

PART VI
Appendix K
Anchoring to Concrete

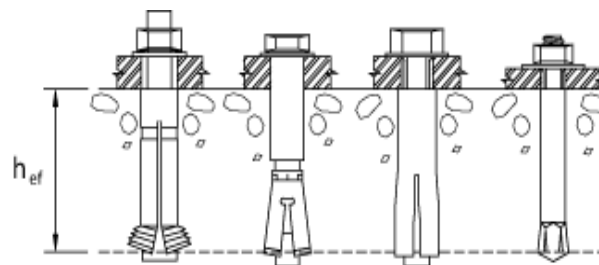
K.1 Definitions

ANCHOR	A steel element used to transmit applied loads either by casting into concrete or post-installed into a hardened concrete member. An anchor includes headed bolts, hooked bolts (J- or L-bolt), headed studs, expansion anchors, or undercut anchors.
ANCHOR GROUP	Anchor group is formed by a number of anchors of approximately equal effective embedment depth with each anchor spaced at less than $3h_{ef}$ from one or more adjacent anchors when subjected to tension, or $3c_{a1}$ from one or more adjacent anchors when subjected to shear. Only those anchors susceptible to the particular failure mode under investigation shall be included in the group.
CLARIFICATION FOR ANCHOR GROUP	Only those anchors susceptible to a particular failure mode out of all potential modes (steel, concrete breakout, pullout, side-face blowout, and pryout) should be considered when evaluating the strength associated with that failure mode.
ANCHOR PULLOUT STRENGTH	The strength corresponding to the anchoring device or a major component of the device sliding out from the concrete without breaking out a substantial portion of the surrounding concrete.
ANCHOR REINFORCEMENT	It is the reinforcement used to transfer the full design load from the anchors into the structural member. See Sec K.5.2.9 or Sec K.6.2.9.
CLARIFICATION FOR ANCHOR REINFORCEMENT	The design and detailing of anchor reinforcement is done specifically for the purpose of transferring anchor loads from the anchors into the structural member. Hairpins are generally used for this purpose (see Clarification for Sections K.5.2.9 and K.6.2.9); however, other configurations that can be shown to effectively transfer the anchor load are acceptable.
ATTACHMENT	The structural assembly, external to the surface of the concrete, that transmits loads to or receives loads from the anchor.

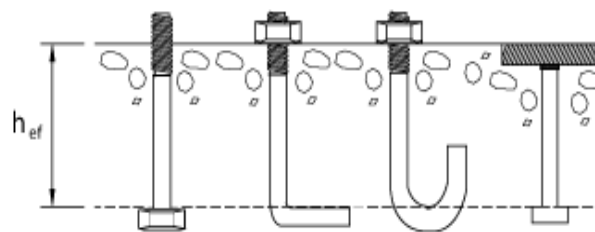
BRITTLE STEEL ELEMENT	An element having a tensile test elongation of less than 14 percent, or reduction in area of less than 30 percent, or both.
CLARIFICATION FOR BRITTLE STEEL ELEMENT AND DUCTILE STEEL ELEMENT	The gauge length specified in the appropriate ASTM standard for the steel shall be used for measuring the 14 percent elongation.
CAST-IN ANCHOR	An anchor either a headed bolt, headed stud, or hooked bolt installed before placing concrete.
CONCRETE BREAKOUT STRENGTH	The strength corresponding to which a volume of concrete surrounding the anchor or group of anchors separates from the member.
CONCRETE PRYOUT STRENGTH	The strength corresponding to formation of a concrete spall behind short, stiff anchors displaced in the direction opposite to the applied shear force.
DISTANCE SLEEVE	A sleeve that encases the center part of an undercut anchor, a torque-controlled expansion anchor, or a displacement-controlled expansion anchor without expanding.
DUCTILE STEEL ELEMENT	An element with a tensile test elongation of not less than 14 percent and reduction in area of at least 30 percent. A steel element meeting the requirements of ASTM A307 shall be considered ductile.
EDGE DISTANCE	The distance to the edge of the concrete surface from center of the anchor closest to edge.
EFFECTIVE EMBEDMENT DEPTH	It 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.
CLARIFICATION FOR EFFECTIVE EMBEDMENT DEPTH	Figure 6.K.1 illustrates the effective embedment depths for a variety of anchor types.

EXPANSION ANCHOR	It is a post-installed anchor, inserted into hardened concrete and it transfers loads to or from the concrete by direct bearing or friction or both. Expansion anchors may be torque-controlled, where the expansion is achieved by a torque acting on the screw or bolt; or displacement-controlled, where the expansion is achieved by impact forces acting on a sleeve or plug and the expansion is controlled by the length of travel of the sleeve or plug.
EXPANSION SLEEVE	The outer part of an expansion anchor that is forced outward by the center part, either by applied torque or impact, to bear against the sides of the predrilled hole.
FIVE PERCENT FRACTILE	Statistically it means 90 percent confidence that there is 95 percent probability of the actual strength exceeding the nominal strength.
CLARIFICATION FOR FIVE PERCENT FRACTILE	The coefficient K_{05} associated with the 5 percent fractile, $X_m - K_{05}S_s$ is determined depending on the number of tests, n , used to compute the sample mean, X_m , and sample standard deviation, S_s . Values of K_{05} range, for example, from 1.645 for $n = \infty$, to 2.010 for $n = 40$, and 2.568 for $n = 10$. With this definition of the 5 percent fractile, the nominal strength in K.4.2 is the same as the characteristic strength in ACI 355.2.
HEADED STUD	A steel anchor conforming to the requirements of AWS D1.1 and affixed to a plate or similar steel attachment by the stud arc welding process before casting.
HOKED BOLT	A cast-in anchor, which is anchored mainly by bearing of the 90° bend (L-bolt) or 180° bend (J-bolt) against the concrete, at its embedded end, and having a minimum e_h of $3d_a$.
POST-INSTALLED ANCHOR	An anchor installed in concrete, which is hardened. Expansion anchors and undercut anchors are examples of post-installed anchors.
PROJECTED AREA	The area on the free surface of the concrete member that is used to represent the larger base of the assumed rectilinear failure surface.
SIDE-FACE BLOWOUT STRENGTH	The strength of anchors with deeper embedment but thinner side cover corresponding to concrete spalling on the side face around the embedded head while no major breakout occurs at the top concrete surface.

SPECIALTY INSERT	Predesigned and prefabricated cast-in anchors specifically designed for attachment of bolted or slotted connections. Specialty inserts are often used for handling, transportation, and erection, but are also used for anchoring structural elements. Specialty inserts are not within the scope of this Appendix.
SUPPLEMENTARY REINFORCEMENT	Reinforcement which acts to restrain potential concrete breakout which is not designed to transfer full design load from the anchors into the structural member.
CLARIFICATION FOR SUPPLEMENTARY REINFORCEMENT	Supplementary reinforcement has a configuration and placement similar to anchor reinforcement but is not specifically designed to transfer loads from the anchors into the structural member. Stirrups for shear reinforcement may fall into this category.
UNDERCUT ANCHOR	It is a post-installed anchor that develops its tensile strength from the mechanical interlock provided by undercutting of the concrete at the embedded end of the anchor. The undercutting is achieved with a special drill before installing the anchor or alternatively by the anchor itself during its installation.



(a) Post-installed anchors



(b) Cast-in-place anchors

Figure 6.K.1 Types of anchors.

K.2 Scope

K.2.1 In this Appendix design requirements are provided for anchors in concrete used to transmit structural loads by means of tension, shear, or a combination of tension and shear between: (a) connected structural elements; or (b) safety-related attachments and structural elements. Safety levels specified are intended for in-service conditions, rather than for short-term handling and construction conditions.

Clarification for Section K.2.1: The scope of Appendix K is restricted to structural anchors that transmit structural loads related to strength, stability, or life safety. Two types of applications are envisioned. The first one is connections between structural elements where the failure of an anchor or an anchor group could result in loss of equilibrium or stability of any portion of the structure. The second one is where safety-related attachments that are not part of the structure (such as sprinkler systems, heavy suspended pipes, or barrier rails) are attached to structural elements. The levels of safety defined by the combinations of load factors and ϕ factors are appropriate for structural applications. Other standards may require more stringent safety levels during temporary handling.

K.2.2 This Appendix is applicable to both cast-in anchors and post-installed anchors. Specialty inserts, through bolts, multiple anchors connected to a single steel plate at the embedded end of the anchors, adhesive or grouted anchors, and direct anchors such as powder or pneumatic actuated nails or bolts, are not included. Reinforcement used as part of the embedment shall be designed in accordance with other parts of this Code.

Clarification for Section K.2.2: It is difficult to prescribe generalized tests and design equations for many insert types because of the wide variety of shapes and configurations of specialty inserts. Hence, they have been excluded from the scope of Appendix K. Adhesive anchors are widely used and can perform adequately. However, such anchors are outside the scope of this Appendix at this time.

K.2.3 Headed bolts and headed studs having a geometry that has been demonstrated to result in a pullout strength in uncracked concrete equal or exceeding $1.4N_p$ (where N_p is given by Eq. 6.K.15) are included. Hooked bolts that have a geometry that has been demonstrated to result in a pullout strength without the benefit of friction in uncracked concrete equal or exceeding $1.4N_p$ (where N_p is given by Eq. 6.K.16) are included. Post-installed anchors that meet the assessment requirements of ACI 355.2 are included. The suitability of the post-installed anchor for use in concrete shall have been demonstrated by the ACI 355.2 prequalification tests.

Clarification for Section K.2.3: Typical cast-in headed bolts and headed studs with geometries consistent with ANSI/ASME B1.1,^{K.1} B18.2.1,^{K.2} and B18.2.6^{K.3} have been tested and proven to behave predictably, so calculated pullout values are acceptable. Post-installed anchors do not have predictable pullout capacities, and therefore are required to be tested. For a post-installed anchor to be used in conjunction with the requirements of this Appendix, the results of the ACI 355.2 tests have to indicate that pullout failures exhibit an acceptable load-displacement characteristic or that pullout failures are precluded by another failure mode.

K.2.4 This Appendix does not cover the load applications that are predominantly high cycle fatigue or impact loads.

Clarification for Section K.2.4: The exclusion of load applications producing high cycle fatigue or extremely short duration impact (such as blast or shock wave), however, does not mean that seismic load effects are excluded from the scope. K.3.3 presents additional requirements for design when seismic loads are included.

K.3 General Requirements

K.3.1 Anchors and anchor groups shall be designed for critical effects of factored loads determined through elastic analysis. Plastic analysis approaches are permitted where nominal strength is controlled by ductile steel elements, provided that deformational compatibility is taken into account.

Clarification for Section K.3.1: If the strength of an anchor group is governed by breakage of the concrete, the behavior is brittle and there is limited redistribution of the forces between the highly stressed and less stressed anchors. In this case, the theory of elasticity is required to be used assuming the attachment that distributes loads to the anchors is sufficiently stiff. The forces in the anchors are considered to be proportional to the external load and its distance from the neutral axis of the anchor group.

If anchor strength is governed by ductile yielding of the anchor steel, significant redistribution of anchor forces can occur. In this case, an analysis based on the theory of elasticity will be conservative. References K.4 to K.6 discuss nonlinear analysis, using theory of plasticity, for the determination of the capacities of ductile anchor groups.

K.3.2 The design strength of anchors shall equal or exceed the largest required strength calculated from the applicable load combinations in Sec 6.2.2.

K.3.3 When anchor design includes earthquake forces for structures assigned to Seismic Design Category C, or D, the additional requirements of Sections K.3.3.1 to K.3.3.6 shall apply.

Clarification for Section K.3.3: Post-installed structural anchors are required to be qualified for Seismic Design Categories C, or D, by demonstrating the ability to undergo large displacements through several cycles as specified in the seismic simulation tests of ACI 355.2. Because ACI 355.2 excludes plastic hinge zones, Appendix K is not applicable to the design of anchors in plastic hinge zones under seismic forces. In addition, the design of anchors for earthquake forces is based on a more conservative approach by the introduction of 0.75 factor on the design strength ϕN_n and ϕV_n for the concrete failure modes, and by requiring the system to have adequate ductility. Anchor strength should be governed by ductile yielding of a steel element. If the anchor cannot meet these ductility requirements, then either the attachment is designed to yield or the calculated anchor strength is substantially reduced to minimize the possibility of a brittle failure. In designing attachments for adequate ductility, the ratio of yield to design strength should be considered. A connection element could yield only to result in a secondary failure as one or more elements strain harden and fail if the design strength is excessive when compared to the yield strength.

The full shear force should be assumed in any direction for a safe design as the direction of shear may not be predictable under seismic conditions.

