Chapter 9 Commentary

CONCRETE STRUCTURE DESIGN REQUIREMENTS

9.1 REFERENCE DOCUMENT: The main concern of Chapter 9 is the proper detailing of reinforced concrete construction for earthquake resistance. The bulk of the detailing requirements in this chapter are contained in ACI 318. The commentary for ACI 318 contains a valuable discussion of the rationale behind detailing requirements that is not repeated here.

9.1.1 Modifications to ACI 318: The modifications noted for ACI 318 are: changes in load factors necessary to coordinate with the equivalent yield basis of this document; additional definitions and provisions necessary for seismic design requirements for structural systems composed of precast elements; and changes that incorporate certain features of the detailing requirements for reinforced concrete that have been adopted into the 1997 *Uniform Building Code* and the 2000 *International Building Code*.

Included as Sec. 9.1.1.4 are two statements on reinforced concrete structural systems incorporating precast concrete elements. One statement refers to Sec. 9.1.1.12 where a new Sec.21.11 is inserted in ACI 318 to cover the design requirements for precast concrete special moment frames and special structural walls. The second statement is based on requirements from 1997 Uniform Building Code and provides design requirements for structures having precast concrete gravity load carrying systems .

For precast concrete special moment frames and special structural walls two design alternatives are permitted. One design alternative is emulation of monolithic reinforced concrete construction. The other alternative is the use of the unique properties of precast elements interconnected predominately by dry joints. For the first alternative Sec. 9.1.1.12 defines in provisions 21.11.2, 21.11.3 and 21.11.5 design procedures ensuring that the resulting structural systems have strength and stiffness characteristics equivalent to those for monolithic reinforced concrete construction. The existing code requirements for monolithic construction then apply for all but the connections. The second alternative, use of the unique properties of precast elements interconnected predominately by dry joints, was covered in an Appendix to Chapter 9 in the 1997 Provisions. Recent advances in the understanding of the seismic behavior of precast/prestressed concrete frame and wall structures, resulting from NIST (Cheok et al.1991,1997,1998), US-PRESSS (Priestley et al., 1991,1996,1999, Nakaki et al.,1999) and JAPAN-PRESSS research programs and the codification of acceptance testing procedures for verification of acceptable behavior by ITG-1 of ACI, 1999, have made possible the elimination of the penalties on the use of precast/prestressed concrete construction that were contained in the Appendix to the 1997 Provisions and the inclusion in Sec. 9.1.1.12 of a new provision 21.11.4 containing appropriate requirements for precast/prestressed concrete seismic-force-resisting systems based entirely on amendments to ACI 318.

Procedures for design of a seismic-force-resisting structural system composed of precast elements interconnected predominately by dry joints require prior acceptance testing of modules of the generic structural system because with the existing state-of-knowledge, it is inappropriate to propose code provisions without such verification. The complexity of structural systems, configurations and details possible with precast concrete elements requires:

1. Selecting functional and compatible details for connections and members that are reliable and can be built with acceptable tolerances;
2. Verifying experimentally the inelastic force-deformation relationships for welded, bolted, or grouted connections proposed for the seismic resisting elements of the building; and
3. Analyzing the building using those connection relationships and the inelastic reversed cyclic loading effects imposed by the anticipated earthquake ground motions.

Research conducted to date (Cheok and Lew, 1991; Elliott et al, 1992; Englekirk, 1987; French et al, 1989; BSSC, 1987; Hawkins and Englekirk, 1987; Jayashanker and French, 1988; Mast, 1992; Nakaki and Englekirk, 1991; Neille, 1977; New Zealand Society, 1991; Pekau and Hum, 1991; Powell et al, 1993; Priestley, 1991; Priestley and Tao, 1992; Stanton et al, 1986; Stanton et al, 1991) documents concepts for design using dry connections and the behavior of structural systems and subassemblages composed of precast elements both at and beyond peak strength levels for non-linear reversed cyclic loadings, and provides the basis for the provisions for interconnected element design in Sec. 21.11.2, and Sec. 21.11.4 of Sec. 9.1.1.12.

