall not exceed the following:

\[ N_{sar} = A_{se,N} \phi_s f_{uta} R \]  
(D.2)

where \( f_{uta} \) shall not be greater than the smaller of \( 1.9 f_{ya} \) or 860 MPa.

The factored concrete breakout resistance of an anchor or an anchor group in tension is determined in accordance with CSA A23.3-14 Clause D.6.2.1 as follows

\[ N_{cbr} = A_{se,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_{br} \]  
(D.3)

Likewise, for an anchor group:

\[ N_{cbgr} = A_{se,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_{br} \]  
(D.4)

where:

- \( A_{Nco} = \) projected concrete failure area of a single anchor with an edge distance equal to or greater than \( 1.5 h_{ef} \)
- \( A_{Nc} = \) projected concrete failure area of a single anchor or group of anchors approximated as the base of the rectilinear geometrical figure that results from projecting the failure surface outward \( 1.5 h_{ef} \) from the centerlines of the anchor, or in the case of a group of anchors, from a line through a row of adjacent anchors. \( A_N \) shall not exceed \( n A_{Nco} \), where \( n \) is the number of anchors in the group that resist tension
3.1 Anchor Principles and Design

\[ \psi_{ec,N} = \text{modification factor for anchor groups loaded by an eccentric tension force} \]
\[ = \frac{1}{\left(1 + \frac{2e'N}{3h_{ef}}\right)} \leq 1 \quad (D.8) \]

\[ \psi_{ed,N} = \text{modification factor for edge effects for single anchors or anchor groups loaded in tension} \]
\[ = 1 \text{ if } c_{a,\text{min}} \geq 1.5h_{ef} \quad (D.10) \]
\[ = 0.7 + 0.3 \frac{c_{a,\text{min}}}{1.5h_{ef}} \text{ if } c_{a,\text{min}} < 1.5h_{ef} \quad (D.11) \]

\[ \psi_{c,N} = \text{modification factor for concrete conditions (uncracked, cracked, reinforced, etc.). Ref. CSA A23.3-14 Clause D.6.2.6 and relevant ICC-ES Evaluation Service Report.} \]

\[ \psi_{cp,N} = \text{modification factor for splitting. Ref. CSA A23.3-14 Clause D.6.2.7 and relevant ICC-ES Evaluation Service Report.} \]

\[ N_{br} = \text{factored concrete breakout resistance of a single anchor in tension incracked concrete} \]
\[ = k\phi_a \lambda a \sqrt{f'c} h_{ef}^{1.5} R \quad (D.6) \]

Ref. CSA A23.3-14 Clause D.6.2.2 for permitted values of the k-factor. The k-factor for post-installed anchors may be increased in accordance with ACI 355.2 product-specific tests, but shall not exceed 10 in cracked concrete.

In accordance with CSA A23.3-14 Clause D.6.3, the factored pullout resistance of an anchor in tension is determined as follows:

\[ N_{epr} = \psi_{ec,N} N_{pr} \quad (D.15) \]

where:

\[ N_{pr} = \text{for post-installed expansion and undercut anchors, the pullout strength shall not be calculated in tension. Values of } N_{pr} \text{ shall be based on the 5% fractile of the results of tests performed and evaluated in accordance with ACI 355.2.} \]

For deep headed anchors placed close to an edge (c < 0.4h_{ef}), side-face blowout may control the design. In most cases, restrictions on the placement of post-installed anchors close to an edge will preclude this mode of failure. For further information, see CSA A23.3-14 Clause D.6.4

The factored bond resistance in tension of a single adhesive anchor determined in accordance with CSA A23.3-14 Clause D.6.5 as follows:

\[ N_{af} = \frac{A_{na}}{A_{Nao}} \psi_{ec,Na} \psi_{cp,Na} N_{baf} \quad (D.20) \]

The nominal concrete breakout strength of anchor groups is likewise determined as follows:

\[ N_{agr} = \frac{A_{na}}{A_{Nao}} \psi_{ec,Na} \psi_{cp,Na} N_{baf} \quad (D.21) \]

where:

\[ A_{Nao} = \text{projected influence area of a single adhesive anchor with an edge distance equal to or greater than } c_{Na} \]
\[ = (2c_{Na})^2 \quad (D.22) \]
\[ c_{Na} = 10d_a \sqrt{\frac{\tau_{uncr}}{7.60}} \quad (D.23) \]
\[ A_{Na} = \text{projected influence area of a single adhesive anchor or group of adhesive anchors approximated as a rectilinear area that projects outward a distance } c_{Na} \text{ from the centerline of the adhesive anchor, or in the case of a group of adhesive anchors, from a line through a row of adjacent adhesive anchors. } A_{Na} \text{ shall not exceed } n A_{Nao}, \text{ where } n \text{ is the number of adhesive anchors in the group that resist tension loads} \]
\[ \psi_{ec,Na} = \text{modification factor for anchor groups loaded by an eccentric tension force} \]
\[ = \frac{1}{\left(1 + \frac{e'N}{c_{Na}}\right)} \leq 1 \quad (D.25) \]
\[ \psi_{ed,Na} = \text{modification factor for edge effects for single adhesive anchors or adhesive anchor groups loaded in tension} \]
\[ = 1.0 \text{ if } c_{a,\text{min}} \geq c_{Na} \quad (D.26) \]
\[ = 0.7 + 0.3 \frac{c_{a,\text{min}}}{c_{Na}} \text{ if } c_{a,\text{min}} < c_{Na} \quad (D.27) \]
\[ \psi_{cp,Na} = \text{modification factor for splitting. Ref. CSA A23.3-14 Clause D.6.5 and/or the relevant ICC-ES Evaluation Service Report for postinstalled adhesive anchors} \]
\[ N_{na} = \text{basic bond strength of a single adhesive anchor in tension in cracked concrete} \]
\[ = \lambda a \phi a \tau_{cr} d_a h_{ef} R \quad (D.24) \]

Where analysis indicates no cracking at service load levels, it is permitted to use \( \tau_{uncr} \) in place of \( \tau_{cr} \).
3.1.7.5 Design Requirements for Shear Loading

The factored resistance of an anchor in shear as governed by steel, \( V_{sar} \), shall be evaluated by calculations based on the properties of the anchor material and the physical dimensions of the anchor. In accordance with CSA A23.3-14 Clause D.7.1, the factored resistance of an anchor in shear shall not exceed the following:

\[
V_{sar} < A_{se,V} \phi_s f_{uta} R
\]  
(D.30)

For cast-in headed bolt and hooked bolt anchors and post-installed anchors without sleeves extending through the shear plane:

\[
V_{sar} < A_{se,V} \phi_s 0.6 f_{uta} R
\]  
(D.31)

where \( f_{uta} \) shall not be greater than the smaller of 1.9\( f_{ya} \) or 860 MPa.

For post-installed anchors with sleeves extending through the shear plane, \( V_{sar} \) shall be based on the 5% fractile of results of tests performed and evaluated in accordance with ACI 355.2.

As per CSA A23.3-14 Clause D.7.1.3, where anchors are used with built-up grout pads, the factored resistances shown above can be reduced by 20%.

The factored concrete breakout resistance of a single anchor loaded in shear is determined in accordance with CSA A23.3-14 Clause D.7.2.1 as follows:

\[
V_{cbr} = \frac{A_{Vc}}{A_{Vco}} \psi_{c,V} \psi_{ed,V} \psi_{h,V} V_{br}
\]  
(D.32)

The concrete breakout resistance of anchor groups is likewise determined as follows:

\[
V_{cbgr} = \frac{A_{Vc}}{A_{Vco}} \psi_{c,V} \psi_{ed,V} \psi_{h,V} V_{br}
\]  
(D.33)

Where:

\[ A_{Vco} \] = projected area for a single anchor in a deep member with a distance from edges equal to or greater than 1.5\( c_{a1} \) in the direction perpendicular to the shear force. It is permitted to evaluate \( A_{Vco} \) as the base of a half pyramid with a side length parallel to the edge of 3\( c_{a1} \) and a depth of 1.5\( c_{a1} \).

\[ A_{Vc} \] = projected area of the failure surface on the side of the concrete member at its edge for a single anchor or a group of anchors. It is permitted to evaluate \( A_{Vc} \) as the base of a truncated half pyramid projected on the side face of the member where the top of the half pyramid is given by the axis of the anchor row selected as critical. The value of \( c_{a1} \) must be taken as the distance from the edge to this axis. \( A_{Vc} \) must not exceed \( nA_{Vco} \) where \( n \) is the number of anchors in the group.

\[ \psi_{c,V} \] = modification factor for concrete conditions (uncracked, cracked, reinforced, etc.). Ref. CSA A23.3-04 Clause D.7.2.7 for permitted values of this factor.

\[ \psi_{ed,V} \] = modification factor for edge effect for a single anchor or group of anchors loaded in shear computed using the smaller value of \( c_{a2} \).

\[ \psi_{h,V} \] = modification factor for shear strength of anchors located in concrete members with \( h < 1.5c_{a1} \)

\[ \psi_{h,V} = \frac{1.5c_{a1}}{h} \]  
(D.42)

\[ V_{br} = \frac{k_{cp}N_{cbr}}{2 \ell d_{o} f_{c} f_{y,a} R} \]  
(D.35)

where \( k_{cp} = 1.0 \) for \( h_{a1} < 65 \text{ mm} \)

\[ k_{cp} = 2.0 \] for \( h_{a1} \geq 65 \text{ mm} \)

Likewise, for anchor groups, the factored pryout resistance is determined as follows:

\[
V_{cpr} = k_{cp}N_{cbgr}
\]  
(D.45)

The factored pryout resistance of a single anchor is determined in accordance with CSA A23.3-14 Clause D.7.3 as follows:

\[
V_{cpr} = k_{cp}N_{cbr}
\]  
(D.44)
3.1 Anchor Principles and Design

3.1.7.6  Interaction – Limit States Design

The provisions of CSA A23.3-14 Annex D and ACI 318-14 Chapter 17 are based on identical assumptions. Refer to Section 3.1.6.7 for more details.

3.1.8 Hilti Simplified Design Tables

The Hilti Simplified Design Tables is not a new “method” of designing an anchor that is different than the provisions of ACI 318-14 Chapter 17 or CSA A23.3 Annex D. Rather, it is a series of pre-calculated tables and reduction factors meant to help the designer create a quick calculation of the capacity of the Hilti anchor system, and still be compliant with the codes and criteria of ACI and CSA.

The Hilti Simplified Design Tables are formatted similar to the Allowable Stress Design (ASD) tables and reduction factors which was a standard of practice for design of post-installed anchors.

The Hilti Simplified Design Tables combine the simplicity of performing a calculation according to the ASD method with the code-required testing, evaluation criteria and technical data in ACI 318-14 Chapter 17 and CSA Annex D.

3.1.8.1 Simplified Tables Data Development

The Simplified Tables have two table types. The single anchor capacity table and the reduction factor table.

Single anchor capacity tables show the design strength (for ACI) or factored resistance (for CSA) in tension and shear for a single anchor. This is the capacity of a single anchor with no edge distance or concrete thickness influences and is based on the assumptions outlined in the footnotes below each table.

Reduction factor tables are created by comparing the single anchor capacity to the capacity that includes the influence of a specific edge distance, spacing, or concrete thickness, using the equations of ACI 318-14 Chapter 17.

3.1.8.2 Hilti Mechanical Anchors or Hilti HIT-Z(-R) Anchor Rods

The single anchor tension capacity is based on the lesser of concrete breakout strength or pullout strength:

\[
\begin{align*}
\text{ACI/AC308: } & \quad \Phi N_n = \min | \Phi N_{cp} : \Phi N_{cr} | \\
\text{CSA: } & \quad N_n = \min | N_{cr} : N_{cp} | \\
& \quad \Phi N_n = N_n
\end{align*}
\]

Concrete breakout and pryout are calculated according to ACI 318-14 Chapter 17 and CSA A23.3 Annex D using the variables from product specific ICC-ES Evaluation Service Reports (ESR’s). These values are equivalent.

Pullout for torque controlled adhesive anchors is not recognized in ACI or CSA, so this is determined from AC308 Section 3.3 and the value of \( N_{p,uncr} \) or \( N_{p,cr} \) from ESR-3187. This is a similar approach to mechanical anchor pullout strength. ACI and CSA values are equivalent.

3.1.8.3 Hilti Adhesive Anchors with Standard Threaded Rods, Rebar, and Hilti HIS-(R)N Internally Threaded Inserts

The single anchor tension capacity is based on the lesser of concrete breakout strength or bond strength:

\[
\begin{align*}
\text{ACI: } & \quad \Phi N_n = \min | \Phi N_{cb} : \Phi N_{a} | \\
\text{CSA/ACI: } & \quad N_n = \min | N_{cb} : N_{a} | \\
& \quad \Phi N_n = N_n
\end{align*}
\]

Concrete breakout, bond, and pryout are calculated according to ACI 318-14 Chapter 17 and CSA A23.3 Annex D using the variables from product specific ICC-ES Evaluation Service Reports (ESR’s). These values are equivalent, however, the values will be calculated based on standard concrete compressive strengths specified in the US or Canada.

3.1.8.4 Steel Strength for All Elements

The steel strength is provided on a separate table and is based on calculations from ACI 318-14 Chapter 17 and CSA A23.3 Annex D. ACI and CSA have different reduction factors for steel strength, thus the values for both ACI and CSA are published.
3.1.8.5 How to Calculate Anchor Capacity Using Simplified Tables

The process for calculating the capacity of a single anchor or anchor group is similar to the ASD calculation process currently outlined in section 3.1.9 of this document.

The design strength (factored resistance) of an anchor is obtained as follows:

**Tension:**

**ACI:**
\[
N_{\text{des}} = n \cdot \min \{ \Phi N_\text{n} \cdot f_{AN} \cdot f_{RN} ; \Phi N_{\text{sa}} \}
\]

**CSA:**
\[
N_{\text{des}} = n \cdot \min \{ N_r \cdot f_{AN} \cdot f_{RN} ; N_{sr} \}
\]

**Shear:**

**ACI:**
\[
V_{\text{des}} = n \cdot \min \{ \Phi V_\text{n} \cdot f_{AV} \cdot f_{RV} \cdot f_{HV} ; \Phi V_{\text{sa}} \}
\]

**CSA:**
\[
V_{\text{des}} = n \cdot \min \{ V_r \cdot f_{AV} \cdot f_{RV} \cdot f_{HV} ; V_{sr} \}
\]

where:

- **n** = number of anchors
- **N_{\text{des}}** = design resistance in tension
- **\Phi N_\text{n}** = design strength in tension considering concrete breakout, pullout, or bond failure (ACI)
- **\Phi N_{\text{sa}}** = design strength in tension considering steel failure (ACI)
- **N_r** = factored resistance in tension considering concrete breakout, pullout, or bond failure (CSA)
- **N_{sr}** = factored resistance in tension considering steel failure (CSA)
- **V_{\text{des}}** = design resistance in shear
- **\Phi V_\text{n}** = design strength in shear considering concrete failure (ACI)
- **\Phi V_{\text{sa}}** = design strength in shear considering steel failure (ACI)
- **V_r** = factored resistance in shear considering concrete failure (CSA)
- **V_{sr}** = factored resistance in shear considering steel failure (CSA)
- **f_{AN}** = adjustment factor for spacing in tension
- **f_{RN}** = adjustment factor for edge distance in tension
- **f_{AV}** = adjustment factor for spacing in shear
- **f_{RV}** = adjustment factor for edge distance in shear
- **f_{HV}** = adjustment factor for concrete thickness in shear (this is a new factor that ASD did not use previously)

Adjustment factors are applied for all applicable near edge and spacing conditions.