K.3.3.1 The provisions of Appendix K are not applicable to the design of anchors in plastic hinge zones of concrete structures under earthquake forces.

Clarification for Section K.3.3.1: Section 3.1 of ACI 355.2 specifically states that the seismic test procedures do not simulate the behavior of anchors in plastic hinge zones. The possible higher level of cracking and spalling in plastic hinge zones are beyond the damage states for which Appendix K is applicable.

K.3.3.2 Post-installed structural anchors shall be qualified for use in cracked concrete and shall have passed the Simulated Seismic Tests in accordance with ACI 355.2. Pullout strength N_p and steel strength of the anchor in shear V_{sa} shall be based on the results of the ACI 355.2 Simulated Seismic Tests.

Clarification for Section K.3.3.2: Anchors that are not suitable for use in cracked concrete should not be used to resist seismic loads.

K.3.3.3 The design strength of anchors associated with concrete failure modes shall be taken as $0.75\phi N_n$ and $0.75\phi V_n$, where ϕ is given in K.4.4, and N_n and V_n are determined in accordance with Sections K.5.2, K.5.3, K.5.4, K.6.2, and K.6.3, assuming the concrete is cracked unless it can be demonstrated that the concrete remains uncracked.

Clarification for Section K.3.3.3: The anchor strength associated with concrete failure modes is to account for increased damage states in the concrete resulting from seismic actions. Because seismic design generally assumes that all or portions of the structure are loaded beyond yield, it is likely that the concrete is cracked throughout for the purpose of determining the anchor strength unless it can be demonstrated that the concrete remains uncracked.

K.3.3.4 Anchors shall be designed to be governed by the steel strength of a ductile steel element as determined in accordance with Sections K.5.1 and K.6.1, unless either Sec K.3.3.5 or Sec K.3.3.6 is satisfied.

Clarification for Section K.3.3.4: Ductile steel anchor elements are required to satisfy the requirements of K.1, Ductile Steel Element. For anchors loaded with a combination of tension and shear, the strength in all loading directions must be controlled by the steel strength of the ductile steel anchor element.

K.3.3.5 Instead of K.3.3.4, the attachment that the anchor is connecting to the structure shall be designed so that the attachment will undergo ductile yielding at a force level corresponding to anchor forces no greater than the design strength of anchors specified in K.3.3.3.

K.3.3.6 Alternative to K.3.3.4 and K.3.3.5, it shall be allowed to take the design strength of the anchors as 0.4 times the design strength determined in accordance with K.3.3.3. For the anchors of stud bearing walls, it shall be allowed to take the design strength of the anchors as 0.5 times the design strength determined in accordance with K.3.3.3.

Clarification for Section K.3.3.6: As a matter of desirable practice, a ductile failure mode in accordance with K.3.3.4 or K.3.3.5 should be provided for in the design of the anchor or the load should be transferred to anchor reinforcement in the concrete. Where geometric or material constraints do not permit, K.3.3.6 allows the design of anchors for nonductile failure modes at a reduced permissible strength to minimize the possibility of a brittle failure. The attachment of light frame stud walls typically involves multiple anchors that allow for load redistribution. This justifies the use of a less conservative factor for this case.

K.3.4 In this Appendix, modification factor λ for lightweight concrete shall be in accordance with Sec 6.1.8.1 unless specifically noted otherwise.

K.3.5 The values of f'_c used for calculation purposes in this Appendix shall not be greater than 70 MPa for cast-in anchors, and 55 MPa for post-installed anchors. Testing is required for post-installed anchors when used in concrete with f'_c exceeding 55 MPa.

Clarification for Section K.3.5: Limited tests of cast-in-place and post-installed anchors in high-strength concrete^{K.7} indicate that the design procedures contained in this Appendix become unconservative, particularly for cast-in anchors in concrete with compressive strengths in the range of 75 to 85 MPa. Until adequate test results are available, an upper limit on f'_c of 70 MPa has been imposed in the design of cast-in-place anchors. This is consistent with Sec. 6.4 and 8.2. The companion ACI 355.2 does not require testing of post-installed anchors in concrete with f'_c greater than 55 MPa because some post-installed anchors may have difficulty expanding in very high-strength concretes. Because of this, f'_c is limited to 55 MPa in the design of post-installed anchors unless testing is performed.

K.4 General Requirements for Strength of Anchors

K.4.1 Strength design of anchors shall be based either on computation using design models that satisfy the requirements of K.4.2, or on test evaluation using the 5 percent fractile of test results for the following:

- (a) Steel strength of anchor in tension (K.5.1);
- (b) Steel strength of anchor in shear (K.6.1);
- (c) Concrete breakout strength of anchor in tension (K.5.2);
- (d) Concrete breakout strength of anchor in shear (K.6.2);
- (e) Pullout strength of anchor in tension (K.5.3);
- (f) Concrete side-face blowout strength of anchor in tension (K.5.4); and
- (g) Concrete pryout strength of anchor in shear (K.6.3).

Anchors shall also have to satisfy the required edge distances, spacings, and thicknesses to preclude splitting failure, as required in K.8.

Clarification for Section K.4.1: This Section gives the requirements for establishing the strength of anchors to concrete. The various types of steel and concrete failure modes for anchors are shown in Figures K.4.1(a) and K.4.1(b). Comprehensive discussions of anchor failure modes are included in References K.8 to K.10. Any model that complies with the requirements of Sections K.4.2 and K.4.3 can be used to establish the concrete related strengths. For anchors such as headed bolts, headed studs, and post-installed anchors, the concrete breakout design methods of Sections K.5.2 and K.6.2 are acceptable. The anchor strength is also dependent on the pullout strength of Sec K.5.3, the side-face blowout strength of Sec K.5.4, and the minimum spacings and edge distances of Sec K.8. The design of anchors for tension recognizes that the strength of anchors is sensitive to appropriate installation; installation requirements are included in Sec K.9. Some post-installed anchors are less sensitive to installation errors and tolerances. This is reflected in varied ϕ factors based on the assessment criteria of ACI 355.2.

Test procedures can also be used to determine the single anchor breakout strength in tension and in shear. The test results, however, are required to be evaluated on a basis statistically equivalent to that used to select the values for the concrete breakout method “considered to satisfy” provisions of Sec K.4.2. The basic strength cannot be taken greater than the 5 percent fractile. The number of tests has to be sufficient for statistical validity and should be considered in the determination of the 5 percent fractile.

K.4.1.1 Except as required in Sec K.3.3, the design of anchors shall satisfy,

$$\phi N_n \geq N_{ua} \quad (6.K.1)$$

$$\phi V_n \geq V_{ua} \quad (6.K.2)$$

K.4.1.2 In Equations 6.K.1 and 6.K.2, ϕN_n and ϕV_n are the lowest design strengths determined from all appropriate failure modes. ϕN_n is the lowest design strength in tension of an anchor or group of anchors as determined from consideration of ϕN_{sa} , $\phi n N_{pn}$, either ϕN_{sb} or ϕN_{sbg} , and either ϕN_{cb} or ϕN_{cbg} . ϕV_n is the lowest design strength in shear of an anchor or a group of anchors as determined from consideration of: ϕV_{sa} , either ϕV_{sb} or ϕV_{sbg} , and either ϕV_{cb} or ϕV_{cbg} .

K.4.1.3 Interaction effects shall be considered in accordance with Sec K.4.3, when both N_{ua} and V_{ua} are present.

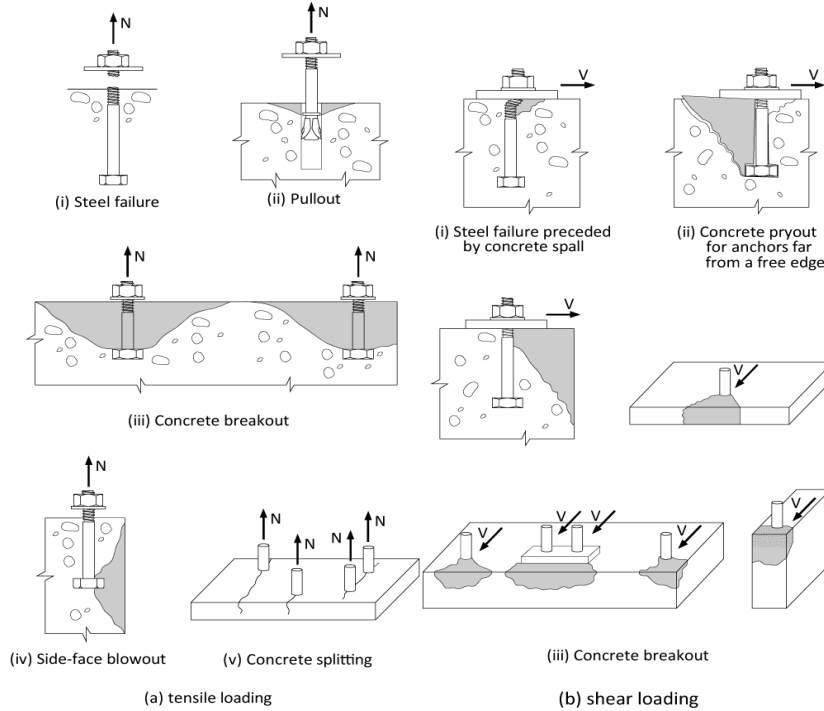


Figure 6.K.2 Failure modes for anchors.

K.4.2 For any anchor or group of anchors, the nominal strength shall be based on design models that result in predictions of strength in substantial agreement with results of comprehensive tests. The materials used in the tests shall be compatible with the materials used in the structure. The nominal strength shall be based on the 5 percent fractile of the basic individual anchor strength. For nominal strengths related to concrete strength, modifications for size effects, the number of anchors, the effects of close spacing of anchors, proximity to edges, depth of the concrete member, eccentric loadings of anchor groups, and presence or absence of cracking shall be taken into account. Limits on edge distances and anchor spacing in the design models shall be consistent with the tests that verified the model.

K.4.2.1 The effect of reinforcement provided to restrain the concrete breakout shall be permitted to be included in the design models used to satisfy Sec K.4.2. Where anchor reinforcement is provided in accordance with Sections K.5.2.9 and K.6.2.9, calculation of the concrete breakout strength in accordance with Sections K.5.2 and K.6.2 is not required.

Clarification for Section K.4.2.1: The addition of reinforcement in the direction of the load to restrain concrete breakout can greatly enhance the strength and deformation capacity of the anchor connection. Such enhancement is practical with cast-in anchors such as those used in precast sections.

References K.8, K.11, K.12, K.13, and K.14 provide information regarding the effect of reinforcement on the behavior of anchors. The effect of reinforcement is not included in the ACI 355.2 anchor acceptance tests or in the concrete breakout calculation method of Sections K.5.2 and K.6.2. The beneficial effect of supplementary reinforcement is recognized by the Condition A ϕ -factors in Sec K.4.4. Anchor reinforcement may be provided instead of calculating breakout strength using the provisions of Sec. 8.2 in conjunction with Sections K.5.2.9 and K.6.2.9.

The breakout strength of an unreinforced connection can be taken as an indication of the load at which significant cracking will occur. Such cracking can represent a serviceability problem if not controlled. (See Clarification for Sec K.6.2.1.)

K.4.2.2 When anchor diameters are not greater than 50 mm, and tensile embedments are not greater than 635 mm in depth, the concrete breakout strength requirements shall be considered satisfied by the design procedure of Sections K.5.2 and K.6.2.

Clarification for Section K.4.2.2: The method for concrete breakout design included as “considered to satisfy” K.4.2 was developed from the Concrete Capacity Design (CCD) Method,^{K.9,K.10} which was an adaptation of the κ Method^{K.15,K.16} and is considered to be accurate, relatively easy to apply, and capable of extension to irregular layouts. The CCD Method predicts the strength of an anchor or group of anchors by using a basic equation for tension, or for shear for a single anchor in cracked concrete, and multiplied by factors that account for the number of anchors, edge distance, spacing, eccentricity, and absence of cracking. The limitations on anchor size and embedment length are based on the current range of test data.

The breakout strength calculations are based on a model suggested in the κ Method. It is consistent with a breakout prism angle of approximately 35° [Figure 6.K.3(a) and (b)].