Emulation of Monolithic Construction Using Strong Connections: For emulation of the behavior of monolithic reinforced concrete construction, Sec. 9.1.1.12 provides two alternatives. Sec. 21.1.3 in Sec. 9.1.1.12 covers structural systems with either "wet" or dry connections. Sec. 21.11.3.2 and 21.11.5 cover structural systems with "strong" connections.

For frame systems that use strong connections, Sec. 21.11.3.2 and 21.11.5 , the different connection categories envisaged are shown in Figure C9.1.1-1. Considerable freedom is given to locating the nonlinear action zones (plastic hinges), along the length of the precast member. Those hinges must be considered to have a length not less than half the member depth and must be separated from the connection by a distance of at least three quarters of the member depth. Wet-joint connections are permitted at the strong connection but not at the hinge location.

Provision 21.11.5.1 makes the strength required for a strong connection dependent on the distances hinges are separated from that connection, the strengths of those hinges and the nonlinear deformation mechanism envisaged. The conditions described by 21.11.5.1 for a beam to continuous column connection are shown in Figure C9.1.1- 2, which is an adaption of Figure R 21.3.4 of ACI 318. Because the strong connection must not yield or slip; its nominal strengths, S_n, in both flexure and shear must be greater than those corresponding to development of the probable strengths M_{pr1} and M_{pr2} at the hinge locations. Figure C9.1.1-2b, illustrates the situation for flexure. Per ACI 318 moments M_{pr1} and M_{pr2} are determined using a strength reduction factor of 1.0 and reinforcing steel stresses of at least $1.25fy$.