For example, the capacity in tension corresponding to the anchor group based on worst case anchor “a” in the figure below is evaluated as follows:

**ACI:**
\[
N_{\text{des}} = 4 \cdot \Phi N_\text{n} \cdot f_{A,x} \cdot f_{A,y} \cdot f_{R,x} \cdot f_{R,y}
\]

**CSA:**
\[
N_{\text{des}} = 4 \cdot N_r \cdot f_{A,x} \cdot f_{A,y} \cdot f_{R,x} \cdot f_{R,y}
\]

Note: designs are for orthogonal anchor bolt patterns and no reduction factor for the diagonally located adjacent anchor is required.

Where anchors are loaded simultaneously in tension and shear, interaction must be considered. The interaction equation is as follows:

**ACI:**
\[
\frac{N_{\text{ua}}}{N_{\text{des}}} + \frac{V_{\text{ua}}}{V_{\text{des}}} \leq 1.2
\]

**CSA:**
\[
\frac{N_{\text{f}}}{N_{\text{des}}} + \frac{V_{\text{f}}}{V_{\text{des}}} \leq 1.2
\]

where:

- **N_{\text{ua}}** = Required strength in tension based on factored load combinations of ACI 318-14 Chapter 5.
- **V_{\text{ua}}** = Required strength in shear based on factored load combinations of ACI 318-14 Chapter 5.
- **N_{\text{f}}** = Required strength in tension based on factored load combinations of CSA A23.3 Chapter 8.
- **V_{\text{f}}** = Required strength in shear based on factored load combinations of CSA A23.3 Chapter 8.
3.1 Anchor Principles and Design

The full tension strength can be permitted if:

\[ \frac{V_{ua}}{V_{des}} \leq 0.2 \]

ACI: \( V_{ua} \) ≤ 0.2

CSA: \( V_{f} \) ≤ 0.2

The full shear strength can be permitted if:

\[ \frac{N_{ua}}{N_{des}} \leq 0.2 \]

ACI: \( N_{ua} \) ≤ 0.2

CSA: \( N_{f} \) ≤ 0.2

3.1.8.6 Allowable Stress Design (ASD)

The values of \( N_{des} \) and \( V_{des} \) developed from Section 3.1.8.5 are design strengths (factored resistances) and are to be compared to the required strength in tension and shear from factored load combinations of ACI 318-14 Chapter 5 or CSA A23.3 Chapter 8.

The design strength (factored resistance) can be converted to an ASD value as follows:

\[ N_{des,ASD} = \frac{N_{des}}{\alpha_{ASD}} \]

\[ V_{des,ASD} = \frac{V_{des}}{\alpha_{ASD}} \]

where:

\[ \alpha_{ASD} = \text{Conversion factor calculated as a weighted average of the load factors for the controlling load combination.} \]

An example for the calculation of \( \alpha_{ASD} \) for ACI is as follows:

Strength design with controlling load combination:

1.2D + 1.6L < \( \phi N_{n} \)

Allowable stress design (ASD):

1.0D + 1.0L < \( \phi N_{n} / \alpha_{ASD} \)

Therefore, for an equivalent level of safety:

\[ \alpha_{ASD} = \frac{(1.2D + 1.6L)}{(1.0D + 1.0L)} \]

If the dead load contribution is 40% and live load contribution is 60%, you will get:

\[ \alpha_{ASD} = \frac{(1.2 \times 0.4 + 1.6 \times 0.6)}{(1.0 \times 0.4 + 1.0 \times 0.6)} \]

\[ \alpha_{ASD} = 1.44 \]

3.1.8.7 Seismic Design

To determine the seismic design strength (factored resistance) a reduction factor, \( \alpha_{seis} \), is applied to the applicable table values. This value of \( \alpha_{seis} \) will be in the footnotes of the relevant design tables.

The value of \( \alpha_{N,seis} \) for tension is based on 0.75 times a reduction factor determined from testing. The total reduction is footnoted in the tables.

The value of \( \alpha_{V,seis} \) for steel failure is based on testing and is typically only applied for shear. There is no additional 0.75 factor. The reduction is footnoted in the tables.

The factored load and associated seismic load combinations that will be compared to the design strength (factored resistance) can be determined from ACI or CSA provisions and national or local code requirements. An additional value for \( \phi_{non-ductile} \) may be needed based on failure mode or ductility of the attached components.

3.1.8.8 Sustained Loads and Overhead Use

Sustained loading is calculated by multiplying the value of \( \phi N_{n} \) or \( N_{b} \) by 0.55 and comparing the value to the tension dead load contribution (and any sustained live loads or other loads) of the factored load. Edge, spacing, and concrete thickness influences do not need to be accounted for when evaluating sustained loads.

3.1.8.9 Accuracy of the Simplified Tables

Calculations using the Simplified Tables have the potential of providing a design strength (factored resistance) that is exactly what would be calculated using equations from ACI 318-14 Chapter 17 or CSA A23.3 Annex D.

The tables for the single anchor design strength (factored resistance) for concrete / bond / pullout failure or steel failure have the same values that will be computed using the provisions of ACI and CSA.

The load adjustment factors for edge distance influences are based on a single anchor near an edge. The load adjustment factors for spacing are determined from the influence of two adjacent anchors. Each reduction factor is calculated for the minimum value of either concrete or bond failure. When more than one edge distance and/or spacing condition exists, the load adjustment factors are multiplied together. This will result in a conservative design when compared to a full calculation based on ACI or CSA. Additionally, if the failure mode in the
single anchor tables is controlled by concrete failure, and the reduction factor is controlled by bond failure, this will also give a conservative value (and vice versa).

The following is a general summary of the accuracy of the simplified tables:

- Single anchor tables have values equivalent to a calculation according to ACI or CSA.
- Since the table values, including load adjustment factors, are calculated using equations that are not linear, linear interpolation is not permitted. Use the smaller of the two table values listed. This provides a conservative value if the application falls between concrete compressive strengths, embedment depths, or spacing, edge distance, and concrete thickness.
- For one anchor near one edge, applying the edge distance factor typically provides accurate values provided the failure mode of the table values is the same. If the failure mode is not the same, the values are conservative.
- For two to four anchors in tension with no edge reductions, applying the spacing factors provides a value that is equivalent to the ACI and CSA calculated values, provided the controlling failure modes of the table values are the same. If the failure mode is not the same, the values are conservative.
- The spacing factor in shear is conservative when compared to two anchors with no edge distance considerations. This factor is based on spacing near an edge and can be conservative for installations away from the edge of the concrete member. Note: for less conservative results, it is possible to use the spacing factor in tension for this application if there is no edge distance to consider.
- The concrete thickness factor in shear is conservative when compared to an anchor with no edge influences. This factor is based on applications near an edge. In the middle of a concrete member this is conservative. Note: for less conservative results, this factor can be ignored if the application is not near an edge.

IMPORTANT NOTE:

For applications such as a four bolt or six bolt anchor pattern in a corner in a thin slab, the calculation can be up to 80% conservative when compared to a calculation according to ACI or CSA, and when using the Hilti PROFIS Anchor Design Software. It is always suggested to use the Hilti PROFIS Anchor Design Software or perform a calculation by hand using the provisions of ACI and CSA to optimize the design. This is especially true when the Simplified Table calculation does not provide a value that satisfies the design requirements. The fact that a Simplified Table calculation does not exceed a design load does not mean the Hilti anchor system will not fulfill the design requirements. Additional assistance can be given by your local Hilti representative.

3.1.8.10 Limitations Using Simplified Tables

There are additional limitations that the Simplified Tables do not consider:

- Load Combinations: Table values are meant to be used with the load combinations of ACI 318-14 Section 5.3 and CSA A23.3 Chapter 8. Other load combinations from other code sections are not considered.
- Supplementary Reinforcement: Table values, including reduction factors, are based on Condition B which does not consider the effects of supplementary reinforcement, nor is there an influence factor that can be applied to account for supplementary reinforcement.
- Eccentric loading: Currently, there is not a method for applying a factor to the tables to account for eccentric loading.
- Moments or Torsion: While a designer can apply a moment or torsion to the anchor system and obtain a specific load per anchor, the tables themselves do not have specific factors to account for moments or torsion applied to the anchor system.
- Standoff: Standoff is not considered in the steel design tables.
- Anchor layout: The Simplified Tables assume an orthogonal layout.

As stated above, while the Simplified Tables are limited in application, the designer can use the Hilti PROFIS Anchor Design Software which does account for the conditions noted above.

There may be additional applications not noted above. Contact Hilti with any questions for specific applications.
3.1 Anchor Principles and Design

3.1.9 Allowable Stress Design (ASD)

3.1.9.1 Allowable Stress Design (ASD) terminology

- $A_{nom}$ = nominal bolt cross sectional area, in.$^2$ (mm$^2$)
- $A_{sl}$ = cross sectional area of anchor sleeve, in.$^2$ (mm$^2$)
- $A_{st}$ = tensile stress area of threaded part, in.$^2$ (mm$^2$)
- $c$ = distance from anchor centerline to the closest free edge of base material, in. (mm)
- $c_{cr}$ = critical edge distance, in. (mm)
- $c_{min}$ = minimum edge distance, in. (mm)
- $d$ = anchor bolt diameter (shank diameter), in. (mm)
- $d_{bit}$ = nominal drill bit diameter, in. (mm)
- $d_{h}$ = diameter of clearance hole in attachment (e.g. baseplate), in. (mm)
- $d_{nom}$ = nominal anchor diameter, in. (mm)
- $d_{o}$ = anchor outside diameter (O.D.), in. (mm)
- $d_{w}$ = washer diameter, in. (mm)
- $f_A$ = adjustment factor for anchor spacing
- $f_c$ = concrete compressive strength as measured by testing of cylinders, psi (MPa)
- $f'_c$ = specified concrete compressive strength, psi (MPa)
- $f_{RN}$ = adjustment factor for edge distance, tension loading
- $f_{RV1}$ = adjustment factor for edge distance, shear loading perpendicular and towards free edge
- $f_{RV2}$ = adjustment factor for edge distance, shear loading parallel to free edge
- $f_{RV3}$ = adjustment factor for edge distance, shear loading perpendicular and away from free edge
- $f_y$ = specified reinforcing bar yield strength, psi (MPa)
- $F_y$ = specified bolt minimum yield strength, psi (MPa)
- $F_u$ = specified bolt minimum ultimate strength, psi (MPa)
- $h$ = thickness of member in which anchor is embedded as measured parallel to anchor axis, in. (mm)
- $h_{ef}$ = effective anchor embedment depth, in. (mm)
- $h_{min}$ = minimum member thickness, in. (mm)
- $h_{nom}$ = distance between base material surface and bottom of anchor (prior to setting is applicable), in. (mm)
- $h_o$ = depth of full diameter hole in base material, in. (mm)
- $l$ = anchor embedded length, in. (mm)
- $l_{th}$ = anchor usable thread length, in. (mm)
- $M_{ult,5\%}$ = characteristic flexural resistance of anchor bolt (5% fractile), in-lb (N·m)
- $N_{allow}$ = allowable tension load, lb (kN)
- $N_d$ = design tension load (unfactored), lb (kN)
- $N_{rec}$ = recommended tension load, lb (kN)
- $s$ = anchor axial spacing, in. (mm)
- $s_{cr}$ = critical spacing between adjacent loaded anchors, in. (mm)
- $s_{min}$ = minimum spacing between adjacent loaded anchors, in. (mm)
- $s_W$ = width of anchor nut across flats, in.$^2$ (mm$^3$)
- $S$ = elastic section modulus of anchor bolt, in.$^3$ (mm$^3$)
- $t_{fix}$ = maximum thickness of attachment (e.g. baseplate) to be fastened, in. (mm)
- $T_{inst}$ = recommended anchor installation torque, ft-lb (N·m)
- $T_{max}$ = maximum tightening torque, ft-lb (N·m)
- $V_{allow}$ = allowable shear load (based on mean value from tests and a global safety factor), lb (kN)
- $V_d$ = design shear load (unfactored), lb (kN)
- $V_{rec}$ = recommended shear load, lb (kN)

3.1.9.2 General requirements and recommended loads

In accordance with the general ASD principles, the design of anchors must satisfy the following conditions:

$N_{service} \leq N_{rec}$
$V_{service} \leq V_{rec}$
whereby \( N_{\text{service}} \) and \( V_{\text{service}} \) are the service tension and shear loads resulting from the governing load combinations (i.e. ASCE 7-10) and \( N_{\text{rec}} \) and \( V_{\text{rec}} \) are the recommended allowable loads for an anchor or a group of anchors.

The ASD method is currently referenced in masonry-related ICC-ES AC01, AC58, AC60, and AC106.

The recommended allowable loads for an anchor or a group of anchors are obtained as follows:

\[
\begin{align*}
N_{\text{rec}} &= N_{\text{allow}} \cdot f_{RN} \cdot f_A \\
V_{\text{rec}} &= V_{\text{allow}} \cdot f_{RV} \cdot f_A
\end{align*}
\]

where:

- \( N_{\text{rec}} \) = recommended tension load
- \( N_{\text{allow}} \) = allowable load (based on the mean value from laboratory testing to failure and a global safety factor)
- \( V_{\text{rec}} \) = recommended shear load
- \( V_{\text{allow}} \) = allowable shear load
- \( f_A \) = adjustment factor for anchor spacing
- \( f_{RN} \) = adjustment factor for edge distance, tension loading
- \( f_{RV1} \) = adjustment factor for edge distance, shear loading perpendicular and toward free edge
- \( f_{RV2} \) = adjustment factor for edge distance, shear loading parallel to free edge
- \( f_{RV3} \) = adjustment factor for edge distance, shear loading perpendicular and away from free edge

Adjustment factors are multiplicative and are applied for all edge and spacing conditions that are less than \( s_{\text{cr}} \) and \( c_{\text{cr}} \), respectively.

For example, the recommended tension load corresponding to anchor “a” in the figure below is evaluated as follows:

\[
F_{\text{rec,a}} = F_{\text{allow,a}} \cdot f_{Rx} \cdot f_{Ry} \cdot f_{Ax} \cdot f_{Ay}
\]

Note that no reduction factor for the diagonally located adjacent anchor is required.