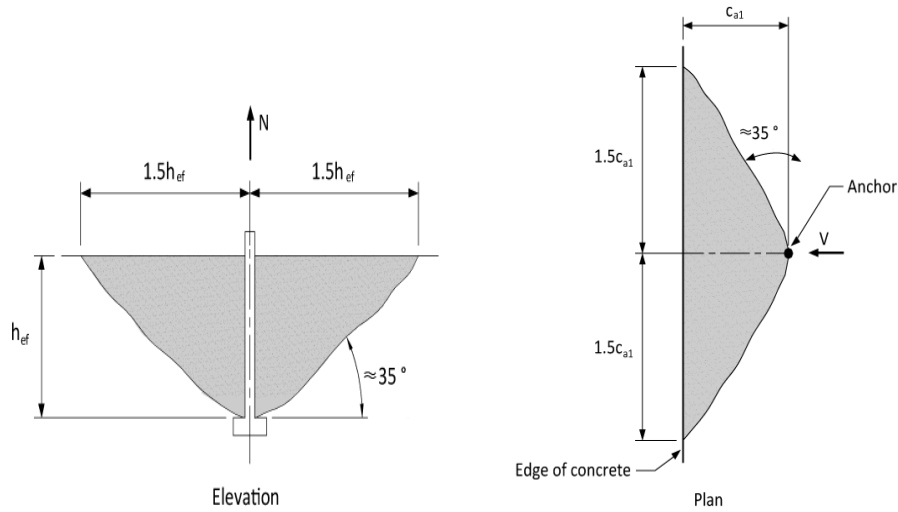


Figure 6.K.3(a) Breakout cone for tension

Figure 6.K.3(b) Breakout cone for shear

K.4.3 Resistance to combined tensile and shear loads shall be considered in design using an interaction expression that results in computation of strength in substantial agreement with results of comprehensive tests. This requirement shall be considered satisfied by Sec K.7.

Clarification for Sections K.4.2 and K.4.3: Sections K.4.2 and K.4.3 establish the performance factors for which anchor design models are required to be verified. Many possible design approaches exist and the user is always permitted to “design by test” using Sec K.4.2 as long as sufficient data are available to verify the model.

K.4.4 For anchors in concrete, strength reduction factor ϕ shall be as follows when the load combinations of Sec 6.2.2 are used:

- (a) When strength of a ductile steel element governs anchor
 - (i) Tension loads 0.75
 - (ii) Shear loads 0.65
- (b) When strength of a brittle steel element governs anchor
 - (i) Tension loads 0.65
 - (ii) Shear loads 0.60

- (c) When concrete breakout, side-face blowout, pullout, or pryout strength governs anchor

		Condition A	Condition B
(i)	Shear Loads	0.75	0.70
(ii)	Tension loads		
	Cast-in headed studs, headed bolts, or hooked bolts	0.75	0.70
	Post-installed anchors with category as determined from ACI 355.2		
	Category 1	0.75	0.65
	(Low sensitivity to installation and high reliability)		
	Category 2	0.65	0.55
	(Medium sensitivity to installation and medium reliability)		
	Category 3	0.55	0.45
	(High sensitivity to installation and lower reliability)		

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 or pryout strength.

Clarification for Section K.4.4: The ϕ factors for steel strength are based on using f_{uta} to determine the nominal strength of the anchor (see Sections K.5.1 and K.6.1) rather than f_{ya} as used in the design of reinforced concrete members. Although the ϕ factors for use with f_{uta} appear low, they result in a level of safety consistent with the use of higher ϕ factors applied to f_{ya} . The smaller ϕ factors for shear than for tension do not reflect basic material differences but rather account for the possibility of a non-uniform distribution of shear in connections with multiple anchors. It is acceptable to have a ductile failure of a steel element in the attachment if the attachment is designed so that it will undergo ductile yielding at a load level corresponding to anchor forces no greater than the minimum design strength of the anchors specified in Sec K.3.3. (See Sec K.3.3.5.)

Two conditions are recognized for anchors governed by the more brittle concrete breakout or blowout failure. If supplementary reinforcement is present (Condition A), greater deformation capacity is provided than in the case where such supplementary reinforcement is not present (Condition B). An explicit design of supplementary

reinforcement is not required. However, the arrangement of supplementary reinforcement should generally conform to that of the anchor reinforcement shown in Figures 6.K.7 and 6.K.11(b). Full development is not required. The strength reduction factors for anchor reinforcement are given in Sections 6.K.7 and K.6.2.9.

The ACI 355.2 tests for sensitivity to installation procedures determine the category appropriate for a particular anchoring device. In the ACI 355.2 tests, the effects of variability in anchor torque during installation, tolerance on drilled hole size, energy level used in setting anchors, and for anchors approved for use in cracked concrete, increased crack widths are considered. The three categories of acceptable post-installed anchors are:

Category 1: low sensitivity to installation and high reliability;

Category 2: medium sensitivity to installation and medium reliability; and

Category 3: high sensitivity to installation and lower reliability.

The capacities of anchors under shear loads are not as sensitive to installation errors and tolerances. Therefore, for shear calculations of all anchors, $\phi = 0.75$ for Condition A and $\phi = 0.70$ for Condition B.

K.5 Design Requirements for Tensile Loading

K.5.1 Steel strength of anchor in tension

K.5.1.1 The nominal strength of an anchor in tension as governed by the steel, N_{sa} , shall be evaluated by calculations based on the properties of the anchor material and the physical dimensions of the anchor.

K.5.1.2 The nominal strength of a single anchor or group of anchors in tension, N_{sa} , shall not be greater than

$$N_{sa} = nA_{se,N}f_{uta} \quad (6.K.3)$$

Where, n is the number of anchors in the group, $A_{se,N}$ is the effective cross-sectional area of a single anchor in tension, mm², and f_{uta} shall not be taken greater than the smaller of $1.9f_{ya}$ and 860 MPa.

Clarification for Section K.5.1.2: The nominal strength of anchors in tension is best represented as a function of f_{uta} rather than f_{ya} because the large majority of anchor materials do not exhibit a well-defined yield point. The American Institute of Steel Construction (AISC) has based tension strength of anchors on $A_{se,N}f_{uta}$ since the 1986 edition of their specifications. The use of Eq. 6.K.3 with Sec 6.2.2 load factors and the ϕ -factors of K.4.4 give design strengths consistent with the AISC Load and Resistance Factor Design Specifications.^{K.19}

The limitation of $1.9f_{ya}$ on f_{uta} is to ensure that, under service load conditions, the anchor does not exceed f_{ya} . The limit on f_{uta} of $1.9f_{ya}$ was determined by converting the LRFD provisions to corresponding service level conditions. For Section 6.2.2, the average load factor of 1.4 (from $1.2D + 1.7L$) divided by the highest ϕ -factor (0.75 for tension) results in a limit of f_{uta}/f_{ya} of $1.4/0.75 = 1.87$. The serviceability limitation of f_{uta} was taken as $1.9f_{ya}$. If the ratio of f_{uta} to f_{ya} exceeds this value, the anchoring may be subjected to service loads above f_{ya} under service loads. Although not a concern for standard structural steel anchors (maximum value of f_{uta}/f_{ya} is 1.6 for ASTM A307), the limitation is applicable to some stainless steels. The effective cross-sectional area of an anchor should be provided by the manufacturer of expansion anchors with reduced cross-sectional area for the expansion mechanism. For threaded bolts, ANSI/ASME B1.1^{K.1} defines $A_{se,N}$ as

$$A_{se,N} = \frac{\pi}{4} \left(d_a - \frac{0.9743}{n_t} \right)^2, \text{ Where } n_t \text{ is the number of threads per mm.}$$

K.5.2 Concrete Breakout Strength of Anchor in Tension

K.5.2.1 The nominal concrete breakout strength, N_{cb} or N_{cbg} , of a single anchor or group of anchors in tension shall not exceed

(a) For a single anchor

$$N_{cb} = \frac{A_{NC}}{A_{NCO}} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \quad (6.K.4)$$

(b) For a group of anchors

$$N_{cbg} = \frac{A_{NC}}{A_{NCO}} \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \quad (6.K.5)$$

Factors $\psi_{ec,N}$, $\psi_{ed,N}$, $\psi_{c,N}$, and $\psi_{cp,N}$ are defined in Sections K.5.2.4, K.5.2.5, K.5.2.6, and K.5.2.7, respectively. A_{NC} is the projected concrete failure area of a single anchor or group of anchors that shall be 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 tensioned anchors in the group. A_{NCO} is the projected concrete failure area of a single anchor with an edge distance equal to or greater than $1.5h_{ef}$.

$$A_{NCO} = 9h_{ef}^2 \quad (6.K.6)$$

Clarification for Section K.5.2.1: The effects of multiple anchors, spacing of anchors, and edge distance on the nominal concrete breakout strength in tension are included by applying the modification factors A_{NC}/A_{NCO} and $\psi_{ed,N}$ in Eq. 6.K.4 and 6.K.5. Figure 6.K.4(a) shows A_{NCO} and the development of Eq. 6.K.6. A_{NCO} is the maximum projected area for a single anchor. Figure 6.K.4(b) shows examples of the projected areas for various single-anchor and multiple-anchor arrangements. Because A_{NC} is the total

projected area for a group of anchors, and A_{Nco} is the area for a single anchor, there is no need to include n , the number of anchors, in Eq. 6.K.4 or 6.K.5. If anchor groups are positioned in such a way that their projected areas overlap, the value of A_{Nc} is required to be reduced accordingly.

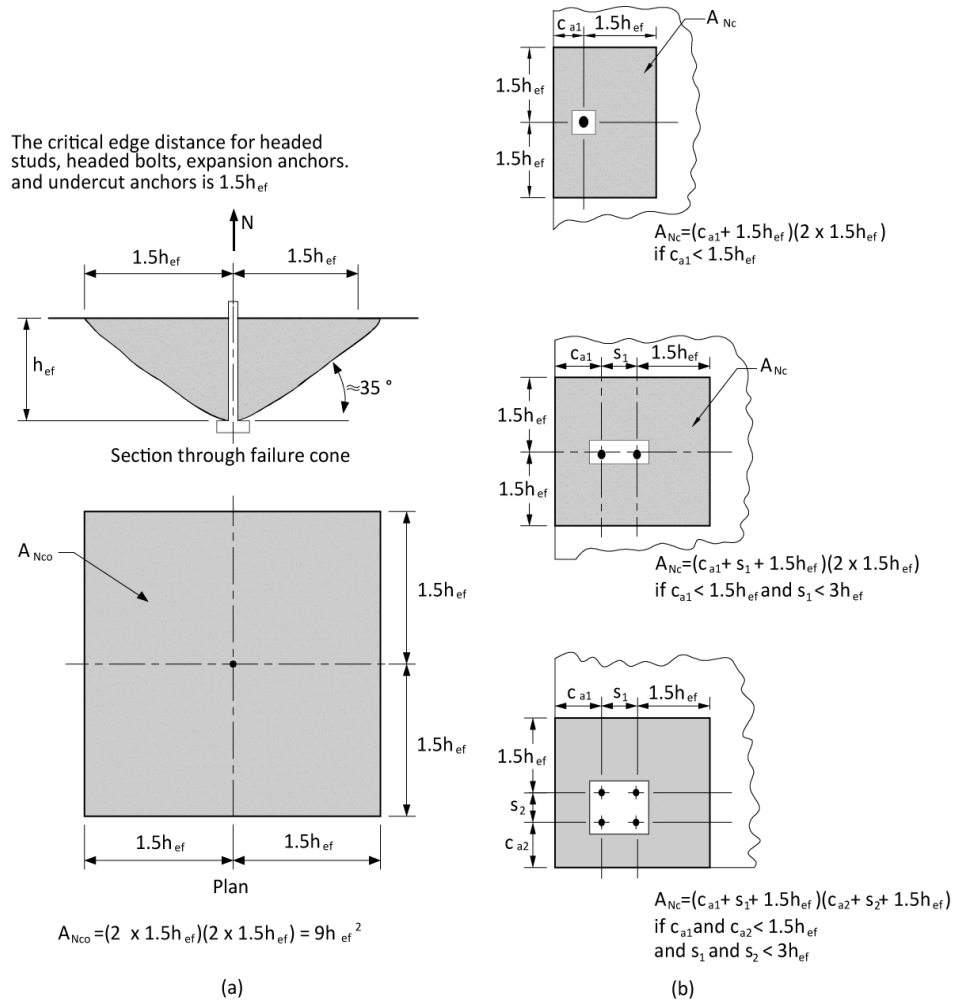


Figure 6.K.4(a) Calculation of A_{Nco} ; and (b) calculation of A_{Nc} for single anchors and groups of anchors.

K.5.2.2 The basic concrete breakout strength of a single anchor in tension in cracked concrete, N_b , shall not exceed

$$N_b = k_c \lambda \sqrt{f'_c} (h_{ef})^{1.5} \quad (6.K.7)$$

Where,

$k_c = 10$ for cast-in anchors; and

$k_c = 7$ for post-installed anchors.