### 3.1.9.3 Critical and minimum spacing and edge distance

Spacing adjustment factors are applicable for cases where the anchor spacing is such that:

\[
s_{\text{min}} \leq s < s_{\text{cr}}
\]

where:

- \( s_{\text{min}} \) = minimum spacing between loaded anchors; and
- \( s_{\text{cr}} \) = critical spacing between loaded anchors (anchor spacing equal to or greater than the one requiring a reduction factor)

Similarly, for near-edge anchors, the edge distance adjustment factor(s) are applicable for cases where the anchor edge distance is such that:

\[
c_{\text{min}} \leq c < c_{\text{cr}}
\]

where:

- \( c_{\text{min}} \) = minimum edge distance; and
- \( c_{\text{cr}} \) = critical edge distance (anchor edge distance equal to or greater than the one requiring a reduction factor)

### 3.1.9.4 Interaction - ASD

Where anchors are loaded simultaneously in tension and shear, interaction must be considered. The usual form of the interaction equation for anchors is as follows:

\[
V_{\text{rec}} = \left[ \frac{N_d}{N_{\text{rec}}} \right]^{\alpha} + \left[ \frac{V_d}{V_{\text{rec}}} \right]^{\alpha} \leq 1.0
\]

where:

- \( N_d \) = design tension load (ASD);
- \( V_d \) = design shear load (ASD); and
- \( \alpha \) = exponent, \( 1 \leq \alpha \leq 2 \)

The value used for \( \alpha \) corresponds to the type of interaction equation being considered. A value of \( \alpha = 1.0 \) corresponds to a straight line interaction equation, while a value of \( \alpha = 5/3 \) corresponds to a parabolic interaction equation.
3.1 Anchor Principles and Design

3.1.9.5 Shear load with lever arm (bolt bending) - ASD

When shear load is applied to a stand-off connection, the anchor bolt is subjected to combined shear and bending, and a separate assessment of the standoff condition is appropriate. In the absence of other guidance, the recommended shear load associated with bolt bending for anchors subjected to shear loads applied at a standoff distance $z$ may be evaluated as follows:

$$V_{rec} = \frac{\alpha_M \cdot M_{um,5\%}}{1.7 \cdot \ell}$$

where:

- $\alpha_M$ = adjustment of bending moment associated with rotational restraint, where $1 \leq \alpha_M \leq 2$
- $V_{rec}$ = recommended shear load corresponding to bending
- $M_{um,5\%}$ = characteristic flexural resistance of a single anchor
- $V_{rec} = \left(1 - \frac{N_d}{N_{rec}}\right)$
- $f_{uta}$ = minimum ultimate tensile strength of anchor
- $S$ = elastic section modulus of anchor bolt at concrete surface (a uniform cross section is assumed)
- $\ell$ = internal lever arm adjusted for spalling of the surface concrete as follows:
  - $\ell = z + (n \cdot d_o)$
- $z$ = distance from center of base plate to surface of concrete (standoff distance)
- $d_o$ = anchor outside diameter at concrete surface
- $n = 0$, for static loading with clamping at the concrete surface as provided by a nut and washer assembly (required for mechanical anchors);
  - $n = 0.5$, for static loading without clamping at the concrete surface, e.g., adhesive anchor without nut and washer at concrete surface

Determination of bolt bending – ASD

Note that stand-off installations of post-installed mechanical anchors require a nut and bearing washer at the concrete surface as shown above for proper anchor function.

3.1.9.6 Increase in capacity for short-term loading – ASD

Some building codes allow a capacity (stress) increase of 1/3 when designing for short-term loading such as wind or seismic. The origin of the 1/3 increase is unclear as it relates to anchor design, but it is generally assumed to address two separate issues: 1) strain-rate effects, whereby the resistance of some materials is increased for transitory stress peaks, and 2) the lower probability of permanent and transitory loads occurring simultaneously.

While Hilti does not include the 1/3 increase in published capacities for anchors in concrete, it is the responsibility of the designer to determine the appropriateness of such a capacity increase under the applicable code.
3.1.10 Design examples

See http://www.us.hilti.com for more design examples.

**Strength Design example, mechanical anchors, KWIK Bolt TZ**

**Objective:**
Determine the controlling design strength in tension and shear.

Check the controlling design strength in tension and shear against the factored service loads in tension and shear.

**Dimensional parameters:**
- $d_0 = \frac{5}{8} \text{ in.}$
- $h_{nom} = 4.75 \text{ in.}$
- $h_{ef} = 4 \text{ in.}$
- $t_{fixture} = 1/2 \text{ in.}$
- $h = 12 \text{ in.}$
- $s_x = 4 \text{ in.}$
- $s_y = 4 \text{ in.}$
- $c_{rx} = 6 \text{ in.}$
- $c_{ry} = 8 \text{ in.}$

**Given:**
- Normal weight concrete, $f'c = 4,000 \text{ psi}$; cracked concrete conditions assumed; seismic design category (SDC) C
- Reference ACI 318-14, Ch. 5 and ICC-ES ESR-1917 for LRFD Factors
- Carbon steel 5/8" x 6" KWIK Bolt TZ anchors; anchors are considered ductile steel elements

**Things to check:**
- Geometry requirements
- Tension design strengths
- Shear design strengths
- Tension/shear interaction

**References:**
- Mechanical anchor – KWIK Bolt TZ
- ACI 318-14 Chapter 17
- ICC-ES ESR-1917

**Calculation per ACI 318-14 Chapter 17, ICC-ES ESR-1917, KWIK Bolt TZ**

Check minimum anchor spacing, edge distance, concrete member thickness

**Notes on tension parameters:**
- $3h_y = (3)(4 \text{ in.}) = 12 \text{ in.}$
- $s = 4 \text{ in.} < 12 \text{ in.} \rightarrow \text{consider group action}$
- $1.5h_y = (1.5)(4 \text{ in.}) = 6 \text{ in.}$
- $c = c_{min} = 6 \text{ in.} \rightarrow \text{no edge influence in tension}$
### 3.1 Anchor Principles and Design

#### 3.1.10 Design examples

<table>
<thead>
<tr>
<th>Calculation per ACI 318-14 Chapter 17, ICC-ES ESR-1917, KWIK Bolt TZ</th>
<th>ACI 318 ref.</th>
<th>ESR ref.</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Notes on tension parameters:</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Check shear parallel to x+ edge → $c_{x1} = 6$ in.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$3c_{x1} = (3)(6)$ in. = 18 in.</td>
<td>17.5.2.1</td>
<td></td>
</tr>
<tr>
<td>$s_y = 4$ in. &lt; 18 in. → consider group action</td>
<td>R17.5.2.1</td>
<td></td>
</tr>
<tr>
<td>Check shear parallel to y+ edge → $c_{y1} = 8$ in.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$1.5c_{y1} = (1.5)(6)$ in. = 9 in.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$8$ in. &lt; 9 in. → consider edge influence</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Minimum base material thickness = 6 in. Actual base material thickness (h) = 12 in. 6 in. < 12 in. → OK

**Notes on tension parameters:**

$h_u = 4$ in. for a 5/8" KWIK Bolt TZ having $h_{nom} = 4.75$ in.

$h_{nom} = 4.75$ in.

Anchor length ($l_{anch}$) = 6" for a 5/8" x 6" KWIK Bolt TZ.

Fixure thickness ($t_{fixture}$) = 1/2 in.

Assume the nut/washer thickness = 3/4 in.

Actual thread length = 2.75 in.

Available thread length = $l_{anch} - h_{ef} = 6$ in. - 4 in. = 2 in.

$t_{fixture} +$ nut/washer thickness = 1/2 in. + 3/4 in. = 1.25 in. 2 in. > 1.25 in. OK

Calculate nominal steel strength in tension: $N_s = 17,170$ lb/anchor

4-anchors in tension. Highest tension load acting on a single anchor = $N_{sa} / 4 = 3,000$ lb / 4-anchors = 750 lb / anchor

Steel strength: $N_s = 17,170$ lb/anchor

**Calculate nominal concrete breakout strength in tension:**

$N_{cbg} = 17.4.2.1 (b)$  Eq. (17.4.2.1b) Section 4.1.3

<table>
<thead>
<tr>
<th>$c_{x} = \infty$</th>
<th>$s_{x} = 4$ in.</th>
<th>$c_{y} = 6$ in.</th>
<th>$c_{y} = 8$ in.</th>
<th>$s_{y} = 4$ in.</th>
<th>$c_{x} = \infty$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$c_{max} = 1.5 \cdot h_{ef} = (1.5)(4)$ in. = 6 in.</td>
<td>if $c \geq 6$ in. → use $l_{anch}$</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$s_{max} = 3 \cdot h_u = (3)(4)$ in. = 12 in.</td>
<td>if $s &gt; 12$ in. → no group action</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$A_{nc} = (c_{x} + s_{x} + c_{y}) (s_{y} + s_{y} + c_{y})$ = (6 in. + 4 in. + 6 in.) (6 in. + 4 in. + 6 in.) = 256 in²</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$A_{nc0} = (9) (hef)^{2}$ = (9)(4 in.)^{2} = 144 in²</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

No tension eccentricity → $\psi_{ec,N} = 1.0$

The smallest edge distance ($c_{ref}$) = 6 in. = $1.5h_{ef}$ → no reduction for edge distance $\psi_{ec,N} = 1.0$ 17.4.2.4 Eq. (17.4.2.4)

Note: cracked concrete conditions have been assumed $\psi_{c,N} = 1.0$ $\psi_{cp,N} = 1.0$

Note: normal weight concrete → $\lambda_a = 1.0$. $N_b = k_{c,cr} \lambda_a \sqrt[5]{f_c} (h_{ef})^{1.5} = (17) (1.0) \sqrt[5]{4,000}$ psi (4 in.)^{1.5} = 8,601 lb

$N_{c,t} = 17.4.2.2 \ (b)$ Eq. (17.4.2.2b) Table 3
3.1.10 Design examples

<table>
<thead>
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<th>Calculation per ACI 318-14 Chapter 17, ICC-ES ESR-1917, KWIK Bolt TZ</th>
<th>ACI 318 ref.</th>
<th>ESR ref.</th>
</tr>
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<tbody>
<tr>
<td>Calculate nominal pullout strength in tension: ( N_{pm,c} = N_{p,use} \sqrt{\frac{f_{ck}}{2,500}} )</td>
<td>–</td>
<td>Section 4.1.8.2 Table 3</td>
</tr>
<tr>
<td>Note: Pullout strength does not need to be considered. Reference Table 3 in ESR-1917.</td>
<td>–</td>
<td>Table 3</td>
</tr>
<tr>
<td>Calculate nominal steel strength in shear: ( V_{sa} )</td>
<td>17.5.1.2</td>
<td>Section 4.1.5</td>
</tr>
<tr>
<td>Four anchors in shear. Highest load acting on a single anchor = ( V_{sa} / 4 = 6,000 \text{ lb} / 4-\text{anchors} = 1,500 \text{ lb} / \text{anchor} )</td>
<td>17.5.1.2</td>
<td>Eq. (17.5.1.2b) Table 3</td>
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<tr>
<td>Steel strength: ( V_{sa} = V_{w} = 7,600 \text{ lb} / \text{anchor} )</td>
<td>17.5.1.2</td>
<td>Eq. (17.5.1.2b) Table 3</td>
</tr>
<tr>
<td>Calculate nominal concrete breakout strength in shear: ( V_{cb} = \frac{A_{vc}}{A_{v0}} \cdot \Psi_{ec,V} \cdot \Psi_{ed,V} \cdot \Psi_{c,V} \cdot \Psi_{h,V} \cdot V_{b} )</td>
<td>17.5.2.1(b)</td>
<td>Section 4.1.6</td>
</tr>
<tr>
<td>Note: Shear load acts in the (-y) direction. ( c_{y} = \infty \rightarrow \text{no concrete breakout assumed in the } -y \text{ direction} )</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>Concrete breakout for shear parallel to the edge ((+x) direction) should be checked per 17.5.2.1(c).</td>
<td>17.5.2.1(c)</td>
<td>–</td>
</tr>
<tr>
<td>Assume the full shear load acts on the row of anchors nearest to the ( x^+ ) edge ( c_{st} = 6 \text{ in.} )</td>
<td>17.5.2.1</td>
<td>–</td>
</tr>
<tr>
<td>( 1.5c_{st} = (1.5) (6 \text{ in.}) = 9 \text{ in.} ) if ( c \geq 9 \text{ in.} \rightarrow \text{use } 1.5c_{st} ) ( h = 12 \text{ in.} )</td>
<td>17.5.2.1</td>
<td>–</td>
</tr>
<tr>
<td>( s_{max} = 3 \cdot c_{st} = (3) (6 \text{ in.}) = 18 \text{ in.} ) if ( s &gt; 18 \text{ in.} \rightarrow \text{no group action} )</td>
<td>17.5.2.1</td>
<td>–</td>
</tr>
<tr>
<td>( A_{vc} = (c_{y} + s_{y} + c_{st}) ) (MINIMUM ( [1.5c_{st} ; h] ) = (8 in. + 4 in. + 9 in.) = 189 in(^2)</td>
<td>17.5.2.1(c)</td>
<td>–</td>
</tr>
<tr>
<td>( A_{v0} = (4.5) (c_{y})^2 = (4.5) (6 \text{ in.})^2 = 162 \text{ in}^2 )</td>
<td>17.5.2.1(c)</td>
<td>–</td>
</tr>
<tr>
<td>No shear eccentricity ( \Psi_{ec,V} = 1.0 )</td>
<td>17.5.2.5</td>
<td>Eq. (17.5.2.5)</td>
</tr>
<tr>
<td>The edge distances perpendicular to the direction of the shear load are defined as ( c_{st} ). Note: 17.5.2.1(c) permits ( \Psi_{ed,V} = 1.0 ) to be used when calculating shear parallel to an edge. The ( \Psi_{ed,V} ) calculation in this example is conservative.</td>
<td>17.5.2.6</td>
<td>Eq. (17.5.2.6b)</td>
</tr>
<tr>
<td>( c_{ab-y} = 8 \text{ in.} ) ( c_{al-y} = \infty ) ( \Psi_{ed,V} = 0.7 + 0.3 \left( \frac{c_{ab-y}}{1.5c_{st}} \right) = 0.7 + 0.3 \left( \frac{8 \text{ in.}}{9 \text{ in.}} \right) = 0.967 )</td>
<td>17.5.2.7</td>
<td>–</td>
</tr>
<tr>
<td>Cracked concrete conditions, no edge reinforcement assumed ( \Psi_{ec,V} = 1.0 ) Check: ( h_a = 12 \text{ in.} \cdot 1.5c_{st} = 9 \text{ in.} ) ( 12 \text{ in.} &gt; 9 \text{ in.} \rightarrow \Psi_{h,V} = 1.0 )</td>
<td>17.5.2.8</td>
<td>Eq. (17.5.2.8)</td>
</tr>
<tr>
<td>Note: normal weight concrete ( \lambda_s = 1.0 )</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>(a) ( V_b = (7) \left( \frac{\ell_s}{d_{anchor}} \right)^{0.3} \lambda_s \sqrt{d_{anchor}} \sqrt{f_{ck}} \left( c_{st} \right)^{1.5} )</td>
<td>17.5.2.2</td>
<td>Eq. (17.5.2.2a) Table 3</td>
</tr>
<tr>
<td>( \ell_s = \text{MINIMUM} \left[ h_a ; 8d_{anchor} \right] = \text{MINIMUM} \left[ 4 \text{ in.} ; 5 \text{ in.} \right] = 4 \text{ in.} )</td>
<td>17.5.2.2(b)</td>
<td>–</td>
</tr>
<tr>
<td>( V_b = (7) \left( \frac{4 \text{ in.}}{0.625 \text{ in.}} \right)^{0.3} (1.0) \sqrt{0.625 \text{ in.} \cdot 6,000 \text{ psi} \cdot (6 \text{ in.})^{1.5}} = 7,456 \text{ lb} )</td>
<td>17.5.2.2(b)</td>
<td>–</td>
</tr>
<tr>
<td>(b) ( V_b = 9 \lambda_s \sqrt{f_{ck}} \left( c_{st} \right)^{1.5} = (9) (1.0) \sqrt{4,000 \text{ psi} \cdot (6 \text{ in.})^{1.5}} = 8,366 \text{ lb} )</td>
<td>17.5.2.2(b)</td>
<td>–</td>
</tr>
<tr>
<td>( V_b = \text{minimum} \left[ (a), (b) \right] = 7,456 \text{ lb} )</td>
<td>17.5.2.2(b)</td>
<td>–</td>
</tr>
<tr>
<td>( V_{cb} = \left( \frac{189 \text{ in}^2}{162 \text{ in}^2} \right) (1.0) (1.0) (1.0) (1.0) (7,456 \text{ lb}) = 8,699 \text{ lb} )</td>
<td>17.5.2.1(b)</td>
<td>Table 3</td>
</tr>
<tr>
<td>Calculate shear parallel to edge: ( V_{cb,parallel} = (2) (8,699 \text{ lb}) = 17,398 \text{ lb} )</td>
<td>17.5.2.1(c)</td>
<td>Table 3</td>
</tr>
</tbody>
</table>
3.1 Anchor Principles and Design

3.1.10 Design examples

<table>
<thead>
<tr>
<th>Calculation per ACI 318-14 Chapter 17, ICC-ES ESR-1917, KWIK Bolt TZ</th>
<th>ACI 318 ref.</th>
<th>ESR ref.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Calculate nominal concrete pryout strength in shear: ( V_{cbp} = (k \cdot N_{cbp}) )</td>
<td>17.5.3.1 (b)</td>
<td>Section 4.1.7</td>
</tr>
<tr>
<td>( V_{cbp} = (k \cdot N_{cbp}) ) ( N_{cbp} = 15,290 ) lb ( h_{ef} = 4 ) in. ( k = 2 )</td>
<td>17.5.3.1 (b)</td>
<td>Eq. (17.5.3.1b)</td>
</tr>
<tr>
<td>( V_{cbp} = (2)(15,290) ) lb = 30,580 lb</td>
<td></td>
<td>Table 3</td>
</tr>
</tbody>
</table>

**Summary**

Anchors are ductile steel elements → check 17.2.3.4.3 (a) first

[Ductility check] Tension calculations per 17.2.3.4.3 (a)

<table>
<thead>
<tr>
<th>Tension</th>
<th>Nominal strength</th>
<th>Design strength</th>
<th>Factored load</th>
<th>% Utilization</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel strength</td>
<td>17,170 lb/anchor</td>
<td>20,604 lb/anchor</td>
<td>375 lb/anchor</td>
<td>1.80%</td>
<td>OK</td>
</tr>
<tr>
<td>Concrete breakout</td>
<td>15,290 lb</td>
<td>-</td>
<td>1,500 lb</td>
<td>17.30%</td>
<td>OK Controls</td>
</tr>
<tr>
<td>Pullout strength</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
</tbody>
</table>

Steel strength does not control: 17.2.3.4.3 (a) is, therefore, not satisfied. Need to satisfy 17.2.3.4.3 (d)

Tension calculations per 17.2.3.4.3 (d)

<table>
<thead>
<tr>
<th>Tension</th>
<th>Nominal strength</th>
<th>Design strength</th>
<th>Factored load</th>
<th>% Utilization</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel strength</td>
<td>17,170 lb/anchor</td>
<td>( \phi_{steel} = 0.75 ) ( 12,878 ) lb/anchor</td>
<td>750 lb/anchor</td>
<td>5.80%</td>
<td>OK</td>
</tr>
<tr>
<td>Concrete breakout</td>
<td>15,290 lb</td>
<td>( \phi_{concrete} = 0.65 ) ( 7,454 ) lb</td>
<td>3,000 lb</td>
<td>40.30%</td>
<td>OK Controls</td>
</tr>
<tr>
<td>Pullout strength</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
</tbody>
</table>

Shear calculations per 17.2.3.5.3 (c)

<table>
<thead>
<tr>
<th>Shear</th>
<th>Nominal strength</th>
<th>Design strength</th>
<th>Factored load</th>
<th>% Utilization</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel strength</td>
<td>7,600 lb/anchor</td>
<td>( \phi_{steel} = 0.65 ) ( 4,940 ) lb/anchor</td>
<td>1,500 lb/anchor</td>
<td>30.40%</td>
<td>OK</td>
</tr>
<tr>
<td>Concrete breakout</td>
<td>17,398 lb</td>
<td>( \phi_{concrete} = 1.0 ) ( 12,179 ) lb</td>
<td>6,000 lb</td>
<td>49%</td>
<td>OK Controls</td>
</tr>
<tr>
<td>Pryout</td>
<td>30,580 lb</td>
<td>( \phi_{concrete} = 1.0 ) ( 21,408 ) lb</td>
<td>6,000 lb</td>
<td>28%</td>
<td>OK</td>
</tr>
</tbody>
</table>

Calculation per ACI 318-14 Chapter 17, ICC-ES ESR-1917, KWIK Bolt TZ

<table>
<thead>
<tr>
<th>Interaction equation</th>
<th>ACI 318 ref.</th>
<th>ESR ref.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Check: ( V_{ua} \leq (0.2)\phi V_{cbp} )</td>
<td>17.6.1</td>
<td>Section 4.1.9</td>
</tr>
<tr>
<td>( V_{ua} = 6,000 ) lb ( (0.2)(12,179) ) lb = 2,436 lb</td>
<td></td>
<td></td>
</tr>
<tr>
<td>( V_{ua} &gt; (0.2)\phi V_{cbp} )</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Check: ( N_{ua} \leq (0.2)\phi N_{cbp} )</td>
<td>17.6.2</td>
<td></td>
</tr>
<tr>
<td>( N_{ua} = 3,000 ) lb ( (0.2)(7,454) ) lb = 1,491 lb</td>
<td></td>
<td></td>
</tr>
<tr>
<td>( N_{ua} &gt; (0.2)\phi N_{cbp} )</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Use interaction equation:

Tri - linear : \( 0.403 + 0.49 = 0.89 < 1.2 \rightarrow OK \)
Parabolic : \( (0.403)^{1.0} + (0.49)^{1.0} = 0.52 < 1.0 \rightarrow OK \)

This fastening satisfies the design criterion that have been assumed.
### 3.1.10 Design examples

#### Strength Design example, mechanical anchors, KWIK HUS-EZ

**Objective:**
- Determine the controlling design strength in tension and shear
- Check the controlling design strength in tension and shear against the factored service loads in tension and shear.

**Dimensional parameters:**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$d_0$</td>
<td>1/2 in.</td>
</tr>
<tr>
<td>$h_{nom}$</td>
<td>3 in.</td>
</tr>
<tr>
<td>$h_{ef}$</td>
<td>2.16 in.</td>
</tr>
<tr>
<td>$t_{fixture}$</td>
<td>3/8 in.</td>
</tr>
<tr>
<td>$h$</td>
<td>6 in.</td>
</tr>
<tr>
<td>$s$</td>
<td>5 in.</td>
</tr>
<tr>
<td>$c$</td>
<td>2 in.</td>
</tr>
</tbody>
</table>

**Given:**
- Normal weight concrete, $f'_{c} = 4,000$ psi; cracked concrete conditions assumed; seismic design category (SDC) C
- Reference ACI 318-14, Ch. 5 and ICC-ES ESR-3027 for LRFD Factors
- Carbon steel 1/2" x 4" KWIK HUS-EZ anchors. Anchors are considered non-ductile steel elements
- 2 – anchors in tension: No tension eccentricity
- 2 – anchors in shear: No shear eccentricity
- [Seismic with $\Omega_0$ (17.2.3.4.3 (d))] $N_{ua} = 1,000$ lb $V_{ua} = 800$ lb towards the fixed edge

**Things to check:**
- Geometry requirements
- Seismic per 17.2.3.4.3 (d)
- Tension design strengths
- Seismic per 17.2.3.5.3 (c)
- Shear design strengths
- Tension/shear interaction

**References:**
- Mechanical anchor – KWIK HUS-EZ
- ACI 318-14, Chapter 17
- ICC-ES ESR-3027

**Calculation per ACI 318-14 Chapter 17, ICC-ES ESR-3027, KWIK HUS-EZ**

<table>
<thead>
<tr>
<th>Requirement</th>
<th>Criterion</th>
<th>ACI 318 Ref.</th>
<th>ESR Ref.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Check minimum anchor spacing, edge distance, concrete member thickness</td>
<td>$c_{min} = 1.75$ in. $s_{min} = 3$ in.</td>
<td>17.7</td>
<td>Section 4.1.10</td>
</tr>
<tr>
<td>Notes on tension parameters:</td>
<td>$3h_{id} = (3)(2.16)$ in. = 6.48 in. $s = 5$ in. &lt; 6.48 in. → consider group action</td>
<td>R17.7</td>
<td>Table 2</td>
</tr>
<tr>
<td>$1.5h_{fy} = (1.5)(2.16)$ in. = 3.24 in. $c = 2$ in. &lt; 3.24 in. → consider edge influence</td>
<td>R17.4.2.1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Notes on shear parameters:</td>
<td>Check shear perpendicular to x + edge $\rightarrow c_{sl} = 2$ in.</td>
<td>17.7</td>
<td>Section 4.1.10</td>
</tr>
<tr>
<td>$3c_{sl} = (3)(2)$ in. = 6 in. $s = 5$ in. &lt; 6 in. → consider group action</td>
<td>17.5.2.1</td>
<td>Table 2</td>
<td></td>
</tr>
<tr>
<td>$1.5c_{sl} = (1.5)(2)$ in. = 3 in. $c = 2$ in. &lt; 3 in. → consider edge influence</td>
<td>R17.5.2.1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$h_{min} = 5.50$ in. $h = 6$ in. &gt; 5.50 in. OK</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Notes on installation:</td>
<td>$h_{id} = 2.16$&quot; for a 1/2&quot; KWIK HUS-EZ having $h_{nom} = 3$&quot;. $h = 6$ in. &gt; 5.50 in. OK</td>
<td>17.7</td>
<td>Section 4.1.10</td>
</tr>
<tr>
<td>The actual length not including the head ($l_{anch}$) = 4&quot; for a 1/2&quot; x 4&quot; KH-EZ.</td>
<td>R17.7</td>
<td>Table 1</td>
<td></td>
</tr>
<tr>
<td>The actual $h_{nom} = l_{anch} - t_{fixture}$ = 4 in. − 0.375 in. = 3.25 in.</td>
<td></td>
<td>Table 2</td>
<td></td>
</tr>
<tr>
<td>$h_{nom} = 3.25$ in. + 0.375 in. = 4 in. OK</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$h_{eff} = 3.625$ in. + 0.375 in. = 4 in. OK</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Calculate nominal steel strength in tension: $N_{sa}$</td>
<td></td>
<td>17.4.1.2</td>
<td>Section 4.1.2</td>
</tr>
<tr>
<td>Two anchors in tension</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Highest tension load acting on a single anchor = $N_{sa} / 2 = 1,000$ lb / 2-anchors = $500$ lb / anchor</td>
<td></td>
<td>17.4.1.2</td>
<td>Table 3</td>
</tr>
<tr>
<td>Steel strength: $N_{sa} = 18,120$ lb/anchor</td>
<td></td>
<td>Eq. (17.4.1.2)</td>
<td></td>
</tr>
</tbody>
</table>
3.1 Anchor Principles and Design