The value of k_c for post-installed anchors shall be permitted to be increased above 7 based on ACI 355.2 product-specific tests, but shall in no case exceed 10. Alternatively, for cast-in headed studs and headed bolts with $280 \text{ mm} \leq h_{ef} \leq 635 \text{ mm}$, N_b shall not exceed

$$N_b = 3.9\lambda\sqrt{f'_c}(h_{ef})^{5/3} \quad (6.K.8)$$

Clarification for Section K.5.2.2: The basic equation for anchor strength was derived^{K.9-K.11, K.16} assuming a concrete failure prism with an angle of about 35° , considering fracture mechanics concepts.

The values of k_c in Eq. 6.K.7 were determined from a large database of test results in uncracked concrete^{K.9} at the 5 percent fractile. The values were adjusted to corresponding k_c values for cracked concrete.^{K.10, K.20} Higher k_c values for post-installed anchors may be permitted, provided they have been determined from product approval testing in accordance with ACI 355.2. For anchors with a deep embedment ($h_{ef} > 280 \text{ mm}$) test evidence indicates the use of $(h_{ef})^{1.5}$ can be overly conservative for some cases. Often, such tests have been with selected aggregates for special applications. An alternative expression (Eq. 6.K.8) is provided using $(h_{ef})^{5/3}$ for evaluation of cast-in anchors with $280 \text{ mm} \leq h_{ef} \leq 635 \text{ mm}$. The limit of 635 mm corresponds to the upper range of test data. This expression can also be appropriate for some undercut post-installed anchors. However, for such anchors, the use of Eq. 6.K.8 should be justified by test results in accordance with Sec K.4.2.

K.5.2.3 If anchors are located less than $1.5h_{ef}$ from three or more edges, the value of h_{ef} used in Eq. 6.K.4 to 6.K.11 shall be the greater of $C_{a, \max}/1.5$ and one-third of the maximum spacing between anchors within the group.

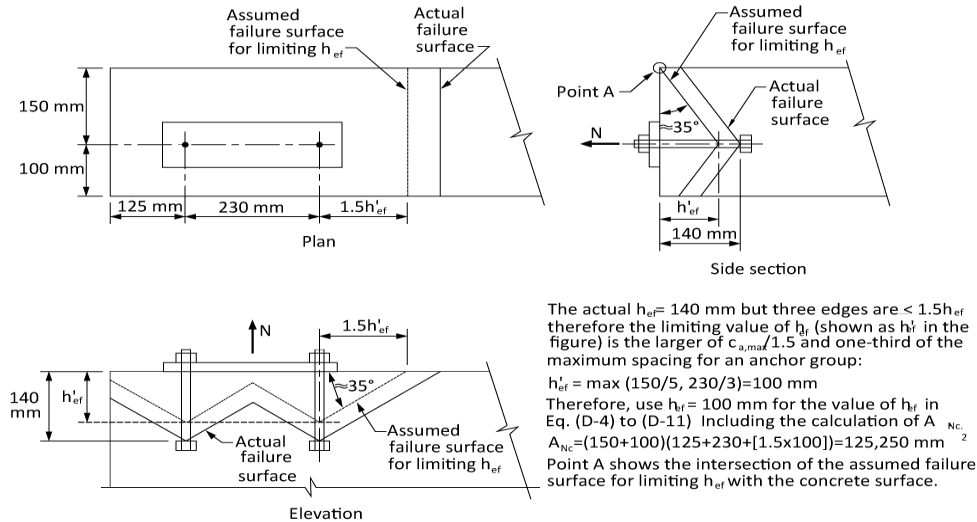


Figure 6.K.5 Tension in narrow members.

Clarification for Section K.5.2.3: For anchors located less than $1.5h_{ef}$ from three or more edges, the tensile breakout strength computed by the CCD Method, which is the basis for Eq. 6.K.4 to 6.K.11, gives overly conservative results.^{K.21} This occurs because the ordinary definitions of A_{Nc}/A_{Nco} do not correctly reflect the edge effects. This problem is corrected by limiting the value of h_{ef} used in Eq. 6.K.4 through 6.K.11 to $C_{a,max}/1.5$, where $C_{a,max}$ is the largest of the influencing edge distances that are less than or equal to the actual $1.5h_{ef}$. In no case should $C_{a,max}/1.5$ be taken less than one-third of the maximum spacing between anchors within the group. The limit on h_{ef} of at least one-third of the maximum spacing between anchors within the group prevents the use of a calculated strength based on individual breakout prisms for a group anchor configuration.

This approach is illustrated in Figure 6.K.5. In this example, the proposed limit on the value of h_{ef} to be used in the computations where, $h_{ef} = C_{a,max}/1.5$, results in $h_{ef} = 100$ mm. For this example, this would be the proper value to be used for h_{ef} in computing the resistance even if the actual embedment depth is larger. The requirement of K.5.2.3 may be visualized by moving the actual concrete breakout surface, which originates at the actual h_{ef} , toward the surface of the concrete parallel to the applied tension load. The value of h_{ef} used in Eq. 6.K.4 to 6.K.11 is determined when either: (a) the outer boundaries of the failure surface first intersect a free edge; or (b) the intersection of the breakout surface between anchors within the group first intersects the surface of the concrete. For the example shown in Figure 6.K.5, Point “A” defines the intersection of the assumed failure surface for limiting h_{ef} with the concrete surface.

K.5.2.4 The modification factor for anchor groups loaded eccentrically in tension, $\psi_{ec,N}$, shall be computed as

$$\psi_{ec,N} = \frac{1}{\left(1 + \frac{2e'_N}{3h_{ef}}\right)} \quad (6.K.9)$$

But, $\psi_{ec,N}$ shall not be taken greater than 1.0. If the loading on an anchor group is such that only some anchors are in tension, only those anchors that are in tension shall be considered when determining the eccentricity e'_N for use in Eq. 6.K.9 and for the calculation of N_{cbg} in Eq. 6.K.5.

In the case where eccentric loading exists about two axes, the modification factor, $\psi_{ec,N}$, shall be computed for each axis individually and the product of these factors used as $\psi_{ec,N}$ in Eq. 6.K.5.

Clarification for Section K.5.2.4: Figure 6.K.6(a) shows a group of anchors that are all in tension but the resultant force is eccentric with respect to the centroid of the anchor group. Groups of anchors can also be loaded in such a way that only some of the anchors are in tension (Figure 6.K.6(b)). In this case, only the anchors in tension are to be considered in the determination of e'_N . The anchor loading has to be determined as the resultant anchor tension at an eccentricity with respect to the center of gravity of the anchors in tension.

K.5.2.5 For single anchors or anchor groups loaded in tension, the modification factor for edge effects, $\psi_{ed,N}$, shall be computed as

For, $c_{a,min} \geq 1.5h_{ef}$

$$\psi_{ed,N} = 1.0 \quad (6.K.10)$$

For, $c_{a,min} < 1.5h_{ef}$

$$\psi_{ed,N} = 0.7 + 0.3 \frac{c_{a,min}}{1.5h_{ef}} \quad (6.K.11)$$

Clarification for Section K.5.2.5: When anchors are located close to an edge so that there is not enough space for a complete breakout prism to develop, the strength of the anchor is further reduced beyond that reflected in A_{Nc}/A_{Nco} . If the smallest side cover distance is greater than or equal to $1.5h_{ef}$, a complete prism can form and there is no reduction ($\psi_{ec,N} = 1$). If the side cover is less than $1.5h_{ef}$, the factor $\psi_{ed,N}$ is required to adjust for the edge effect.^{K.9}

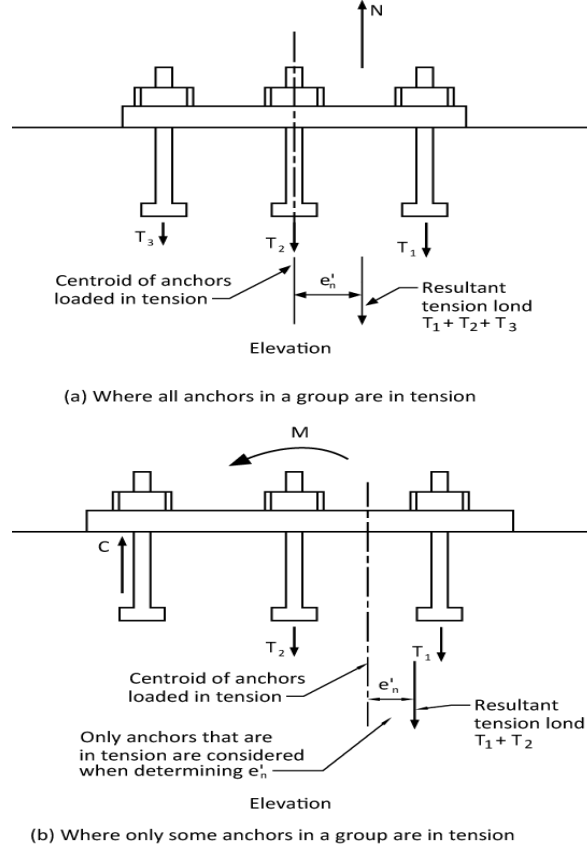


Figure 6.K.6 Definition of e'_N for a group of anchors.

K.5.2.6 When anchors are located in a region of a concrete member where analysis indicates no cracking at service load levels, the following modification factor shall be permitted:

$\psi_{c,N} = 1.25$ for cast-in anchors; and

$\psi_{c,N} = 1.4$ for post-installed anchors, where the value of k_c used in Eq. 6.K.7 is 7.

Where the value of k_c used in Eq. 6.K.7 is taken from the ACI 355.2 product evaluation report for post-installed anchors qualified for use in both cracked and uncracked concrete, the values of k_c and $\psi_{c,N}$ shall be based on the ACI 355.2 product evaluation report.

Where the value of k_c used in Eq. 6.K.7 is taken from the ACI 355.2 product evaluation report for post-installed anchors qualified for use in uncracked concrete, $\psi_{c,N}$ shall be taken as 1.0.

When analysis indicates cracking at service load levels, $\psi_{c,N}$ shall be taken as 1.0 for both cast-in anchors and post-installed anchors. Post-installed anchors shall be qualified for use in cracked concrete in accordance with ACI 355.2. The cracking in the concrete shall be controlled by flexural reinforcement distributed in accordance with 10.6.4, or equivalent crack control shall be provided by confining reinforcement.

Clarification for Section K.5.2.6: Post-installed and cast-in anchors that have not met the requirements for use in cracked concrete according to ACI 355.2 should be used in uncracked regions only. The analysis for the determination of crack formation should include the effects of restrained shrinkage (see Sec. 8.1). The anchor qualification tests of ACI 355.2 require that anchors in cracked concrete zones perform well in a crack that is 0.3 mm wide. If wider cracks are expected, confining reinforcement to control the crack width to about 0.3 mm should be provided.

The concrete breakout strengths given by Eq. 6.K.7 and 6.K.8 assume cracked concrete (that is, $\psi_{c,N} = 1.0$) with $\psi_{c,N}k_c = 10 = 10$ for cast-in-place, and 7 for post-installed (cast-in 40 percent higher). When the uncracked concrete $\psi_{c,N}$ factors are applied (1.25 for cast-in, and 1.4 for post-installed), the results are $\psi_{c,N}k_c$ factors of 13 for cast-in and 10 for post-installed (25 percent higher for cast-in). This agrees with field observations and tests that show cast-in anchor strength exceeds that of post-installed for both cracked and uncracked concrete.

K.5.2.7 The modification factor for post-installed anchors designed for uncracked concrete in accordance with Sec K.5.2.6 without supplementary reinforcement to control splitting, $\psi_{cp,N}$, shall be computed as follows using the critical distance c_{ac} as defined in Sec K.8.6.

For, $c_{a,min} \geq c_{ac}$

$$\psi_{cp,N} = 1.0 \quad (6.K.12)$$

For, $c_{a,min} < c_{ac}$

$$\psi_{cp,N} = \frac{c_{a,min}}{c_{ac}} \quad (6.K.13)$$

But $\psi_{cp,N}$ determined from Eq. 6.K.13 shall not be taken less than $1.5h_{ef}/C_{ac}$, where the critical distance C_{ac} is defined in Sec K.8.6. For all other cases, including cast-in anchors, $\psi_{cp,N}$ shall be taken as 1.0.