3.1.10 Design examples

<table>
<thead>
<tr>
<th>Calculation per ACI 318-14 Chapter 17, ICC-ES ESR-3027, KWIK HUS-EZ</th>
<th>ACI 318 ref.</th>
<th>ESR ref.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Calculate nominal concrete breakout strength in tension: ( N_{c,l} = \frac{A_{c,l}}{A_{c,co}} \cdot \psi_{ec,N} \cdot \psi_{ed,N} \cdot \psi_{c,N} \cdot \psi_{cp,N} \cdot N_{c} )</td>
<td>Eq. (17.4.2.1b)</td>
<td>Section 4.1.3</td>
</tr>
<tr>
<td>( c_{a1} = \infty ) ( s_{a} = 5 \text{ in.} ) ( c_{a2} = \infty ) ( c_{y} = \infty ) ( c_{y} = 2 \text{ in.} ) ( c_{max} = 1.5 \cdot h_{ef} = (1.5) (2.16 \text{ in.}) = 3.24 \text{ in.} ) if ( c \geq 3.24 \text{ in.} \rightarrow 1.5 \cdot h_{ef} ) ( s_{max} = 3 \cdot h_{ef} = (3) (2.6 \text{ in.}) = 6.48 \text{ in.} ) if ( s &gt; 6.48 \text{ in.} \rightarrow ) no group action ( A_{nc} = (c_{a1} + s_{a} + c_{y})(c_{y} + c_{y}) = (3.24 \text{ in.} + 5 \text{ in.} + 3.24 \text{ in.}) (3.24 \text{ in.} + 2 \text{ in.}) = 60.15 \text{ in}^2 ) ( A_{nc0} = 9 (b_{ef})^2 = (9) (2.16 \text{ in.})^2 = 42 \text{ in}^2 )</td>
<td>17.4.2.1</td>
<td>–</td>
</tr>
<tr>
<td>No tension eccentricity ( \rightarrow \psi_{ec,N} = 1.0 )</td>
<td>Eq. (17.4.2.4)</td>
<td>–</td>
</tr>
<tr>
<td>( \psi_{ec,N} = 0.70 + 0.3 \left( \frac{c_{a,cm}}{1.5h_{ef}} \right) ) ( = 0.70 + 0.3 \left( \frac{2 \text{ in.}}{(1.5) (2.16 \text{ in.})} \right) = 0.885 )</td>
<td>17.4.2.5</td>
<td>–</td>
</tr>
<tr>
<td>Note: cracked concrete conditions have been assumed. ( \psi_{c,N} = 1.0; \psi_{cp,N} = 1.0 )</td>
<td>–</td>
<td>Table 3</td>
</tr>
<tr>
<td>Note: normal weight concrete ( \rightarrow \lambda_{a} = 1.0 ) ( N_{b} = k_{c,cr} \lambda_{a} \sqrt{f_{c}'} (h_{ef})^{1.5} = (17) (1.0) \sqrt{4,000} (2.16 \text{ in.})^{1.5} = 3,413 \text{ lb} )</td>
<td>Eq. (17.4.2.2a)</td>
<td>Table 3</td>
</tr>
<tr>
<td>( N_{eq} = \frac{60.15 \text{ in}^2}{42 \text{ in}^2} \left( \frac{1}{0.85} \right) \left( \frac{1}{1.0} \right) \left( \frac{3,413 \text{ lb}}{1.0} \right) = 4,326 \text{ lb} )</td>
<td>17.4.2.1</td>
<td>(b)</td>
</tr>
<tr>
<td>Calculate nominal pullout strength in tension: ( N_{eq} = \frac{f_{c}'}{2500} \sqrt{V_{s}} )</td>
<td>–</td>
<td>Section 4.1.8.2</td>
</tr>
<tr>
<td>Note: Pullout strength does not need to be considered. Reference Table 3 in. ESR-3027.</td>
<td>–</td>
<td>Table 3</td>
</tr>
<tr>
<td>Calculate nominal steel strength in shear: ( V_{sa} )</td>
<td>17.5.1.2</td>
<td>Section 4.1.5</td>
</tr>
<tr>
<td>Two anchors in shear</td>
<td>–</td>
<td>Table 4</td>
</tr>
<tr>
<td>Highest load acting on a single anchor = ( V_{sa} / 2 ) = 800 lb / 2-anchors = 400 lb / anchor</td>
<td>17.5.1.2</td>
<td>(b)</td>
</tr>
<tr>
<td>Steel strength: ( V_{sa} = V_{eq} = 5,547 \text{ lb/anchor} )</td>
<td>17.5.2.1</td>
<td>(b)</td>
</tr>
<tr>
<td>Calculate nominal concrete breakout strength in shear: ( V_{cbg} = \frac{A_{c,b}}{A_{c,co}} \cdot \psi_{ec,V} \cdot \psi_{ed,V} \cdot \psi_{c,V} \cdot \psi_{h,V} \cdot V_{b} )</td>
<td>17.5.2.1</td>
<td>(b)</td>
</tr>
<tr>
<td>( c_{a1} = 2 \text{ in.} ) ( s_{a} = 5 \text{ in.} ) ( c_{a2} = \infty ) ( c_{y} = \infty ) ( 1.5c_{y} = 3 \text{ in.} ) ( c_{max} = 1.5 \cdot h_{ef} = (1.5c_{y}) (2.6 \text{ in.}) = 3.9 \text{ in.} ) ( s_{max} = 3 \cdot h_{ef} = (3) (2.6 \text{ in.}) = 7.8 \text{ in.} ) ( A_{nc} = (c_{a1} + s_{a} + c_{y})(1.5c_{y}) = (2.6 \text{ in.} + 5 \text{ in.} + 3.9 \text{ in.}) (1.5c_{y}) = (3 \text{ in.} + 5 \text{ in.} + 3.9 \text{ in.}) (3 \text{ in.}) = 33 \text{ in}^2 ) ( A_{nc0} = (4.5) (c_{y})^2 = (4.5) (2.6 \text{ in.})^2 = 18 \text{ in}^2 )</td>
<td>17.5.2.1</td>
<td>(b)</td>
</tr>
<tr>
<td>No shear eccentricity ( \rightarrow \psi_{ec,V} = 1.0 )</td>
<td>Eq. (17.5.2.5)</td>
<td>–</td>
</tr>
<tr>
<td>Edge projections in x+ and x- directions are assumed to be infinite for purposes of concrete breakout calculations in shear ( \rightarrow \psi_{ed,V} = 1.0 )</td>
<td>17.5.2.6</td>
<td>–</td>
</tr>
<tr>
<td>Cracked concrete conditions, no edge reinforcement assumed ( \rightarrow \psi_{c,V} = 1.0 )</td>
<td>Eq. (17.5.2.6a)</td>
<td>–</td>
</tr>
</tbody>
</table>
### 3.1.10 Design examples

![Formula and calculation](image)

#### Calculation per ACI 318-14 Chapter 17, ICC-ES ESR-3027, KWIK HUS-EZ

<table>
<thead>
<tr>
<th>Calculation</th>
<th>ACI 318 ref.</th>
<th>ESR ref.</th>
</tr>
</thead>
</table>
| \[
\psi_{h,v} = \left( \frac{1.5c_{sl}}{h_a} \right)^{0.2} \] Check: \( h_a = 6 \text{ in.} ; 1.5c_{sl} = 3 \text{ in.} \) \( 6 \text{ in.} > 3 \text{ in.} \) \( \rightarrow \psi_{h,v} = 1.0 \) | 17.5.2.8 Eq. (17.5.2.8) |

**Note:** normal weight concrete \( \rightarrow \lambda_a = 1.0 \)

\[(a) \, V_b = (7) \left( \frac{\ell_s}{d_a} \right)^{0.2} \lambda_a \sqrt{d_a} \sqrt{f'_{c}} (c_{sl})^{1.5} \]

\( \ell_s = \text{MINIMUM} \left[ h_a; 8d_{anchor} \right] = \text{MINIMUM} \left[ 2.16 \text{ in.}; 4 \text{ in.} \right] = 2.16 \text{ in.} \)

\[(b) \, V_b = 9 \lambda_a f'_c (c_{sl})^{1.5} = (9) (1.0) 4,000 \text{ psi} (2 \text{ in.})^{1.5} = 1,610 \text{ lb} \]

**Calculations**

\[V_{cbg} = (1.0) (1.0) (1.0) (1.0) (1,186 \text{ lb}) = 2,174 \text{ lb} \]

**Shear calculations per 17.2.3.5.3 (c)**

<table>
<thead>
<tr>
<th>Shear</th>
<th>Nominal strength</th>
<th>Design strength</th>
<th>Factored load</th>
<th>% Utilization</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel strength</td>
<td>5,547 lb/anchor</td>
<td>( \phi_{steel} = 0.60 ) 3,328 lb/anchor</td>
<td>400 lb/anchor</td>
<td>12.00%</td>
<td>OK</td>
</tr>
<tr>
<td>Concrete breakout</td>
<td>2,174 lb</td>
<td>( \phi_{steel} = 1.0 ) ( \phi_{concrete} = 0.70 ) 1,523 lb</td>
<td>800 lb</td>
<td>52.5%</td>
<td>OK controls</td>
</tr>
<tr>
<td>Pryout</td>
<td>4,326 lb</td>
<td>( \phi_{steel} = 1.0 ) ( \phi_{concrete} = 0.70 ) 3,028 lb</td>
<td>800 lb</td>
<td>26%</td>
<td>OK</td>
</tr>
</tbody>
</table>

**Summary**

Cannot check 17.2.3.4.3 (a) because KWIK HUS-EZ is a non-ductile anchor element. Check 17.2.3.4.3 (d)
### 3.1 Anchor Principles and Design

#### 3.1.10 Design examples

<table>
<thead>
<tr>
<th>Calculation per ACI 318-14 Chapter 17, ICC-ES ESR-3027, KWIK HUS-EZ</th>
<th>ACI 318 ref.</th>
<th>ESR ref.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Interaction equation</td>
<td>17.6</td>
<td>Section 4.1.9</td>
</tr>
<tr>
<td>Check: ( V_{ua} \leq (0.2)\Phi V_{cbg} )</td>
<td></td>
<td>17.6.1</td>
</tr>
<tr>
<td>( V_{ua} = 800 \text{ lb} ) ( (0.2) (1,523 \text{ lb}) = 305 \text{ lb} )</td>
<td></td>
<td></td>
</tr>
<tr>
<td>( V_{ua} &gt; (0.2)\Phi V_{cbg} )</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Check: ( N_{ua} \leq (0.2)\Phi N_{cbg} )</td>
<td>17.6.2</td>
<td>-</td>
</tr>
<tr>
<td>( N_{ua} = 1,000 \text{ lb} ) ( (0.2) (2,109 \text{ lb}) = 422 \text{ lb} )</td>
<td></td>
<td></td>
</tr>
<tr>
<td>( N_{ua} &gt; (0.2)\Phi N_{cbg} )</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Use interaction equation:

- **Tri-linear**: \( 0.47 + 0.53 = 1.0 < 1.2 \rightarrow \text{OK} \)
- **Parabolic**: \( (0.47)^{5/3} + (0.53)^{5/3} = 0.288 + 0.347 = 0.635 < 1.0 \rightarrow \text{OK} \)

This fastening satisfies the design criteria that have been assumed.
3.1.10 Design Examples

Objective:
Determine the controlling design strength in tension and shear.
Check the design strengths against the factored service loads.

Dimensional parameters:

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$h_{xf}$</td>
<td>15 in.</td>
</tr>
<tr>
<td>$h$</td>
<td>24 in.</td>
</tr>
<tr>
<td>$s_x$</td>
<td>8 in.</td>
</tr>
<tr>
<td>$s_{y1}$</td>
<td>12 in.</td>
</tr>
<tr>
<td>$s_{y2}$</td>
<td>12 in.</td>
</tr>
<tr>
<td>$c_{x}$</td>
<td>6 in.</td>
</tr>
<tr>
<td>$c_{y}$</td>
<td>$\infty$</td>
</tr>
<tr>
<td>$d_a$</td>
<td>1 in.</td>
</tr>
<tr>
<td>$d_{hole}$</td>
<td>1.125 in.</td>
</tr>
</tbody>
</table>

Given:
Normal weight concrete, $f'c = 6,000$ psi; cracked concrete conditions assumed; seismic design category (SDC) D
Use ACI 318-14 Ch. 5 and ICC-ES ESR-3187 for LRFD Factors

=> Assume condition B for all $q$ factors and temperature range A

HIT-HY 200 Adhesive with 1" ASTM A193 B7 threaded rod; anchors are considered ductile steel elements

Four anchors in tension: Tension eccentricity = 2.59 in.
Six anchors in shear: no eccentricity in shear

[Seismic without $\Omega_0$ (17.2.3.4.3 (a))] $M_{ua}$ (x-axis) = 184,000 in-lb $V_{ua} = 6,800$ lb (+x direction)
[Seismic with $\Omega_0$ (17.2.3.4.3 (d))] $M_{ua}$ (x-axis) = 230,000 in-lb $V_{ua} = 8,500$ lb (+x direction)

Things to check:
- Geometry requirements
- Tension design strengths
- Shear design strengths
- Ductility per 17.2.3.4.3 (a) and 17.2.3.4.3 (d)
- Seismic per 17.2.3.5.3
- Seismic/Shear Interaction
- Sustained load per 17.3.1.2

References:
- ACI 318-14 Chapter 17
- ICC-ES Acceptance Criteria AC308
- ICC-ES ESR-3187

Determine Distribution of Loads on Anchor Group

<table>
<thead>
<tr>
<th>Anchor</th>
<th>Seismic tension without $\Omega_0$</th>
<th>Seismic tension with $\Omega_0$</th>
<th>Seismic shear without $\Omega_0$</th>
<th>Seismic shear with $\Omega_0$</th>
<th>Sustained tension load (non seismic)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>3,205 lb/anchor</td>
<td>4,006 lb/anchor</td>
<td>1,133 lb/anchor</td>
<td>1,417 lb/anchor</td>
<td>1,742 lb/anchor</td>
</tr>
<tr>
<td>2</td>
<td>1,271 lb/anchor</td>
<td>1,589 lb/anchor</td>
<td>1,133 lb/anchor</td>
<td>1,417 lb/anchor</td>
<td>691 lb/anchor</td>
</tr>
<tr>
<td>3</td>
<td>0</td>
<td>0</td>
<td>1,133 lb/anchor</td>
<td>1,417 lb/anchor</td>
<td>0</td>
</tr>
<tr>
<td>4</td>
<td>3,205 lb/anchor</td>
<td>4,006 lb/anchor</td>
<td>1,133 lb/anchor</td>
<td>1,417 lb/anchor</td>
<td>1,742 lb/anchor</td>
</tr>
<tr>
<td>5</td>
<td>1,271 lb/anchor</td>
<td>1,589 lb/anchor</td>
<td>1,133 lb/anchor</td>
<td>1,417 lb/anchor</td>
<td>691 lb/anchor</td>
</tr>
<tr>
<td>6</td>
<td>0</td>
<td>0</td>
<td>1,133 lb/anchor</td>
<td>1,417 lb/anchor</td>
<td>0</td>
</tr>
<tr>
<td>Resultant</td>
<td>8,952 lb</td>
<td>11,190 lb</td>
<td>6,800 lb</td>
<td>8,500 lb</td>
<td>–</td>
</tr>
</tbody>
</table>
3.1 Anchor Principles and Design

3.1.10 Design examples

Calculation per ACI 318-14 Chapter 17, ICC-ES ESR-3187*, HIT-HY 200

Check minimum: anchor spacing, edge distance and member thickness

**MINIMUM** \[ \{ s_x : s_y : s_z \} = \text{MINIMUM } \{ 8 \text{ in.} : 12 \text{ in.} : 12 \text{ in.} \} = 8 \text{ in.} \]

**MINIMUM** \[ \{ c_x : c_y : c_z \} = \text{MINIMUM } \{ 6 \text{ in.} : \infty : 12 \text{ in.} \} = 6 \text{ in.} \]

\[ c_{\text{min}} = 5 \text{ in.} \quad c_{\text{max}} = 1.5h_y = (1.5)(15 \text{ in.}) = 22.5 \text{ in.} \]

Note: if an edge distance is > 1.5\(h_y\), it is not assumed to influence the anchor capacity unless splitting is considered: 5 in. ≤ 6 in. ≤ 22.5 in. OK

\[ h_{\text{min}} = h_y + 2d_{\text{hole}} = 15 \text{ in.} + 2(1.125 \text{ in.}) = 17.25 \text{ in.} \]

\[ h = 24 \text{ in.} > 17.25 \text{ in.} \quad \text{OK} \]

**MINIMUM** \[ \{ x : y1 : y2 \} = \text{MINIMUM } \{ 8 \text{ in.} : 12 \text{ in.} : 12 \text{ in.} \} = 8 \text{ in.} \]

\[ \text{min} = 5 \text{ in.} \quad \text{smax} = 3h_y = (3)(15 \text{ in.}) = 45 \text{ in.} \]

**NOTE:** anchors spaced > 3\(h_y\) are not assumed to act as a group in tension; 5 in. ≤ 8 in. ≤ 45 in. OK

\[ c_{\text{max}} = 1.5 \cdot h_y = 22.5 \text{ in.} \text{ if } c \geq 22.5 \text{ in.} \rightarrow \text{use } 1.5 \cdot h_y \]

\[ s_{\text{max}} = 3 \cdot h_y = 45 \text{ in.} \text{ if } s > 45 \text{ in.} \rightarrow \text{no group action} \]

\[ AN_{c} = (c_{x} + s_{x} + c_{x}) (c_{y} + s_{y} + c_{y}) = (22.5 \text{ in.} + 8 \text{ in.} + 6 \text{ in.})(22.5 \text{ in.} + 12 \text{ in.} + 22.5 \text{ in.}) = 2,080.5 \text{ in}^2 \]

\[ AN_{c0} = (9) (h_y)^2 = (9)(15 \text{ in.})^2 = 2,025 \text{ in}^2 \]

**Note:** tension eccentricity determined using PROFIS Anchor software.

\[ e_{N'} = 2.592 \text{ in.} \text{ (Distance from } N_{\text{resultant}} \text{ to C.O.G. of the anchors in tension)} \]

\[ \psi_{c,N} = \frac{1}{1 + \left( \frac{2e_{N'}}{3h_y} \right)} = 0.90 \]

\[ \psi_{c,N} = 0.7 + 0.3 \left( \frac{c_{\text{min}}}{1.5h_y} \right) = 0.7 + 0.3 \left( \frac{6 \text{ in.}}{(1.5)(15 \text{ in.})} \right) = 0.78 \]

**Note:** cracked concrete conditions have been assumed. \(\psi_{c,N} = 1.0; \psi_{cp,N} = 1.0\)

\[ N_{b} = k_{b} \lambda_{b} \sqrt{f_{c'}^2} (h_y)^{1.5} = (17)(1.0) \sqrt{6,000 \text{ psi}} (15 \text{ in.})^{1.5} = 76,500 \text{ lb} \]

\[ N_{\text{dog}} = \frac{2,080.5 \text{ in}^2}{2,025 \text{ in}^2} (0.897)(0.78)(1.0)(1.0)(76,500) = 54,991 \text{ lb} \]

Calculate nominal bond strength in tension: \(N_{bg} = \frac{A_{Nc} \cdot \psi_{ec,N} \cdot \psi_{ec,N} \cdot \psi_{cp,N} \cdot N_{b}}{A_{Nco}} \)

\[ c_{Na} = (10)(1.125 \text{ in.}) \sqrt{\frac{\tau_{\text{uncr}}}{1,100}} \quad \tau_{\text{uncr}} = 1,670 \text{ psi} \quad c_{Na} = 12.86 \text{ in.} \]

\[ c_{\text{max}} = c_{Na} \quad \text{if } c \geq c_{Na} \rightarrow \text{use } c_{Na} \quad s_{\text{max}} = s_{Na} \quad \text{if } s \geq s_{Na} \rightarrow \text{no group action} \quad [s_{Na} = 2c_{Na}] \]

\[ A_{Na} = (c_{x} + s_{x} + c_{x})(c_{y} + s_{y} + c_{y}) = (12.86 \text{ in.} + 8 \text{ in.} + 6 \text{ in.})(12.86 \text{ in.} + 12 \text{ in.} + 12.86 \text{ in.}) = 1,013.2 \text{ in}^2 \]

\[ A_{Nao} = (2c_{Na})^2 = (2)(12.86 \text{ in.})^2 = 661.5 \text{ in}^2 \]

---

**R17.4.2.1** Section 4.1.4

**Table 12**
### 3.1.10 Design examples

<table>
<thead>
<tr>
<th>Calculation per ACI 318-14 Chapter 17, ICC-ES ESR-3187*, HIT-HY 200</th>
<th>ACI 318 ref.</th>
<th>ESR ref.*</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \psi_{ed,Na} = 0.7 + 0.3 \left( \frac{c_{Na}}{c_{Na}} \right) )</td>
<td>17.4.5.4 (17.4.5.4b)</td>
<td>Section 4.1.4 D.5.3.12 Eq. (D-16m)</td>
</tr>
<tr>
<td>Number of anchors in tension = 4, ( e'<em>{N} = 2.592 \text{ in.; } \psi</em>{ed,Na} = \frac{1}{1 + \left( \frac{e'<em>{N}}{c</em>{Na}} \right)} = \frac{1}{1 + \left( \frac{2.592 \text{ in.}}{12.86 \text{ in.}} \right)} = 0.83 )</td>
<td>17.4.5.3 (17.4.5.3)</td>
<td>Section 4.1.4 D.5.3.11 Eq. (D-16i)</td>
</tr>
<tr>
<td>Adjustment to bond strength = ( \left( \frac{f'_{c}}{2,500} \right) \cdot 0.1 = \frac{6,000}{2,500} \cdot 0.1 = 1.99 ) (See ESR-3187, Table 14, footnote 1)</td>
<td>17.4.5.5 (17.4.5.5a)</td>
<td>Section 4.1.4 D.5.3.14 Eq. (D-16o)</td>
</tr>
</tbody>
</table>

Note: cracked concrete conditions have been assumed. \( \psi_{ed,Na} = 1.0 \)

Note: cracked concrete conditions have been assumed. Normal-weight concrete: \( \lambda_{a} = 1.0 \)

\( \tau_{k,cr} = (1.09) (805 \text{ psi}) = 877 \text{ psi as per ESR-3187, Table 14} \)

\( \alpha_{N,seis} = 1.00 \) as per ESR-3187, Table 14

\( \tau_{k,seis} = \tau_{k,cr} \)

\( N_{ba} = (\lambda_{a}) (\tau_{k,cr}) (\frac{\pi}{4}) (d_{a}) (h_{ef}) = (1.0) (877) (\frac{\pi}{4}) (1 \text{ in.}) (15 \text{ in.}) = 41,328 \text{ lb} \)

\( N_{ag} = (0.84) (0.83) (1.0) (41,328 \text{ lb}) = 44,133 \text{ lb} \)

\( \psi_{ed,V} = 1.0 \)

Steel strength \( \alpha_{V,seis} V_{sa} = 0.7 \) \( V_{sa} = 45,425 \text{ lb/anchor} \)

\( \alpha_{V,seis} V_{sa} = (0.70) (45,425 \text{ lb}) = 31,798 \text{ lb/anchor} \)

Calculate nominal concrete breakout strength in shear: \( V_{cbg} \)

No shear eccentricity \( \rightarrow \psi_{ed,V} = 1.0 \)

\( \psi_{ed,V} = 1.0 \) if \( c_{42,min} > 1.5 c_{st} \)

\( \psi_{ed,V} = \sqrt{\frac{1.5 c_{st}}{h_{u}}} \)

Check: \( h_{u} = 24 \text{ in.} ; 1.5c_{st} = 9 \text{ in.} \) 24 in. > 9 in. \( \rightarrow \psi_{ed,V} = 1.0 \)
### 3.1 Anchor Principles and Design

#### 3.1.10 Design examples

**Calculation per ACI 318-14 Chapter 17, ICC-ES ESR-3187*, HIT-HY 200**

<table>
<thead>
<tr>
<th>Note: normal weight concrete → ( \lambda_a = 1.0 )</th>
<th>ACI 318 ref.</th>
<th>ESR ref.*</th>
</tr>
</thead>
<tbody>
<tr>
<td>(a) ( V_b = (7) \left( \frac{f_{c'}}{d_a} \right)^{0.2} \lambda_a \sqrt{d_a} \sqrt[3]{f'<em>{c}} \ (c_a)</em>{1.5}^{1.5} )</td>
<td>17.5.2.2</td>
<td>-</td>
</tr>
<tr>
<td>( f_a = \text{MINIMUM } [h_{a} \ : \ 8d_{a}] = \text{MINIMUM } [15 , \text{in.} \ : \ 8 , \text{in.}] = 8 , \text{in.} )</td>
<td>Eq. (17.5.2.2a)</td>
<td>-</td>
</tr>
<tr>
<td>( V_b = (7) \left( \frac{8 , \text{in.}}{1 , \text{in.}} \right)^{0.2} = (1.0) \sqrt{1 , \text{in.}} \cdot \sqrt[3]{6,000 , \text{psi}} \ (6 , \text{in.})^{1.5} = 12,078 , \text{lb} )</td>
<td>Eq. (17.5.2.2b)</td>
<td>-</td>
</tr>
<tr>
<td>(b) ( V_b = 9 \lambda_a \sqrt{f'<em>{c}} \ (c_a)</em>{1.5}^{1.5} = (9) (1.0) \sqrt{6,000} \ (6 , \text{in.})^{1.5} = 10,246 , \text{lb} )</td>
<td>-</td>
<td>Section 4.1.6</td>
</tr>
<tr>
<td>( V_b = \text{minimum } [(a), (b)] = 10,246 , \text{lb} )</td>
<td>-</td>
<td></td>
</tr>
</tbody>
</table>

**Calculate nominal concrete pryout strength in shear:** \( V_{cp} = (k_{cp}) \ (\text{MINIMUM } [N_{cpg} ; N_{cg}]) \)

| Note: tension eccentricity not considered for pryout. \( \psi_{k,x,Na} = 1.0 \) | 17.4.2.4 | - |
| \( \psi_{k,x,Na} = 0.7 + 0.3 \left( \frac{C_{\text{min}}}{1.5 \psi_{y,c}} \right) = 0.7 + 0.3 \left( \frac{6 \, \text{in.}}{1.5 \ (15 \, \text{in})} \right) = 0.78 \) | Eq. (17.4.2.5b) | - |

**Calculate \( N_{cg} \) for 6-anchors.**

| Note: cracked concrete conditions have been assumed. \( \psi_{k,Na} = 1.0 \) \( \psi_{\text{cr},Na} = 1.0 \) | 17.4.2.6 | - |
| Note: cracked concrete conditions have been assumed. Normal weight concrete \( \rightarrow \lambda_a = 1.0 \). \( N_a = k_{c,cr} \lambda_a \sqrt{f'_{c}} \ (h_{a})^{1.5} = (17) (1.0) \sqrt{6,000} \ (15 \, \text{in.})^{1.5} = 76,500 \, \text{lb} \) | 17.4.2.2 | Section 4.1.4 |
| \( N_{cpg} = \left( \frac{2,518 \, \text{in}^2}{2,025 \, \text{in}^2} \right)^{(1.0) (0.78) (1.0) (1.0) (76,500 \, \text{lb})} = 74,212 \, \text{lb} \) | Eq. (17.4.2.1b) | D.5.3.7 |
| Calculate \( N_{cg} \) for 6-anchors. \( C_{c} = \infty \ \ s_x = 8 \, \text{in.} \ \ c_x = 6 \, \text{in.} \ \ c_y = \infty \ \ s_y = 12 \, \text{in.} \ \ s_y = 12 \, \text{in.} \ \ c_y = \infty \) \( c_{Na} = 10 \ \left(1 \, \text{in.} \right) \left( \frac{1,820}{1,100} \right) = 12.86 \, \text{in.} \) \( A_{Na} = (c_x + s_x + c_y) (c_x + s_y) (c_y + s_x + c_y) = (12.86 \, \text{in.} + 8 \, \text{in.} + 6 \, \text{in.}) \times (12.86 \, \text{in.} + 12 \, \text{in.} + 12 \, \text{in.} + 12.86 \, \text{in.}) = 1,335.5 \, \text{in}^2 \) \( A_{\text{cr,Na}} = (2c_{Na})^2 = (2 \times 12.86 \, \text{in.})^2 = 661.5 \, \text{in}^2 \) | 17.4.5.1 | D.5.3.8 |
| \( \psi_{k,x,Na} = 0.7 + 0.3 \left( \frac{C_{\text{min}}}{c_{Na}} \right) = 0.7 + 0.3 \left( \frac{6 \, \text{in.}}{12.86 \, \text{in.}} \right) = 0.84 \) | Eq. (17.4.5.4b) |  |
| \( \psi_{k,x,Na} = 1.0 \) | 17.4.5.3 | Section 4.1.4 |

---

\( 378 \, \text{in}^2 \)

\( 8 \, \text{in.} \)

\( 1 \, \text{in.} \)

\( 378 \, \text{in}^2 \)

\( 8 \, \text{in.} \)

\( 1 \, \text{in.} \)

\( 378 \, \text{in}^2 \)

\( 8 \, \text{in.} \)

\( 1 \, \text{in.} \)

\( 378 \, \text{in}^2 \)

\( 8 \, \text{in.} \)

\( 1 \, \text{in.} \)

\( 378 \, \text{in}^2 \)

\( 8 \, \text{in.} \)

\( 1 \, \text{in.} \)
3.1.10 Design examples

Calculation per ACI 318-14 Chapter 17, ICC-ES ESR-3187*, HIT-HY 200

Note: cracked concrete conditions have been assumed. $\psi_{cp/Na} = 1.0$

Note: cracked concrete conditions have been assumed. Normal weight concrete: $\lambda_a = 1.0$

$\psi_{cp/Na} = 1.0$

Eq. (17.4.5.5a)

$\frac{1,335,5.5}{661.5.5} = 2$

Eq. (17.4.5.5b)

$17.4.5.6$

Section 4.1.4

$D.5.3.9$

Eq. (D-16f)

$V_{cpg} = (k_{cp/Na}) (MINIMUM \ [N_{cbg}; N_{ag}])$

$N_{cbg} = 74,212\ lb\quad N_{ag} = 70,087\ lb\quad h_{ef} = 15\ in.\quad k_{cp} = 2$

Bond Strength controls: $V_{cpg} = (2) (70,087\ lb) = 140,174\ lb$

Summary

Check sustained tension load

$0.55\ \Phi_{bond} N_{ba} \geq N_{u,s}$

$\Phi_{bond} = 0.65 \quad N_{ba} = 41,328\ lb/anchor \quad N_{u,s} = 1,742\ lb/anchor$

$0.55\ (0.65) (41,328\ lb/anchor) = 14,774\ lb/anchor > 1,742\ lb/anchor$

Anchors are ductile steel elements $\rightarrow$ check 17.2.3.4.3 (a) first

[Ductility check] Tension calculations per 17.2.3.4.3 (a)

<table>
<thead>
<tr>
<th>Tension</th>
<th>Nominal strength</th>
<th>Design strength</th>
<th>Factored load</th>
<th>% Utilization</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel strength</td>
<td>75,710 lb/anchor</td>
<td>90,852 lb/anchor</td>
<td>3,205 lb/anchor</td>
<td>3.5%</td>
<td>OK</td>
</tr>
<tr>
<td>Concrete breakout</td>
<td>54,991 lb</td>
<td>8,952 lb</td>
<td>16.3%</td>
<td>OK</td>
<td></td>
</tr>
<tr>
<td>Bond strength</td>
<td>44,133 lb</td>
<td>8,952 lb</td>
<td>20.3%</td>
<td>OK Controls</td>
<td></td>
</tr>
</tbody>
</table>

Steel strength does not control: 17.2.3.4.3 (a) is, therefore, not satisfied. Need to satisfy 17.2.3.4.3 (d)

Tension calculations per 17.2.3.4.3 (d)