Clarification for Section K.5.2.7: The design provisions in K.5 are based on the assumption that the basic concrete breakout strength can be achieved if the minimum edge distance, $C_{a,min}$, equals $1.5h_{ef}$. However, test results^{K.22} indicate that many torque-controlled and displacement-controlled expansion anchors and some undercut anchors require minimum edge distances exceeding $1.5h_{ef}$ to achieve the basic concrete breakout strength when tested in uncracked concrete without supplementary reinforcement to control splitting. When a tension load is applied, the resulting tensile stresses at the embedded end of the anchor are added to the tensile stresses induced due to anchor installation, and splitting failure may occur before reaching the concrete breakout strength defined in Sec K.5.2.1. To account for this potential splitting mode of failure, the basic concrete breakout strength is reduced by a factor $\psi_{cp,N}$ if $C_{a,min}$ is less than the critical edge distance C_{ac} . If supplementary reinforcement to control splitting is present or if the anchors are located in a region where analysis indicates cracking of the concrete at service loads, then the reduction factor $\psi_{cp,N}$ is taken as 1.0. The presence of supplementary reinforcement to control splitting does not affect the selection of Condition A or B in Sec K.4.4.

K.5.2.8 Where an additional plate or washer is added at the head of the anchor, it shall be permitted to calculate the projected area of the failure surface by projecting the failure surface outward $1.5h_{ef}$ from the effective perimeter of the plate or washer. The effective perimeter shall not exceed the value at a section projected outward more than the thickness of the washer or plate from the outer edge of the head of the anchor.

K.5.2.9 Where anchor reinforcement is developed in accordance with Sec. 8.3 on both sides of the breakout surface, the design strength of the anchor reinforcement shall be permitted to be used instead of the concrete breakout strength in determining ϕN_n . A strength reduction factor of 0.75 shall be used in the design of the anchor reinforcement.

Clarification for Section K.5.2.9: For conditions where the factored tensile force exceeds the concrete breakout strength of the anchor(s) or where the breakout strength is not evaluated, the nominal strength can be that of anchor reinforcement properly anchored as illustrated in Figure 6.K.7. Care needs to be taken in the selection and positioning of the anchor reinforcement. The anchor reinforcement should consist of stirrups, ties, or hairpins placed as close as practicable to the anchor. Only reinforcement spaced less than $0.5h_{ef}$ from the anchor centerline should be included as anchor reinforcement. The research^{K.14} on which these provisions is based was limited to anchor reinforcement with maximum diameter similar to a No. 16 bar. It is beneficial for the anchor reinforcement to enclose the surface reinforcement. In sizing the anchor reinforcement, use of a 0.75 strength reduction factor ϕ is recommended as is used for strut-and-tie models. As a practical matter, use of anchor reinforcement is generally limited to cast-in-place anchors.

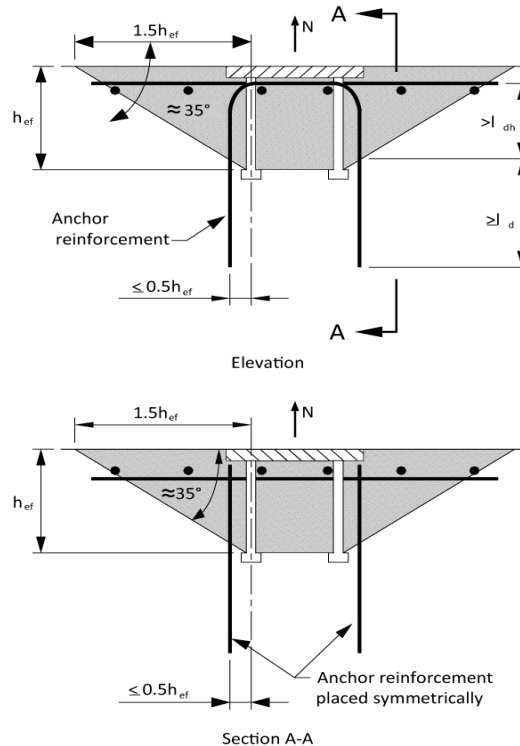


Figure 6.K.7 Anchor reinforcement for tension.

K.5.3 Pullout Strength of Anchor in Tension

K.5.3.1 The nominal pullout strength of a single anchor in tension, N_{pn} , shall not exceed

$$N_{pn} = \psi_{c,p} N_p \quad (6.K.14)$$

Where, $\psi_{c,p}$ is defined in K.5.3.6.

K.5.3.2 The values of N_p for post-installed expansion and undercut anchors shall be based on the 5 percent fractile of results of tests performed and evaluated according to ACI 355.2. It is not permissible to calculate the pullout strength in tension for such anchors.

Clarification for Section K.5.3.2: The pullout strength equations given in Sections K.5.3.4 and K.5.3.5 are only applicable to cast-in headed and hooked anchors;^{K.8.K.23} they are not applicable to expansion and undercut anchors that use various mechanisms for end anchorage unless the validity of the pullout strength equations are verified by tests.

K.5.3.3 It shall be permitted to evaluate the pullout strength in tension for single cast-in headed studs and headed bolts using Sec K.5.3.4. For single J- or L-bolts, it shall be permitted to evaluate the pullout strength in tension using K.5.3.5. Alternatively, it shall be permitted to use values of N_p based on the 5 percent fractile of tests performed and evaluated in the same manner as the ACI 355.2 procedures but without the benefit of friction.

Clarification for Section K.5.3.3: The pullout strength in tension of headed studs or headed bolts can be increased by providing confining reinforcement, such as closely spaced spirals, throughout the head region. This increase can be demonstrated by tests.

K.5.3.4 For a single headed stud or headed bolt, the pullout strength in tension, N_p , for use in Eq. 6.K.14, shall not exceed

$$N_p = 8A_{brg}f'_c \quad (6.K.15)$$

Clarification for K.5.3.4: The value computed from Eq. 6.K.15 corresponds to the load at which crushing of the concrete occurs due to bearing of the anchor head.^{K.8.K.13} It is not the load required to pull the anchor completely out of the concrete, so the equation contains no term relating to embedment depth. Local crushing of the concrete greatly reduces the stiffness of the connection, and generally will be the beginning of a pullout failure.

K.5.3.5 For a single hooked bolt, the pullout strength in tension, N_p , for use in Eq. 6.K.14 shall not exceed

$$N_p = 0.9f'_c e_h d_a \quad (6.K.16)$$

Where, $3d_a \leq e_h \leq 4.5d_a$

Clarification for Section K.5.3.5: Eq. 6.K.16 for hooked bolts was developed by Lutz based on the results of Reference K.23. Reliance is placed on the bearing component only, neglecting any frictional component because crushing inside the hook will greatly reduce the stiffness of the connection, and generally will be the beginning of pullout failure. The limits on e_h are based on the range of variables used in the three tests programs reported in Reference K.23.

K.5.3.6 When an anchor is located in a region of a concrete member where analysis indicates no cracking at service load levels, the following modification factor shall be permitted $\psi_{c,p} = 1.4$ Where analysis indicates cracking at service load levels, $\psi_{c,p}$ shall be taken as 1.0.

K.5.4 Concrete Side-Face Blowout Strength of a Headed Anchor in Tension

Clarification for Section K.5.4: Concrete side-face blowout strength of a headed anchor in tension. For side-face blowout, the design requirements are based on the recommendations of Reference K.24. These requirements are applicable to headed anchors that usually are cast-in anchors. Splitting during installation rather than side-face blowout generally governs post-installed anchors, and is evaluated by the ACI 355.2 requirements.

K.5.4.1 With deep embedment close to an edge ($h_{ef} > 2.5c_{a1}$), the nominal side-face blowout strength, $h_{ef} > N_{sb}$, of a single headed anchor shall not exceed

$$N_{sb} = (13c_{a1}\sqrt{A_{brg}})\lambda\sqrt{f'_c} \quad (6.K.17)$$

If c_{a2} for the single headed anchor is less than $3c_{a1}$, the value of N_{sb} shall be multiplied by the factor $\frac{1+\frac{c_{a2}}{c_{a1}}}{4}$, where, $1.0 \leq c_{a2}/c_{a1} \leq 3.0$.

K.5.4.2 For multiple headed anchors with deep embedment close to an edge ($h_{ef} > 2.5hc_{a1}$) and anchor spacing less than $6c_{a1}$, the nominal strength of those anchors susceptible to a side-face blowout failure N_{sbg} shall not exceed

$$N_{sbg} = \left(1 + \frac{s}{6c_{a1}}\right) N_{sb} \quad (6.K.18)$$

Where, s is the distance between the outer anchors along the edge, and N_{sb} is obtained from Eq. 6.K.17 without modification for a perpendicular edge distance.

Clarification for Section K.5.4.2: Only those anchors close to an edge ($h_{ef} > 2.5c_{a1}$), that are loaded in tension should be considered when determining nominal side-face blowout strength for multiple headed anchors. Their strength should be compared to the proportion of the tensile load applied to those anchors.

K.6 Design Requirements for Shear Loading

K.6.1 Steel Strength of Anchor in Shear

K.6.1.1 For an anchor in shear, the nominal strength governed by steel, V_{sa} , shall be evaluated by calculations based on the properties of the anchor material and the physical dimensions of the anchor.

K.6.1.2 For a single anchor or group of anchors in shear, the nominal strength, V_{sa} , shall not exceed (a) to (c):

(a) For cast-in headed stud anchor

$$V_{sa} = nA_{se,V}f_{uta} \quad (6.K.19)$$

Where, n is the number of anchors in the group, $A_{se,V}$ is the effective cross-sectional area of a single anchor in shear, mm^2 , and f_{uta} shall not be taken greater than the smaller of $1.9f_{ya}$ and 860 MPa.

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

$$V_{sa} = 0.6nA_{se,V}f_{uta} \quad (6.K.20)$$

Where, n is the number of anchors in the group, $A_{se,V}$ is the effective cross-sectional area of a single anchor in shear, mm^2 , and f_{uta} shall not be taken greater than the smaller of $1.9f_{ya}$ and 860 MPa.

(c) For post-installed anchors where sleeves extend through the shear plane, V_{sa} shall be based on the results of tests performed and evaluated according to ACI 355.2. Alternatively, Eq. 6.K.20 shall be permitted to be used.

Clarification for Section K.6.1.2: The nominal shear strength of anchors is best represented as a function of f_{uta} rather than f_{ya} because the large majority of anchor materials do not exhibit a well-defined yield point. Welded studs develop a higher steel shear strength than headed anchors due to the fixity provided by the weld between the studs and the base plate. The use of Eq. 6.K.19 and 6.K.20 with 6.2.2 load factors and the ϕ -factors of K.4.4 give design strengths consistent with the AISC Load and Resistance Factor Design Specifications.^{K.19}

The limitation of $1.9f_{ya}$ on f_{uta} is to ensure that, under service load conditions, the anchor stress does not exceed f_{ya} . The limit on f_{uta} of $1.9f_{ya}$ was determined by converting the LRFD provisions to corresponding service level conditions as discussed in Clarification for K.5.1.2.

The effective cross-sectional area of an anchor should be provided by the manufacturer of expansion anchors with reduced cross-sectional area for the expansion mechanism. For threaded bolts, ANSI/ASME B1.1^{K.1} defines $A_{se,V}$ as

$$A_{se,V} = \frac{\pi}{4} \left(d_a - \frac{0.9743}{n_t} \right)^2$$

Where, n_t is the number of threads per mm.

K.6.1.3 Where anchors are used with built-up grout pads, the nominal strengths of Sec K.6.1.2 shall be multiplied by a 0.80 factor.

K.6.2 Concrete Breakout Strength of Anchor in Shear

K.6.2.1 The nominal concrete breakout strength, V_{cb} or V_{cbg} , in shear of a single anchor or group of anchors shall not exceed:

- (a) For shear force perpendicular to the edge on a single anchor

$$V_{cb} = \frac{A_{vc}}{A_{vco}} \psi_{ed,V} \psi_{ec,V} \psi_{ch,V} V_b \quad (6.K.21)$$

- (b) For shear force perpendicular to the edge on a group of anchors

$$V_{cbg} = \frac{A_{vc}}{A_{vco}} \psi_{ec,V} \psi_{ed,V} \psi_{ch,V} V_b \quad (6.K.22)$$

- (c) For shear force parallel to an edge, V_{cb} or V_{cbg} shall be permitted to be twice the value of the shear force determined from Eq. 6.K.21 or 6.K.22, respectively, with the shear force assumed to act perpendicular to the edge and with $\psi_{ed,V}$ taken equal to 1.0.

- (d) For anchors located at a corner, the limiting nominal concrete breakout strength shall be determined for each edge, and the minimum value shall be used.

Factors $\psi_{ec,V}$, $\psi_{ed,V}$, $\psi_{ch,V}$, and $\psi_{cb,V}$ are defined in Sections K.6.2.5, K.6.2.6, K.6.2.7, and K.6.2.8, respectively. V_b is the basic concrete breakout strength value for a single anchor. A_{vc} is the 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 shall be 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} shall be taken as the distance from the edge to this axis. A_{vc} shall not exceed nA_{vco} , where n is the number of anchors in the group.

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

$$A_{vco} = 4.5(c_{a1})^2 \quad (6.K.23)$$

Where anchors are located at varying distances from the edge and the anchors are welded to the attachment so as to distribute the force to all anchors, it shall be permitted to evaluate the strength based on the distance to the farthest row of anchors from the edge. In this case, it shall be permitted to base the value of c_{a1} on the distance from the edge to the axis of the farthest anchor row that is selected as critical, and all of the shear shall be assumed to be carried by this critical anchor row alone.

Clarification for Section K.6.2.1: The shear strength equations were developed from the CCD Method. They assume a breakout cone angle of approximately 35° (see Figure 6.K.3(b)), and consider fracture mechanics theory. The effects of multiple anchors, spacing of anchors, edge distance, and thickness of the concrete member on nominal concrete breakout strength in shear are included by applying the reduction factor of A_{vc}/A_{vco} in Eq. 6.K.21 and 6.K.22, and $\psi_{ec,V}$ in Eq. 6.K.22. For anchors far from the edge, Sec K.6.2 usually will not govern. For these cases, Sections K.6.1 and K.6.3 often govern.

Figure 6.K.8(a) shows A_{vco} and the development of Eq. 6.K.23. A_{vco} is the maximum projected area for a single anchor that approximates the surface area of the full breakout prism or cone for an anchor unaffected by edge distance, spacing, or depth of member. Figure 6.K.8(b) shows examples of the projected areas for various single-anchor and multiple-anchor arrangements. A_{vc} approximates the full surface area of the breakout cone for the particular arrangement of anchors. Because A_{vc} is the total projected area for a group of anchors, and A_{vco} is the area for a single anchor, there is no need to include the number of anchors in the equation.

When using Eq. 6.K.22 for anchor groups loaded in shear, both assumptions for load distribution illustrated in examples on the right side of Figure 6.K.8(b) should be considered because the anchors nearest the edge could fail first or the whole group could fail as a unit with the failure surface originating from the anchors farthest from the edge. If the anchors are welded to a common plate, when the anchor nearest the front edge begins to form a failure cone, shear load would be transferred to the stiffer and stronger rear anchor. For this reason, anchors welded to a common plate do not need to consider the failure mode shown in the upper right figure of Figure 6.K.8(b). The PCI Design Handbook approach^{K.18} suggests in Sec 6.5.2.2 that the strength of the anchors away from the edge be considered. Because this is a reasonable approach, assuming that the anchors are spaced far enough apart so that the shear failure surfaces do not intersect,^{K.11} Sec K.6.2 allows such a procedure. If the failure surfaces do not intersect, as would generally occur if the anchor spacing s is equal to or greater than $1.5c_{a1}$, then after

formation of the near-edge failure surface, the higher strength of the farther anchor would resist most of the load. As shown in the bottom right example in Figure 6.K.8(b), it would be appropriate to consider the shear strength to be provided entirely by this anchor with its much larger resisting failure surface. No contribution of the anchor near the edge is then considered. Checking the near-edge anchor condition is advisable to preclude undesirable cracking at service load conditions. Further discussion of design for multiple anchors is given in Reference K.8.

For the case of anchors near a corner subjected to a shear force with components normal to each edge, a satisfactory solution is to check independently the connection for each component of the shear force. Other specialized cases, such as the shear resistance of anchor groups where all anchors do not have the same edge distance, are treated in Reference K.11.

The detailed provisions of 6.K.8(a) apply to the case of shear force directed toward an edge. When the shear force is directed away from the edge, the strength will usually be governed by Sec K.6.1 or Sec K.6.3.

The case of shear force parallel to an edge is shown in Figure 6.K.8(c). A special case can arise with shear force parallel to the edge near a corner. In the example of a single anchor near a corner (see Figure 6.K.8(d)), the provisions for shear force applied perpendicular to the edge should be checked in addition to the provisions for shear force applied parallel to the edge.

K.6.2.2 For a single anchor in cracked concrete, the basic concrete breakout strength in shear, V_b shall not exceed

$$V_b = 0.6 \left(\frac{l_e}{d_a} \right)^{0.2} \sqrt{d_a} \lambda \sqrt{f'_c} (c_{a1})^{1.5} \quad (6.K.24)$$

Where, l_e is the load-bearing length of the anchor for shear ($l_e = h_{ef}$) for anchors with a constant stiffness over the full length of embedded section, such as headed studs and post-installed anchors with one tubular shell over full length of the embedment depth, $l_e = 2d_a$ for torque-controlled expansion anchors with a distance sleeve separated from expansion sleeve, and $l_e \leq 8d_a$ in all cases.

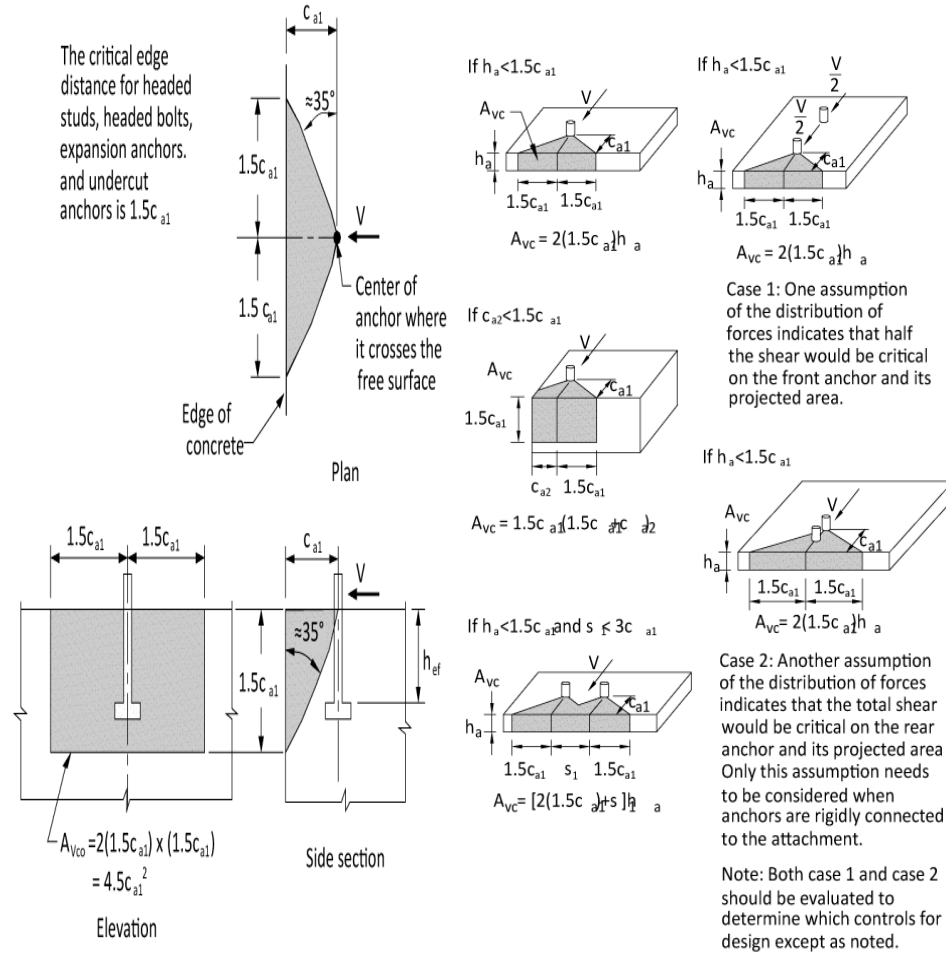
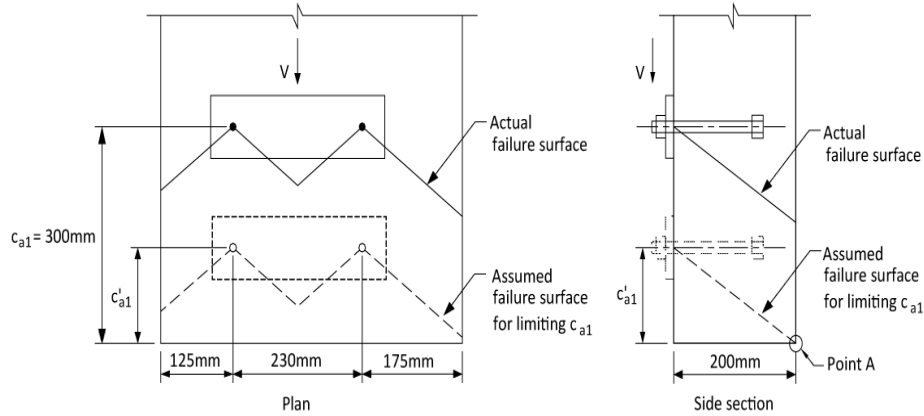


Figure 6.K.8 (a) Calculation of A_{vco} .

Figure 6.K.8 (b) Calculation of A_{vc} for single anchors and groups of anchors.

Figure 6.K.8(c) Shear force parallel to an edge.

Figure 6.K.8(d) Shear force near a corner.



The actual $c_{a1} = 300\text{mm}$ but two orthogonal edges c_{a2} and h_a are $< 1.5c$ therefore the limiting value of c_{a1} (shown as c'_{a1} in the figure) is the larger of $c_{a2, \max}/1.5$, $h_a/1.5$ and one-third of the maximum spacing for an anchor group:

$$c'_{a1} = \max(175/1.5, 200/1.5, 230/3) = 133\text{mm}$$

Therefore, use $c'_{a1} = 133\text{mm}$ in Eq. (D-21) to (D-28) including the calculation of A_{vc} :

$$A_{vc} = (125+230+175)(1.5 \times 133) = 105,735\text{mm}^2$$

Point a shows the intersection of the assumed failure surface for limiting c_{a1} with the concrete surface.

Figure 6.K.9 Shear when anchors are influenced by three or more edges.

Clarification for Section K.6.2.2: Like the concrete breakout tensile strength, the concrete breakout shear strength does not increase with the failure surface, which is proportional to $(C_{a1})^2$. Instead, the strength increases proportionally to $(C_{a1})^{1.5}$ due to size effect. The strength is also influenced by the anchor stiffness and the anchor diameter.^{K.9-K.11, K.16}

The constant, 0.6, in the shear strength equation was determined from test data reported in Reference K.9 at the 5 percent fractile adjusted for cracking.

K.6.2.3 For cast-in headed studs, headed bolts, or hooked bolts that are continuously welded to steel attachments having a minimum thickness equal to the greater of 10 mm and half of the anchor diameter, the basic concrete breakout strength in shear of a single anchor in cracked concrete, V_b , shall not exceed

$$V_b = 0.66 \left(\frac{I_e}{d_a} \right)^{0.2} \sqrt{d_a} \lambda \sqrt{f'_c} (c_{a1})^{1.5} \quad (6.K.25)$$

Where, I_e is defined in Sec K.6.2.2, provided that:

- for groups of anchors, the strength is determined based on the strength of the row of anchors farthest from the edge;
- anchor spacing, s , is not less than 65 mm; and
- reinforcement is provided at the corners if $C_{a2} \leq 1.5h_{ef}$.

Clarification for Section K.6.2.3: For the case of cast-in headed bolts continuously welded to an attachment, test data^{K.25} show that somewhat higher shear strength exists, possibly due to the stiff welding connection clamping the bolt more effectively than an attachment with an anchor gap. Because of this, the basic shear value for such anchors is increased. Limits are imposed to ensure sufficient rigidity. The design of supplementary reinforcement is discussed in References K.8, K.11, and K.12.

K.6.2.4 If anchors are influenced by three or more edges, the value of C_{a1} used in Eq. 6.K.23 to 6.K.29 shall not exceed the greatest of: $C_{a2}/1.5$ in either direction, $h_a/1.5$; and one-third of the maximum spacing between anchors within the group.

Clarification for Section K.6.2.4: If anchors are influenced by three or more edges where any edge distance is less than $1.5C_{a1}$, the shear breakout strength computed by the basic CCD Method, which is the basis for Equations 6.K.21 to 6.K.29, gives safe but overly conservative results. These cases were studied for the κ Method^{K.16} and the problem was pointed out by Lutz.^{K.21} Similarly, the approach used for tensile breakouts in Sec K.5.2.3, strength is correctly evaluated if the value of c_{a1} used in Equations 6.K.21 to 6.K.29 is limited to the maximum of $C_{a2}/1.5$ in each direction, $h_a/1.5$, and one-third of the maximum spacing between anchors within the group. The limit on c_{a1} of at least one-third of the maximum spacing between anchors within the group prevents the use of a calculated strength based on individual breakout prisms for a group anchor configuration.