<table>
<thead>
<tr>
<th>Tension</th>
<th>Nominal strength</th>
<th>Design strength</th>
<th>Factored load</th>
<th>% Utilization</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel strength</td>
<td>75,710 lb/anchor</td>
<td>$\phi_{steel} = 0.75$</td>
<td>4,006 lb/anchor</td>
<td>7.1%</td>
<td>OK</td>
</tr>
<tr>
<td>Concrete breakout</td>
<td>54,991 lb</td>
<td>$\phi_{concrete} = 0.65$</td>
<td>11,190 lb</td>
<td>41.7%</td>
<td>OK</td>
</tr>
<tr>
<td>Bond strength</td>
<td>44,133 lb</td>
<td>$\phi_{concrete} = 0.65$</td>
<td>11,190 lb</td>
<td>52.0%</td>
<td>OK controls</td>
</tr>
</tbody>
</table>

Shear calculations per 17.2.3.5.3 (c)

<table>
<thead>
<tr>
<th>Shear</th>
<th>Nominal strength</th>
<th>Design strength</th>
<th>Factored load</th>
<th>% Utilization</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel strength</td>
<td>31,798 lb/anchor</td>
<td>$\phi_{steel} = 0.65$</td>
<td>1,417 lb/anchor</td>
<td>6.9%</td>
<td>OK</td>
</tr>
<tr>
<td>Concrete breakout</td>
<td>23,907 lb</td>
<td>$\phi_{concrete} = 0.70$</td>
<td>8,500 lb</td>
<td>50.8%</td>
<td>OK controls</td>
</tr>
<tr>
<td>Pryout</td>
<td>140,174 lb</td>
<td>$\phi_{concrete} = 0.70$</td>
<td>8,500 lb</td>
<td>9%</td>
<td>OK</td>
</tr>
</tbody>
</table>
### 3.1 Anchor Principles and Design

#### 3.1.10 Design examples

<table>
<thead>
<tr>
<th>Calculation per ACI 318-14 Chapter 17, ICC-ES ESR-3187*, HIT-HY 200</th>
<th>ACI 318 ref.</th>
<th>ESR ref.*</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Interaction equation</strong></td>
<td>17.6</td>
<td>Section 4.1.12</td>
</tr>
<tr>
<td>Check: $V_{ua} \leq (0.2) \Phi V_{cbg}$</td>
<td>17.6.1</td>
<td>–</td>
</tr>
<tr>
<td>$V_{ua} = 8,500$ lb</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$V_{ua} &gt; (0.2) \Phi V_{cbg}$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Check: $N_{ua} \leq (0.2) \Phi N_{ag}$</td>
<td>17.6.2</td>
<td>–</td>
</tr>
<tr>
<td>$N_{ua} = 11,190$ lb</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$N_{ua} &gt; (0.2) \Phi N_{cbg}$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Use interaction equation:</td>
<td>17.6.3</td>
<td>Eq. (17.6.3)</td>
</tr>
<tr>
<td>Tri-linear: $0.520 + 0.508 = 1.028 &lt; 1.2 \rightarrow OK$</td>
<td>R17.6</td>
<td>–</td>
</tr>
<tr>
<td>Parabolic: $(0.520)^{5/3} + (0.508)^{5/3} = 0.336 + 0.323 = 0.66 &lt; 1.00 \rightarrow OK$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>This fastening satisfies the design criteria that have been assumed.</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
3.1.11 Torquing and pretensioning of anchors

Application of torque is intended to induce a tension force in the anchor bolt. It is therefore important that the torque-tension relationship associated with the anchor nut, washer and threaded anchor element be maintained as close to factory conditions as possible during anchor installation. This is best accomplished by keeping the anchor assembly in its packaging to prevent undue contamination with dust, oil, etc. prior to anchor installation. Note that damage to anchor threads as caused by attempts to re-straighten an anchor after installation, hammer impacts, etc., can significantly alter the torque-tension relationship and result in improper anchor function under load, including failure. Likewise, application of lubricants to the threads may generate excessive pretension loads in the anchor during torquing, which can also result in failure.

There are three possible reasons to apply torque to an anchor bolt in concrete or masonry:

1. To produce a clamping force, therefore eliminating gaps and play within the connected parts. Note that this clamping force is not assumed to be sufficient to permit the shear resistance of the anchorage to be determined on the basis of baseplate friction (i.e., as a slip-critical condition) owing to the relaxation of clamping forces over time.

2. To produce a pretension force in the anchor bolt which is resisted by a corresponding pre-compression in the base material (concrete or masonry). Pretension force serves to reduce anchor displacements under service load and may also serve to reduce the fatigue effects of cyclic loading.

3. To verify the anchorage will hold the tensile preload generated by the recommended torque. This helps reduce the likelihood of a grossly misinstalled anchor and/or completely unsuitable base material.

Anchor pretensioning forces dissipate over time due to relaxation in the concrete and, to a lesser degree, in the bolt threads. Re-torquing anchors can result in a higher level of residual prestress.

Anchor pretensioning should not be counted on for cases where cracking of the concrete may occur (i.e., earthquake loading).

3.1.12 Design of anchors for fatigue

The design of structural elements to resist fatigue loading can have a significant effect on the connection design. The reader is referred to relevant standards for additional information on this subject. Design of anchors for fatigue should consider the following points:

1. The application of preload to prevent stress fluctuations in the anchor rod element may be complicated by gradual loss of preload over time, particularly in cases where cracking in the base material may occur, and the fact that many anchor designs do not provide sufficient gauge length to permit the development of a meaningful degree of preload strain.

2. Design of anchor groups for fatigue is often far more critical than the design of a single anchor due to the unequal distribution of loads. Load distribution is affected by anchor slip as well as by the degree of annular gap between the anchor and the baseplate and the specific location of the anchor with respect to the hole in the baseplate. It is therefore recommended that where anchor groups are to be subjected to significant fatigue loading, the annular gap between the anchors and the baseplate be eliminated through the use of weld washers, grout, or other means.

3. Secondary flexural stresses as generated by eccentricities or gaps in the connection may be critical to the fatigue behavior of the anchor.

3.1.13 Design of anchors for fire

Building codes are generally silent on the need to design anchors specifically for fire conditions. It may be assumed, however, that structural connections to concrete or masonry involving sustained dead and live loads should be protected for fire exposure in the same manner as other structural steel elements, i.e., through the use of appropriate fireproofing materials, concrete cover, etc.

In some cases, it may be necessary to ascertain the length of time over which unprotected anchorages will survive fire exposure. The design of anchors for fire conditions is predicated on the availability of test data for the performance of anchors subjected to a standardized time-temperature curve (e.g., ASTM E 119, ISO 834) while under load.
3.1 Anchor Principles and Design

3.1.14 Design of post-installed reinforcing bar connections

Previous to this section, design of post-installed threaded rod and rebar have followed the anchoring provisions of ACI 318-14 Chapter 17 and CSA A23.3-14 Annex D. Another common and long-standing application of anchoring adhesives is the installation of deformed reinforcing bars in holes drilled in concrete to emulate the behavior of cast-in-place reinforcing bars.

This section is a supplement to the Hilti North America Post-Installed Reinforcing Bar Guide and is an alternative to considering the post-installed rebar as an “anchor”. Refer to the Guide for a comprehensive description of post-installed reinforcing bar design with Hilti adhesive anchor systems. Contact Hilti Technical Services with questions.

Adhesive anchor systems are qualified in accordance with ICC-ES Acceptance Criteria for Post-Installed Adhesive Anchors in Concrete Elements (AC308). Hilti HIT-RE 500 V3 and HIT-HY 200 adhesives are recognized for use with post-installed reinforcing bars in ICC-ES Evaluation Service Reports ESR-3814 and ESR-3187. Based on these recognitions, reinforcing bars installed with HIT-RE 500 V3 and HIT-HY 200 may now be designed using two methods:

1. Development and splice length provisions in ACI 318-14 (Chapters 18 and 25) and CSA A23.3-14 (Chapters 12 and 21)
2. Anchoring to concrete provisions in ACI 318-14 Chapter 17 and CSA A23.3-14 Annex D.

Within this section, development and splice lengths are provided according to ACI 318-14 Chapter 25 and CSA A23.3-14 Chapter 12 calculations (see item 1 above). In addition, embedment depths provided for anchorage calculations correspond to development of reinforcing bars following an approach outlined in a paper published in the ACI Structural Journal (see item 2 above).

Post-installed reinforcing bar installations in accordance with ACI 318-14 and CSA A23.3-14 can also be designed using Hilti’s PROFIS Anchor and PROFIS Rebar software. You can access PROFIS Anchor and PROFIS Rebar at www.us.hilti.com in the U.S., and at www.hilti.ca in Canada.

3.1.14.1 Development and splicing using ACI 318-14 Chapter 25 provisions

ACI 318-14 Chapter 25 contains provisions for reinforcing bar development and splice lengths in non-seismic applications. Development lengths are assumed to preclude concrete splitting and reinforcing bar pullout failure prior to “development” (attainment) of bar yield stress. Although the term “lap splice” implies direct transfer of stress from bar to bar, forces between bars are transferred via struts and hoop stresses in the concrete. The ICC-ES acceptance criteria for adhesive anchors in concrete, AC308, now includes procedures and requirements for the recognition of post-installed designed reinforcing using the development length provisions of ACI 318-14 Chapters 18 and 25. Hilti HIT-RE 500 V3 and HIT-HY 200 are recognized in ICC-ES ESR-3814 and ESR-3187, respectively, for this purpose.

Splicing of post-installed reinforcing bars

Tension development length: ACI 318-14 25.4.2.3

Under the conditions given in ESR-3814 and ESR-3187 as revised in July 2015, design of post-installed reinforcing bars with HIT-RE 500 V3 and HIT-HY 200, respectively, may be performed using the applicable provisions of ACI 318-14 Chapters 18 and 25. The basic expression for tension development length in Chapter 25 is provided in the following equation as:

\[
l_d = \left[ \frac{3}{40\lambda} f_c \psi_t \frac{\psi_s \psi_c}{(c_0 + k_b/d_b)} \right] d_b
\]

where the confinement term \((c_0 + k_b/d_b)\) shall not be taken as greater than 2.5 and the design value of \(l_d\) shall not be less than 12 inches per 25.4.2.1.

Note: Because the “top bar” factor, \(\psi_t\), accounts for bar position effects in freshly poured concrete, it may be neglected for the drilled-in portion of post-installed bars. \(\psi_t\) must be applied where applicable for the freshly cast-in portion of new bars and for the spliced portions of existing cast-in-place bars.
Tension lap splices: ACI 318-14 25.5.2

A Class B splice taken as the greater of $1.3\ell_d$ and 12 inches is required in all cases unless 1) the area of reinforcement provided is at least twice that determined by analysis over the entire length of the splice and 2) one-half or less of the total reinforcement is spliced within the lap length. Where 1) and 2) are satisfied, a Class A splice taken as the greater of $1.0\ell_d$ and 12 inches may be used.

Table 89 in Sec. 3.2.3, and Table 83 in Sec. 3.2.4 provide a summary of calculated development and splice lengths for a range of concrete strengths for HIT-HY 200 and HIT-RE 500 V3 for the specific case where the confinement term $(c_b + K_{tr})/d_b$ has been taken as the maximum value of 2.5. Refer to Chapter 6 of the Post-Installed Reinforcing Bar Guide and ESRs for additional design information relating to development length and lap splices.

3.1.14.2 Development and splicing using CSA A23.3-14 Chapter 12 provisions

CSA A23.3-14 Chapter 12 contains provisions for reinforcing bar development and splice lengths in non-seismic applications analogous to those of ACI 318-14. While not formally recognized in ICC-ES ESRs, Hilti HIT-RE 500 V3 and HIT-HY 200 is commonly used with the provisions of CSA A23.3-14 based on the testing performed in accordance with ICC-ES AC308. As with the design for ACI 318-14 Chapter 25, design of post-installed reinforcing bars with HIT-RE 500 V3 and HIT-HY 200 may be performed equivalently to cast-in reinforcing bars using the applicable equations in CSA A23.3-14. The basic expression for tension development length is provided in the equation as follows:

\[ \ell_d = 1.15 \frac{k_1 k_2 k_3 k_4 f_y}{(d_{cs} + K_{tr}) \sqrt{f_c}} A_b \]

where the confinement term $(d_{cs} + K_{tr})$ shall not be taken greater than 2.5 and the design value of $\ell_d$ shall not be less than 300 mm per 12.2.1.

Note: Because the "top bar" factor, $k_1$, accounts for bar position effects in freshly poured concrete, it may be neglected for the drilled-in portion of post-installed bars. $k_1$ must be applied where applicable for the freshly cast-in portion of new bars and for the spliced portions of existing cast-in-place reinforcement.

Tension lap splices: CSA A23.3-14 12.15.1

A Class B splice taken as the greater of $1.3\ell_d$ and 300 mm is required in all cases unless 1) the area of reinforcement provided is at least twice that determined by analysis over the entire length of the splice and 2) one-half or less of the total reinforcement is spliced within the lap length. Where 1) and 2) are satisfied, a Class A splice taken as the greater of $1.0\ell_d$ and 300 mm may be used.

Table 94 in Sec. 3.2.3, and Table 88 in Sec. 3.2.4 provides a summary of development and splice lengths for a range of concrete strengths for HIT-HY 200 and HIT-RE 500 V3 according to CSA provisions for the specific case where the confinement term $(d_{cs} + K_{tr})$ has been taken as the maximum value of $2.5d_s$. Refer to Chapter 6 of the Post-Installed Reinforcing Bar Guide and ESRs for additional design information relating to development length and lap splices.
3.1 Anchor Principles and Design

Design example — development lengths for negative reinforcement slab extension

Requirement: Establish the embedment for post-installed reinforcing bars for a slab extension as shown in the figure below.

![Diagram of slab extension](image)

New-to-existing slab connection

Step 1: Parameters

Existing construction (E) slab, 8-inch thick, 4000 psi normal weight concrete, Gr. 60 reinforcement, #6 bars at 12-inch on-center spacing. Note: other detailing not shown.

New construction (N) slab, 8-inch thick, 5000 psi normal weight concrete, Gr. 60 reinforcement, #6 bars at 12-inch on-center spacing.

Step 2: Determine the development length for a Class B splice

Assuming $K_t$ is equal to zero (i.e., no transverse reinforcing present to restrain splitting), with 12-inch spacing and 1.5-inch cover, $c_b = (1.5 + 0.625/2) = 1.875$ inches and $(c_b + K_t)/d_b = (1.875 + 0)/0.625 = 2.5$. With $(c_b + K_t)/d_b \geq 2.5$ and less than 12 inches of concrete in the slab member installed below existing bars, Table 89 in Sec. 3.2.4, applies to both existing and new reinforcement. From Table 83 in Sec. 3.2.4, the Class B splice length of a #6 bar in 4000 psi concrete is 22 inches.

Per ACI 318-14 25.5.1.3, the distance between a post-installed reinforcing bar and an existing cast-in-place reinforcing bar to which the post-installed bar is spliced shall be no greater than the lesser of 6 in. and one-fifth of the development length. For this example $22/5 = 4.5$ in.