This approach is illustrated in Figure 6.K.9. In this example, the limit on the value of c_{a1} is the largest of $C_{a2}/1.5$ in either direction, $h_a/1.5$, and one-third the maximum spacing between anchors for anchor groups results in $C_{a1} = 133$ mm. For this example, this would be the proper value to be used for C_{a1} in computing V_{cb} or V_{cbg} , even if the actual edge distance that the shear is directed toward is larger. The requirement of Sec K.6.2.4 may be visualized by moving the actual concrete breakout surface originating at the actual c_{a1} toward the surface of the concrete in the direction of the applied shear load. The value of C_{a1} used in Equations 6.K.21 to 6.K.29 is determined when either: (a) the outer boundaries of the failure surface first intersect a free edge; or (b) the intersection of the breakout surface between anchors within the group first intersects the surface of the concrete. For the example shown in Figure 6.K.9, Point “A” shows the intersection of the assumed failure surface for limiting C_{a1} with the concrete surface.

K.6.2.5 The modification factor for anchor groups loaded eccentrically in shear, $\psi_{ec,V}$, shall be computed as

$$\psi_{ec,V} = \frac{1}{\left(1 + \frac{2e'_V}{3c_{a1}}\right)} \quad (6.K.26)$$

But $\psi_{ec,V}$ shall not be taken greater than 1.0.

If the loading on an anchor group is such that only some anchors are loaded in shear in the same direction, only those anchors that are loaded in shear in the same direction shall be considered when determining the eccentricity of e_v' for use in Eq. 6.K.26 and for the calculation of V_{cbg} in Eq. 6.K.22.

Clarification for Section K.6.2.5: This section provides a modification factor for an eccentric shear force toward an edge on a group of anchors. If the shear force originates above the plane of the concrete surface, the shear should first be resolved as a shear in the plane of the concrete surface, with a moment that may or may not also cause tension in the anchors, depending on the normal force. Figure 6.K.10 defines the term e_v' for calculating the $\psi_{ec,V}$ modification factor that accounts for the fact that more shear is applied to one anchor than others, tending to split the concrete near an edge.

K.6.2.6 The modification factor for edge effect for a single anchor or group of anchors loaded in shear, $\psi_{ed,V}$, shall be computed as

For, $c_{a2} \geq 1.5c_{a1}$

$$\psi_{ed,V} = 1.0 \quad (6.K.27)$$

For, $c_{a2} < 1.5c_{a1}$

$$\psi_{ed,V} = 0.7 + 0.3 \frac{c_{a2}}{1.5c_{a1}} \quad (6.K.28)$$

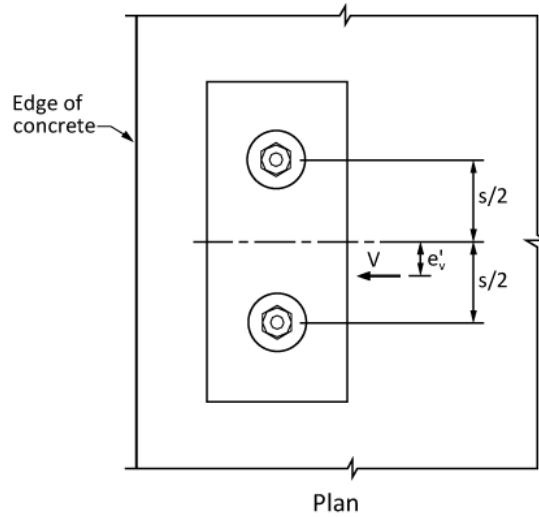


Figure 6.K.10 Definition of e_v' for a group of anchors.

K.6.2.7 For anchors located in a region of a concrete member where analysis indicates no cracking at service loads, the modification factor shall be permitted as $\psi_{c,V} = 1.4$.

For anchors located in a region of a concrete member where analysis indicates cracking at service load levels, the following modification factors shall be permitted:

$\psi_{c,V} = 1.0$ for anchors in cracked concrete with no supplementary reinforcement or edge reinforcement smaller than a No. 13 bar;

$\psi_{c,V} = 1.2$ for anchors in cracked concrete with reinforcement of a No. 13 bar or greater between the anchor and the edge; and

$\psi_{c,V} = 1.4$ for anchors in cracked concrete with reinforcement of a No. 13 bar or greater between the anchor and the edge, and with the reinforcement enclosed within stirrups spaced at not more than 100 mm.

Clarification for Section K.6.2.7: Torque-controlled and displacement-controlled expansion anchors are permitted in cracked concrete under pure shear loadings.

K.6.2.8 The modification factor for anchors located in a concrete member where $h_a < 1.5c_{a1}$, $\psi_{h,V}$ shall be computed as

$$\psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \quad (6.K.29)$$

But, $\psi_{h,V}$ shall not be taken less than 1.0.

Clarification for Section K.6.2.8: For anchors located in a concrete member where, $h_a < 1.5c_{a1}$ tests^{K.8.K.14} have shown that the concrete breakout strength in shear is not directly proportional to the member thickness h_a . The factor $\psi_{h,V}$ accounts for this effect.

K.6.2.9 Where anchor reinforcement is either developed in accordance with Sec 8.2 on both sides of the breakout surface, or encloses the anchor and is developed beyond the breakout surface, the design strength of the anchor reinforcement shall be permitted to be used instead of the concrete breakout strength in determining ϕV_n . A strength reduction factor of 0.75 shall be used in the design of the anchor reinforcement.

Clarification for Section K.6.2.9: For conditions where the factored shear force exceeds the concrete breakout strength of the anchor(s) in shear, or where the breakout strength is not evaluated, the nominal strength can be that of anchor reinforcement properly anchored as shown in Figure 6.K.11(a) and (b). To ensure yielding of the anchor reinforcement, the enclosing anchor reinforcement in Figure 6.K.11(a) should be in contact with the anchor and placed as close as practicable to the concrete surface. The research^{K.14} on which the provisions for enclosing reinforcement (see Figure 6.K.11(a)) are based was limited to anchor reinforcement with maximum diameter similar to a No. 16 bar. The larger bend radii associated with larger bar diameters may significantly reduce the effectiveness of the anchor reinforcement, and therefore anchor reinforcement with a diameter larger than No. 19 is not recommended.

The reinforcement could also consist of stirrups and ties (as well as hairpins) enclosing the edge reinforcement embedded in the breakout cone and placed as close to the anchors as practicable (see Figure 6.K.11(b)). Only reinforcement spaced less than the lesser of $0.5c_{a1}$ and $0.3c_{a2}$ from the anchor centerline should be included as anchor reinforcement. In this case, the anchor reinforcement must be developed on both sides of the breakout surface. For equilibrium reasons, an edge reinforcement must be present. The research on which these provisions are based was limited to anchor reinforcement with maximum diameter similar to a No. 19 bar.

Because the anchor reinforcement is placed below where the shear is applied (see Figure 6.K.11(b)), the force in the anchor reinforcement will be larger than the shear force. In sizing the anchor reinforcement, use of a 0.75 strength reduction factor ϕ is recommended as used for shear and for strut-and-tie models. As a practical matter, the use of anchor reinforcement is generally limited to cast-in-place anchors.

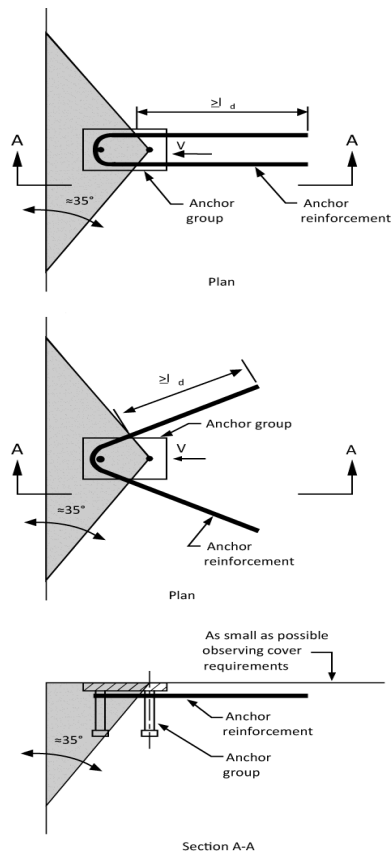


Figure 6.K.11(a) Hairpin anchor reinforcement for shear.

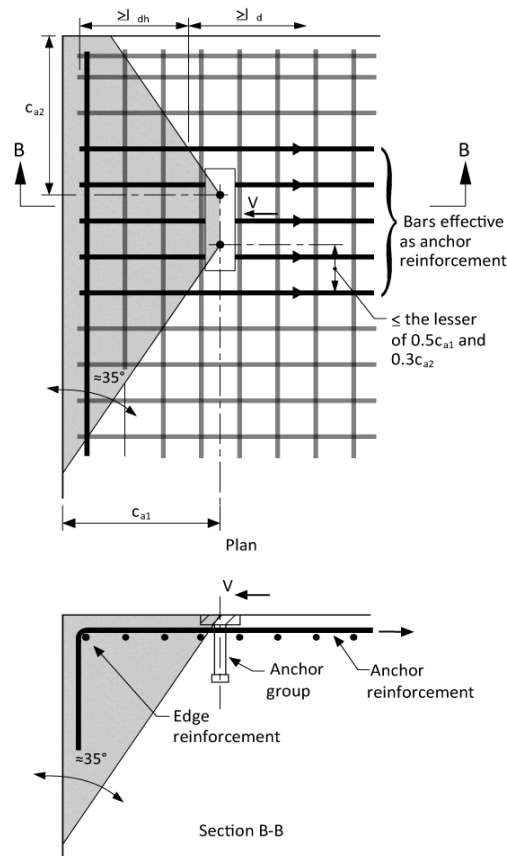


Figure 6.K.11(b) Edge reinforcement and anchor reinforcement for shear.

K.6.3 Concrete Pryout Strength of Anchor in Shear

Clarification for Section K.6.3: Concrete pryout strength of anchor in shear

Reference K.9 indicates that the pryout shear resistance can be approximated as one to two times the anchor tensile resistance with the lower value appropriate for h_{ef} less than 65 mm.

K.6.3.1 The nominal pryout strength, V_{cp} or V_{cpg} shall not exceed:

(a) For a single anchor

$$V_{cp} = k_{cp} N_{cb} \quad (6.K.30)$$

(b) For a group of anchors

$$V_{cpg} = k_{cp} N_{cbg} \quad (6.K.31)$$

Where,

$k_{cp} = 1.0$ for $h_{ef} < 65$ mm < 65 mm; and $k_{cp} = 2.0$ for $h_{ef} \geq 65$ mm.

N_{cb} and N_{cbg} shall be determined from Eq. 6.K.4 and 6.K.5, respectively.

K.7 Interaction of Tensile and Shear Forces

Unless determined in accordance with Sec K.4.3, anchors or groups of anchors that are subjected to both shear and axial loads shall be designed to satisfy the requirements of Sections K.7.1 to K.7.3. The value of ϕN_n shall be as required in Sec K.4.1.2. The value of ϕV_n shall be as defined in Sec K.4.1.2.

Clarification for Section K.7: Interaction of tensile and shear forces

The shear-tension interaction expression has traditionally been expressed as

$$\left(\frac{N_{ua}}{N_n} \right)^\zeta + \left(\frac{V_{ua}}{V_n} \right)^\zeta \leq 1.0$$

Where, ζ varies from 1 to 2. The current trilinear recommendation is a simplification of the expression where, $\zeta = 5/3$ (Figure 6.K.12). The limits were chosen to eliminate the requirement for computation of interaction effects where very small values of the

second force are present. Any other interaction expression that is verified by test data, however, can be used to satisfy Sec K.4.3.

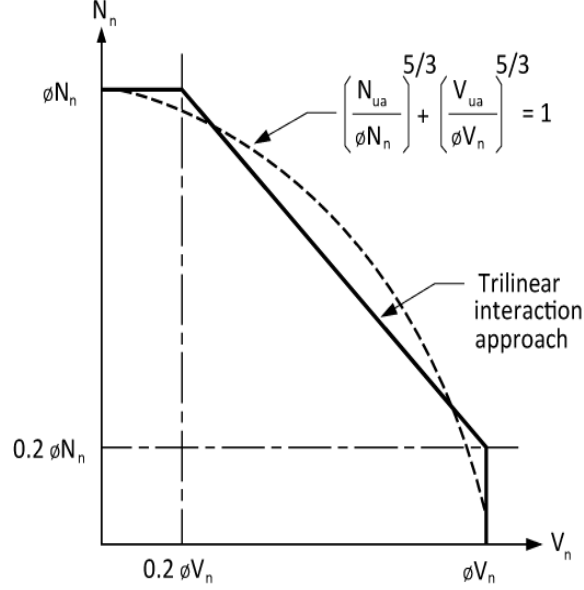


Figure 6.K.12 Shear and tensile load interaction equation.

K.7.1 If $V_{ua} \leq 0.2\phi V_n$, then full strength in tension shall be permitted: $\phi N_n \geq N_{ua}$.

K.7.2 If $N_{ua} \leq 0.2\phi N_n$, then full strength in tension shall be permitted: $\phi V_n \geq V_{ua}$.

K.7.3 If $V_{ua} > 0.2\phi V_n$ and $N_{ua} > 0.2\phi N_n$, then

$$\frac{N_{ua}}{\phi N_n} + \frac{V_{ua}}{\phi V_n} \leq 1.2 \quad (6.K.32)$$

K.8 Edge Distance, Spacing & Thickness Required Against Splitting Failure

Minimum spacing and edge distances for anchors and minimum thicknesses of members shall conform to Sections K.8.1 to K.8.6, unless supplementary reinforcement is provided to control splitting. Lesser values from product-specific tests performed in accordance with ACI 355.2 shall be permitted.

Clarification for Section K.8: Required edge distances, spacing, and thicknesses to preclude splitting failure

The minimum spacing, edge distances, and thicknesses are very dependent on the anchor characteristics. Installation forces and torques in post-installed anchors can cause splitting of the surrounding concrete. Such splitting also can be produced in subsequent torquing during connection of attachments to anchors including cast-in anchors. The primary source of values for minimum spacing, edge distances, and thicknesses of post-installed anchors should be the product-specific tests of ACI 355.2. In some cases, however, specific products are not known in the design stage. Approximate values are provided for use in design.

K.8.1 Unless determined in accordance with Sec K.8.4, minimum center-to-center spacing of anchors shall be $4d_a$ for untorqued cast-in anchors, and $6d_a$ for torque cast-in anchors and post-installed anchors.

K.8.2 Unless determined in accordance with Sec K.8.4, minimum edge distances for cast-in headed anchors that will not be torqued shall be based on specified cover requirements for reinforcement in Sec 8.1.7. For cast-in headed anchors that will be torqued, the minimum edge distances shall be $6d_a$.

Clarification for Section K.8.2: Because the edge cover over a deep embedment close to the edge can have a significant effect on the side face blowout strength of Sec K.5.4, in addition to the normal concrete cover requirements, it may be advantageous to use larger cover to increase the side-face blowout strength.

K.8.3 Unless determined in accordance with Sec K.8.4, minimum edge distances for post-installed anchors shall be based on the greater of specified cover requirements for reinforcement in Sec 8.1, or minimum edge distance requirements for the products as determined by tests in accordance with ACI 355.2, and shall not be less than 2.0 times the maximum aggregate size. In the absence of product-specific ACI 355.2 test information, the minimum edge distance shall be taken as not less than:

Undercut anchors	$6d_a$
Torque-controlled anchors	$8d_a$
Displacement-controlled anchors	$10d_a$

Clarification for Section K.8.3: Drilling holes for post-installed anchors can cause microcracking. The requirement for a minimum edge distance twice the maximum aggregate size is to minimize the effects of such microcracking.

K.8.4 For anchors where installation does not produce a splitting force and that will remain untorqued, if the edge distance or spacing is less than those specified in Sections K.8.1 to K.8.3, calculations shall be performed by substituting for d_a a smaller value d'_a that meets the requirements of Sections K.8.1 to K.8.3. Calculated forces applied to the anchor shall be limited to the values corresponding to an anchor having a diameter of d'_a .

Clarification for Section K.8.4: In some cases, it may be desirable to use a larger diameter anchor than the requirements on K.8.1 to K.8.3 permit. In these cases, it is permissible to use a larger-diameter anchor provided the design strength of the anchor is based on a smaller assumed anchor diameter, d'_a .

K.8.5 The value of h_{ef} for an expansion or undercut post-installed anchor shall not exceed the greater of 2/3 of the member thickness and the member thickness minus 100 mm.

Clarification for Section K.8.5: This minimum thickness requirement is not applicable to through-bolts because they are outside the scope of Appendix K. In addition, splitting failures are caused by the load transfer between the bolt and the concrete. Because through-bolts transfer their load differently than cast-in or expansion and undercut anchors, they would not be subject to the same member thickness requirements. Post-installed anchors should not be embedded deeper than 2/3 of the member thickness.

K.8.6 Unless determined from tension tests in accordance with ACI 355.2, the critical edge distance, C_{ac} , shall not be taken less than:

Undercut anchors	$2.5h_{ef}$
Torque-controlled anchors	$4h_{ef}$
Displacement-controlled anchors	$4h_{ef}$

Clarification for Section K.8.6: The critical edge distance C_{ac} is determined by the corner test in ACI 355.2. Research has indicated that the corner-test requirements are not met with $C_{a,min} = 1.5h_{ef}$ for many expansion anchors and some undercut anchors because installation of these types of anchors introduces splitting tensile stresses in the concrete that are increased during load application, potentially resulting in a premature splitting failure. To permit the design of these types of anchors when product-specific information is not available, conservative default values for C_{ac} are provided.

K.8.7 Project drawings and project specifications shall specify use of anchors with a minimum edge distance as assumed in design.

K.9 Installation of Anchors

Clarification for Section K.9: Installation of anchors

Many anchor performance characteristics depend on proper installation of the anchor. Anchor strength and deformation capacity can be assessed by acceptance testing under ACI 355.2. These tests are carried out assuming that the manufacturer's installation directions will be followed. Certain types of anchors can be sensitive to variations in hole diameter, cleaning conditions, orientation of the axis, magnitude of the installation torque, crack width, and other variables. Some of this sensitivity is indirectly reflected in the assigned ϕ values for the different anchor categories, which depend in part on the results of the installation safety tests. Gross deviations from the ACI 355.2 acceptance testing results could occur if anchor components are incorrectly exchanged, or if anchor installation criteria and procedures vary from those recommended. Project specifications should require that anchors be installed according to the manufacturer's recommendations.

K.9.1 Anchors shall be installed in accordance with the project drawings and project specifications.

K. 10 References of Appendix K

- K.1. ANSI/ASME B1.1, "Unified Inch Screw Threads (UN and UNR Thread Form)," ASME, Fairfield, NJ, 1989.
- K.2. ANSI/ASME B18.2.1, "Square and Hex Bolts and Screws, Inch Series," ASME, Fairfield, NJ, 1996.
- K.3. ANSI/ASME B18.2.6, "Fasteners for Use in Structural Applications," ASME, Fairfield, NJ, 1996.
- K.4. Cook, R. A., and Klingner, R. E., "Behavior of Ductile Multiple-Anchor Steel-to-Concrete Connections with Surface-Mounted Baseplates," *Anchors in Concrete: Design and Behavior*, SP-130, American Concrete Institute, Farmington Hills, MI, 1992, pp. 61-122.
- K.5. Cook, R. A., and Klingner, R. E., "Ductile Multiple-Anchor Steel-to-Concrete Connections," *Journal of Structural Engineering*, ASCE, V. 118, No. 6, June 1992, pp. 1645-1665.
- K.6. Lotze, D.; Klingner, R. E.; and Graves III, H. L., "Static Behavior of Anchors under Combinations of Tension and Shear Loading," *ACI Structural Journal*, V. 98, No. 4, July-Aug. 2001, pp. 525-536.

-
- K.7. Primavera, E. J.; Pinelli, J.-P.; and Kalajian, E. H., “Tensile Behavior of Cast-in-Place and Undercut Anchors in High-Strength Concrete,” *ACI Structural Journal*, V. 94, No. 5, Sept.-Oct. 1997, pp. 583-594.
 - K.8. Design of Fastenings in Concrete, Comité Euro-International du Béton (CEB), Thomas Telford Services Ltd., London, Jan. 1997.
 - K.9. Fuchs, W.; Eligehausen, R.; and Breen, J., “Concrete Capacity Design (CCD) Approach for Fastening to Concrete,” *ACI Structural Journal*, V. 92, No. 1, Jan.-Feb. 1995, pp. 73-93. Also discussion, *ACI Structural Journal*, V. 92, No. 6, Nov.-Dec. 1995, pp. 787-802.
 - K.10. Eligehausen, R., and Balogh, T., “Behavior of Fasteners Loaded in Tension in Cracked Reinforced Concrete,” *ACI Structural Journal*, V. 92, No. 3, May-June 1995, pp. 365-379.
 - K.11. “Fastenings to Concrete and Masonry Structures, State of the Art Report,” Comité Euro-International du Béton (CEB), Bulletin No. 216, Thomas Telford Services Ltd., London, 1994.
 - K.12. Klingner, R.; Mendonca, J.; and Malik, J., “Effect of Reinforcing Details on the Shear Resistance of Anchor Bolts under Reversed Cyclic Loading,” *ACI JOURNAL*, Proceedings V. 79, No. 1, Jan.-Feb. 1982, pp. 3-12.
 - K.13. ACI Committee 349, “Code Requirements for Nuclear Safety Related Concrete Structures (ACI 349-01),” American Concrete Institute, Farmington Hills, MI, 2001, 134 pp.
 - K.14. Eligehausen, R.; Mallee, R.; and Silva, J., *Anchorage in Concrete Construction*, Ernst & Sohn (J. T. Wiley), Berlin, May 2006, 380 pp.
 - K.15. Eligehausen, R.; Fuchs, W.; and Mayer, B., “Load Bearing Behavior of Anchor Fastenings in Tension,” *Betonwerk + Fertigteiltechnik*, 12/1987, pp. 826-832, and 1/1988, pp. 29-35.
 - K.16. Eligehausen, R., and Fuchs, W., “Load Bearing Behavior of Anchor Fastenings under Shear, Combined Tension and Shear or Flexural Loadings,” *Betonwerk + Fertigteiltechnik*, 2/1988, pp. 48-56.
 - K.17. Farrow, C. B., and Klingner, R. E., “Tensile Capacity of Anchors with Partial or Overlapping Failure Surfaces: Evaluation of Existing Formulas on an LRFD Basis,” *ACI Structural Journal*, V. 92, No. 6, Nov.-Dec. 1995, pp. 698-710.

-
- K.18. PCI Design Handbook, 5th Edition, Precast/Prestressed Concrete Institute, Chicago, IL, 1999.
- K.19. “AISC Load and Resistance Factor Design Specifications for Structural Steel Buildings,” Dec. 1999, 327 pp.
- K.20. Zhang, Y.; Klingner, R. E.; and Graves III, H. L., “Seismic Response of Multiple-Anchor Connections to Concrete,” ACI Structural Journal, V. 98, No. 6, Nov.-Dec. 2001, pp. 811-822.
- K.21. Lutz, L., “Discussion to Concrete Capacity Design (CCD) Approach for Fastening to Concrete,” ACI Structural Journal, Nov.-Dec. 1995, pp. 791-792. Also authors’ closure, pp. 798-799.
- K.22. Asmus, J., “Verhalten von Befestigungen bei der Versagensart Spalten des Betons (Behavior of Fastenings with the Failure Mode Splitting of Concrete),” dissertation, Universität Stuttgart, Germany, 1999.
- K.23. Kuhn, D., and Shaikh, F., “Slip-Pullout Strength of Hooked Anchors,” Research Report, University of Wisconsin-Milwaukee, submitted to the National Codes and Standards Council, 1996.
- K.24. Furche, J., and Eligehausen, R., “Lateral Blow-out Failure of Headed Studs Near a Free Edge,” Anchors in Concrete—Design and Behavior, SP-130, American Concrete Institute, Farmington Hills, MI, 1991, pp. 235-252.
- K.25. Shaikh, A. F., and Yi, W., “In-Place Strength of Welded Studs,” PCI Journal, V. 30, No. 2, Mar.-Apr. 1985.

PART VI
Appendix L
Information on Steel Reinforcement

Information on sizes, areas, and weights of various steel reinforcement is presented below to facilitate the use of Bangladesh National Building Code.

Table 6.L.1: Sizes, Areas, and Weights of Reinforcing Bars (ASTM Standard)

Bar Size, No.*	Nominal Diameter, mm	Nominal Area, mm ²	Nominal Mass, kg/m
10	9.5	71	0.560
13	12.7	129	0.994
16	15.9	199	1.552
19	19.1	284	2.235
22	22.2	387	3.042
25	25.4	510	3.973
29	28.7	645	5.060
32	32.3	819	6.404
36	35.8	1006	7.907
43	43.0	1452	11.38
57	57.3	2581	20.24

**Bar numbers approximate the number of millimeters of the nominal diameter of the bar. Within the text in Chapters, the referred ϕ (diameter in mm) of bars correspond to the figures in the first column (Bar size, no.)*