Step 3: Specification

Install #6 Gr. 60 reinforcing bars at 12 inches on center with a minimum 24-inch embedment (22-inch splice plus 2-inch end cover) using Hilti HIT-RE 500 V3 as shown in the figure above. Locate post-installed bars within 4-1/2 in. of existing bars to be spliced. Install in accordance with Hilti instructions for Use. Do not damage existing reinforcing. Roughen interface to 1/4-inch amplitude prior to placement of post-installed bars.

Note: ACI 318-14 and ICC Evaluation Service Reports refer to the manufacturer instructions for use as the Manufacturer’s Published Installation Instructions (MPII).

3.1.14.3 Development of post-installed reinforcing bars based on ACI 318-14 Chapter 17 and CSA A23.3-14 Annex D anchorage provisions

ACI 318-14 Chapter 17 and CSA A23.3 Annex D contain design provisions for the determination of the tensile strength of post-installed adhesive anchors in concrete, whereby the strength is taken as the minimum of the resistances corresponding to steel rupture, concrete breakout, and bond failure at the adhesive-to-concrete interface. Since the establishment of development length is based on the assumption of the attainment of a minimum strength corresponding to yield of the bar, the design equations for anchorage can also be applied to this problem. Within the May-June 2013 issue of the ACI Structural Journal, “Recommended Procedures for Development and Splicing of Post-Installed Bonded Reinforcing Bars in Concrete Structures” by Charney, Pal and Silva provides a methodology for establishing the bar embedment to develop the bar using the concepts of anchorage contained in ACI 318-14 Chapter 17. This methodology is similarly applicable to CSA A23.3 Annex D.

Note: This procedure is not addressed in ACI 318-14 or in CSA A23.3. As stated in Charney, et al., the assumptions made to ensure bar yield are a matter of judgment and may require unique determination for specific applications and conditions. For specific cases, contact Hilti.

The use of bond values corresponding to the assumption of uncracked vs. cracked concrete for the design of reinforcing bar embedment is a matter of judgment. The ACI 318-14 Chapter 25 development length provisions do not explicitly consider reduction of bond corresponding to cracking of the concrete that may arise in conjunction with structure loading, shrinkage, etc. Furthermore, while the development length provisions of the code appear to consider only the nominal yield strength of the bar, it may be prudent to provide sufficient embedment to develop the actual bar yield, generally assumed to be 125% of the nominal value. Tables 90 and 95 in Sec. 3.2.3, and Tables 84 and 89 in Sec. 3.2.4 provide calculated embedments to develop Gr. 60 rebar based on the application of anchor theory. For the development of these Tables, the strengths corresponding to the applicable limit states for single bars in tension for non-seismic applications (i.e., SDC A and B) in ACI 318-14 Chapter 17 are taken as follows:
Section through a shotcrete onlay wall with post-installed dowels designed using anchor provisions whereby the concrete breakout and bond strengths with reduction factors applied are each set equal to the assumed bar yield strength. In CSA calculations, R for breakout and bond calculations is conservatively assumed to equal 1.0. The resulting expressions are solved for \( h_{ef} \) as follows:

As governed by single-bar concrete breakout strength:

\[
A_b \cdot 1.25 f_y = \phi_b k_{c,uncr} \sqrt{f_c} h_{ef,breakout}^{1.5}
\]

\[
h_{ef,breakout} = \left( \frac{A_b \cdot 1.25 f_y}{\phi_b k_{c,uncr} \sqrt{f_c}} \right)^{2/3}
\]

with minimum \( c_0 \) and \( s \) intended to preclude edge and spacing effects.

As governed by single-bar bond strength:

\[
A_{se,\chi} \cdot 1.25 f_y = \phi_b k_{c,uncr} \phi_d h_{ef,bond}
\]

\[
h_{ef,bond} = \frac{A_{se,\chi} \cdot 1.25 f_y}{\phi_b k_{c,uncr} \phi_d}
\]

with minimum \( c_0 \) and \( s \) intended to preclude edge and spacing effects.

The controlling embedment (in this case, the larger value of \( h_{ef,breakout} \) and \( h_{ef,bond} \)), is reported in the table together with the accompanying edge distances and spacing values.

**Note:** ACI 318-14 17.3.2.3 states: “For adhesive anchors with embedment depths \( 4d_s \leq h_w \leq 20d_s \) the bond strength requirements shall be considered satisfied by the design procedure of 17.4.6.” In accordance with 17.3.2.3, ESR-3814 and ESR-3187 limit anchorage embedment depths to this range of values. These requirements recognize the limits of the uniform bond model adopted by ACI.

Anchorages embedded in the Hilti North American Product Technical Guide are based on assumptions that are intended to achieve development of the nominal yield stress in reinforcing bars. In some cases, the recommended bar embedment exceeds, by an acceptable margin, the 20 bar diameter limit established in ACI 318-14 Chapter 17 for the applicability of the uniform bond model. It is Hilti’s view that the conservatism of the underlying assumptions, taken in aggregate, is sufficient to offset any reduction in the effective bond stress associated with these bond lengths. The designer may elect to employ alternate design assumptions for bar development based on the specific conditions for a given design.

**Spacing:** To account for the influence of nearby bars on the concrete resistance (breakout/bond failure), minimum spacing is calculated as the greater of \( 20d_b \left( \tau_{k,uncr}/1100 \text{ psi} \right)^{0.5} \) and \( 3h_{ef,breakout} \).

**Edge distance:** To account for the effect of edge distance on the concrete resistance, minimum edge distance is calculated as the greater of \( 10d_b \left( \tau_{k,uncr}/1100 \text{ psi} \right)^{0.5} \) and \( 1.5h_{ef,breakout} \).

In addition, for uncracked concrete, the minimum edge distance may be governed by the value of \( c_{ac} \), the critical edge distance for splitting failure where the design assumes uncracked concrete and where there is no reinforcing to control splitting cracks.

In this case, the value of \( c_{ac} \) is taken as:

\[
c_{ac} = h_{ef} \left( \frac{\tau_{k,uncr}}{1160} \right)^{0.4} \left( 3.1 - 0.7 \frac{h}{h_{ef}} \right)
\]

where \( h \leq 2.4 \) and \( h_{ef} \).

\( \tau_{k,uncr} \) is the characteristic bond strength stated in the Evaluation Service Report and cannot be taken as larger than:

\[
k_{c,uncr} \frac{h_{ef}}{\sqrt{f_c}} \tau_{k,uncr} = \frac{\pi d}{k_{c,uncr}}
\]

The application of \( c_{ac} \) to post-installed reinforcing bars in uncracked concrete is advisable where the EOR determines that splitting will be critical for the behavior of the connection at ultimate loads. The figure below represents two possible extremes for the calculation of edge distance. Condition I illustrates a bar anchored in a relatively thin and lightly reinforced slab. In this case, splitting is likely to be critical and the application of \( c_{ac} \) as calculated in the equation above is advisable. At the other extreme, Condition II is represented by bars embedded in a heavily reinforced foundation where the ratio of \( h \) to \( h_{ef} \) is large as judged by the EOR. In Condition II, splitting is unlikely to control the behavior and the \( c_{ac} \) term may be neglected. In all cases, proper judgment based on the loading and geometry of the connection should be applied.
3.1 Anchor Principles and Design

Note: Tables 90 and 95 in Sec. 3.2.3, and Tables 84 and 89 in Sec. 3.2.4 provide suggested embedment depths, edge distances, and spacing values for the development of Grade 60 reinforcing bars. Bond strengths have not been reduced for seismic loading and a bar overstrength factor of 125% of nominal yield has been applied in the determination of the values provided. Bond strengths include reductions for sustained tension loading as provided in the applicable evaluation reports; however, no additional reduction in accordance with ACI 318-14 17.3.1.2 has been included. Where bars are used to resist sustained tension loading, increases in the tabulated embedment values may be appropriate. Consult Hilti Technical Services for further information.

3.1.14.4 Development of post-installed wall/column starter bars in a linear array based on ACI 318-14 Chapter 17 and CSA A23.3-14 Annex D anchorage provisions

Note: This procedure is not addressed in ACI 318-14 or in CSA A23.3. For additional information see May-June 2013 issue of the ACI Structural Journal, “Recommended Procedures for Development and Splicing of Post-Installed Bonded Reinforcing Bars in Concrete Structures” by Charney, Pal and Silva. As addressed in this study, the assumptions made to ensure bar yield are a matter of judgment and may require unique determination for specific applications and conditions. For specific cases, contact Hilti.

In Tables 91 through 93 and 96 through 98 in Sec. 3.2.3, and Tables 85 through 87 and 90 through 92 in Sec. 3.2.4, the expressions presented in the study have been expanded to include the effects of set spacing of starter bars in a linear array at set spacings. To produce these tables ANc is defined by ACI 318-14, 17.4.2.1 and Fig. R17.4.2.1 (CSA A23.3 D.6.2.1 and Fig. D.7). ANs is defined by ACI 318-14, 17.4.5.1 and Fig. R17.4.5.1 (CSA A23.3 D.6.5.1 and Fig. D.11). ANco and ANao are defined by Equations (17.4.2.1c) and (17.4.5.1c) (CSA Equations (D.5) and (D.22)), respectively. Inclusion of these terms permits the effects of edge distance to be considered on the concrete breakout and bond strengths. Using the relationships above, the equations below for hef for both breakout and bond can be found, the larger of which is taken as the final embedment depth.

\[
N_{\text{steel,m}} = n_{\text{bars}} \bar{A}_b (1.25 f_y)
\]
\[
N_{\text{breakout,m}} = \left( \frac{A_{\text{nc}}}{A_{\text{nc}} + A_{\text{ns}}} \right) \phi_{c,\text{uncr}} \sqrt{f'_c} h_{\text{ef,breakout}}^{1.5}
\]
\[
N_{\text{bond,m}} = \left( \frac{A_{\text{na}}}{A_{\text{na}} + A_{\text{ns}}} \right) \phi_{b,\text{t,uncr}} \frac{\pi d}{n_d} h_{\text{ef,bond}}
\]

whereby the concrete breakout and bond strengths with reduction factors applied are each set equal to the assumed bar yield strength and the resulting expressions are solved for hef as follows: As governed by multiple-bar breakout strength:

\[
h_{\text{ef,breakout}} = \left( \frac{1}{n_{\text{bars}} \bar{A}_b (1.25 f_y)} \right)^{2/3}
\]

with minimum ca intended to preclude edge effects.

Note: The term \((A_{\text{nc}} / A_{\text{ns}})\) depends on hef. For this reason, the calculation of hef using the equation above may require iteration.
As governed by multiple-bar bond strength:

\[ h_{ef,bond,m} = \frac{1}{\phi_{\text{bar}}} \left( \frac{A_{\text{bars}}}{A_{\text{nc}}} \right) (1.25 f_y) \]

with minimum \( c_a \) intended to preclude edge effects.

In Equations (6) and (7), \( \left( \frac{A_{\text{bars}}}{A_{\text{nc}}} \right) \) and \( \left( \frac{A_{\text{bars}}}{A_{\text{nc}}} \right) \) shall not be taken as less than 1.0.

**Note:** Anchorage embeddings published in the Hilti North American Product Technical Guide are based on assumptions that are intended to achieve development of 125% of the nominal yield stress in reinforcing bars. In some cases, the recommended bar embedment exceeds the 20 bar diameter limit established in ACI 318-14 Chapter 17 for the applicability of the uniform bond model. It is Hilti’s view that the conservatism of the underlying assumptions, taken in aggregate, is sufficient to offset any reduction in the effective bond stress associated with these bond lengths. The designer may elect to employ alternate design assumptions for bar development based on the specific conditions for a given design.

**Edge distance:** To account for the effect of edge distance on the concrete resistance, minimum edge distance is calculated as the greater of \( 10d_b \left( \frac{f_{\text{c}}}{1100 \text{ psi}} \right)^{0.5} \) and \( 1.5h_{\text{et,breakout}} \).

In addition, for uncracked concrete, the minimum edge distance may be governed by the value of \( c_{\text{ac}} \), the critical edge distance for splitting failure, where the design assumes uncracked concrete and there is no reinforcing to control splitting cracks.

**General assumptions:** Embedment depths in Tables 91 through 93 and 96 through 98 in Sec. 3.2.3, and Tables 85 through 87 and 92 in Sec. 3.2.4, are predicated on the assumptions that 1) edge distances are no less than \( c_{\text{ac}} \) and \( c_{\text{ac}} \) as defined by ACI 318-14 Chapter 17 and AC308, respectively, 2) all bars in the group are loaded equally, 3) uncracked concrete conditions apply, 4) that orthogonal bars are spaced far enough away to preclude edge and spacing effects, 5) bars are NOT subject to sustained tension, and 6) that the number of bars in a linear array, \( n_{\text{bars}} \), is equal to 10.

**Note:** Tables 91 through 93 and 96 through 98 in Sec. 3.2.3, and Tables 85 through 87 and 92 in Sec. 3.2.4, provide calculated embedment depths, edge distances, and spacing values corresponding to the stated assumptions that are intended to develop 125% of nominal yield in Grade 60 reinforcing bars under non-earthquake conditions (i.e., SDC A and B). The use of other assumptions (e.g., bond values corresponding to cracked concrete or seismic loading, omission of the bar overstrength multiplier, etc.) will result in different embedment depths, spacings, and edge distances that may be appropriate for specific design conditions. Consult Hilti Technical Services for further information.

**Design example — development of wall-to-slab starter bars in linear 24-inch-on-center array using anchor theory**

**Requirement:** Provide post-installed starter bars for a new wall on an existing lightly reinforced slab-on-grade as shown in the figure below installed with HIT-RE 500-V3. Analysis indicates that there will be no cracking during service loading and no sustained tensile loads on the reinforcing bars.

**Step 1: Establish requirements for the new bars**

Existing construction  (E) 12-inch thick foundation, 6000 psi normal weight concrete, Gr. 60 reinforcement.

New construction  (N) 10-inch wide wall as shown, 5000 psi normal weight concrete, Gr. 60 reinforcement, #5 bars at 24-inch on-center spacing.

**Step 2: Determine the development length for the wall starter bars using anchor theory**

From Table 85 in Sec. 3.2.4, a #5 bar with \( f_{\text{c}} = 6000 \text{ psi} \) produces \( h_{ef} = 9 \text{ inches} \) with \( c_{\text{ac}} \geq 25 \text{ inches} \).

**Step 3: Specification**

Gr. 60 #5 reinforcing bars installed at an 9-inch embedment with Hilti HIT-RE 500 V3 at 24-inch O.C. no less than 25 inches from all edges. Install per Hilti Instructions for Use. Do not damage existing reinforcing. Roughen interface to 1/4-inch amplitude prior to placement of post-installed bars.

**Note:** ACI 318-14 and ICC Evaluation Service Reports refer to the manufacturer instructions for use as the Manufacturer’s Published Installation Instructions (MPII).