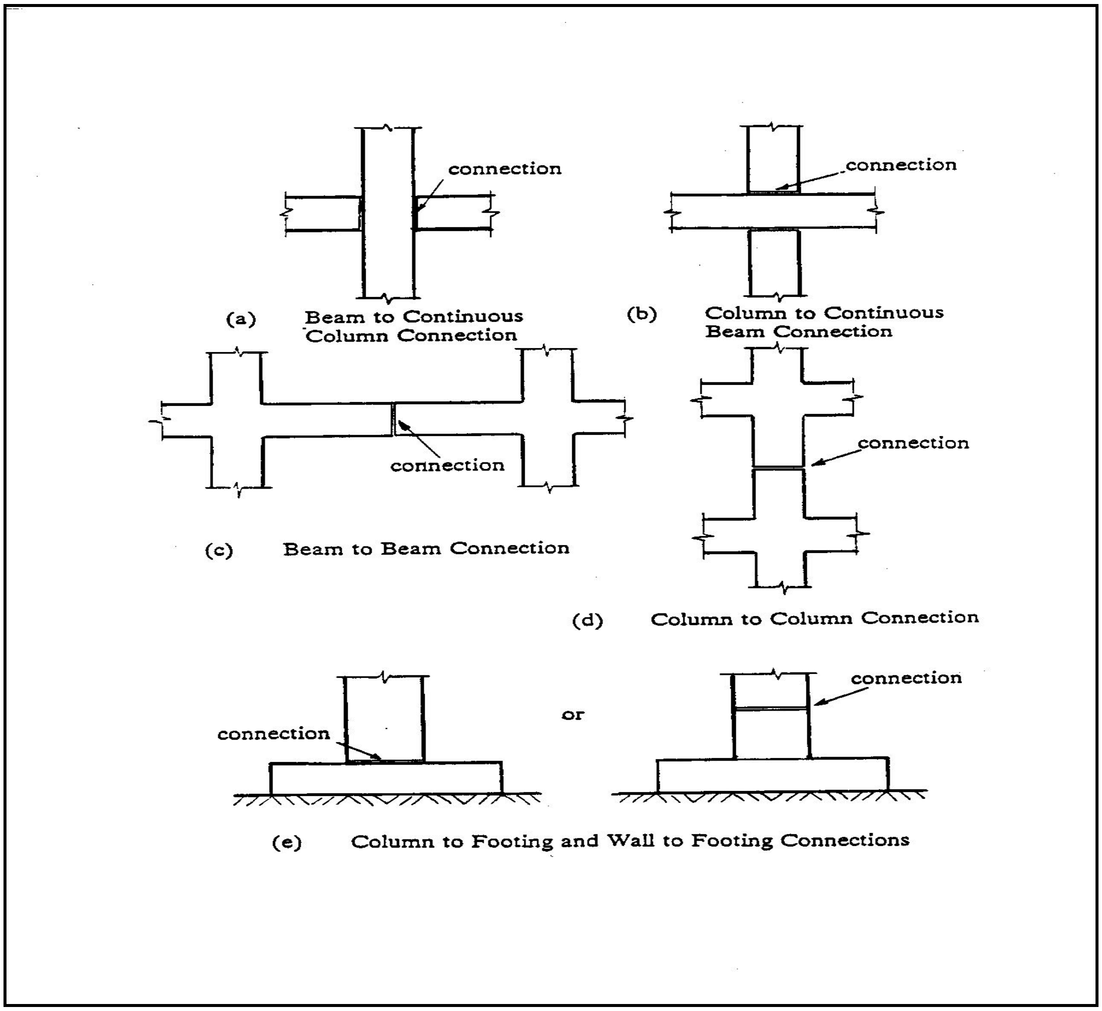

FIGURE C9.1.1-1 Connection categories.

For columns above the ground floor, moments at a joint may be limited by flexural strengths of the beams framing into that joint. However, for a strong column-weak beam deformation mechanism, dynamic inelastic analysis and studies of strong motion measurements have shown that beam end moments are not equally divided between top and bottom columns even where the columns have equal stiffness. Elastic analysis predicts moments as shown in Figure C9.1. 1-3b. Accordingly, provision 21.11.5.4 is included for the mid-height column connection. Further background information on the *Provisions* is provided in Ghosh et al.,1997.

Emulation of Monolithic Construction Using Ductile Connections: In Sec. 9.1.1.12 provision 21.11.3.1 covers the situation for both *frame* and panel systems where the connections used have adequate nonlinear response characteristics and it is not necessary to ensure plastic hinges remote form the connections. Usually physical testing is required to prove that a connection has the necessary nonlinear response characteristics. Warnes (1992) and Yee (1991) have documented one connection type that has such characteristics.

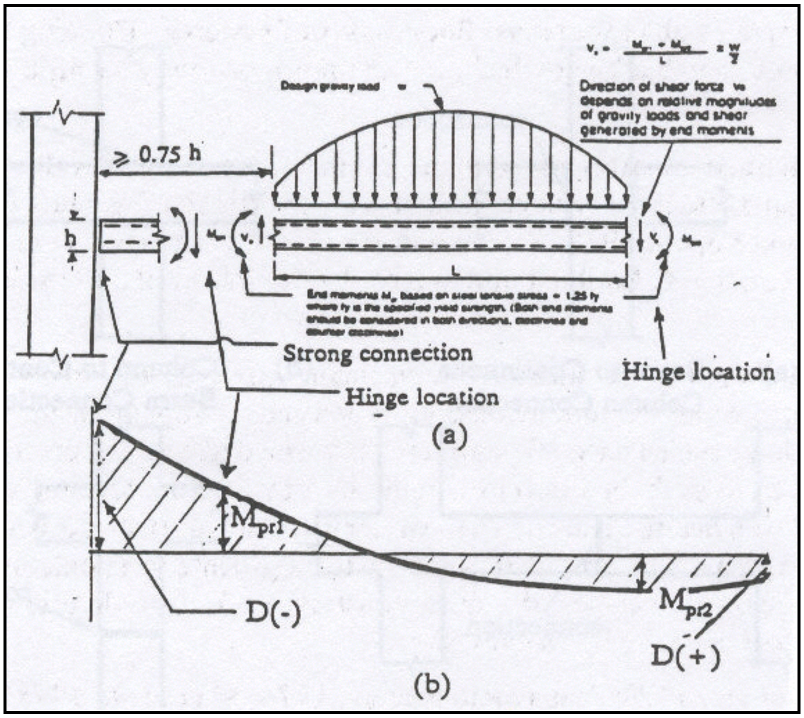

FIGURE C9.1.1-2 Design forces for strong connections between beams and continuous columns.

The designer needs to consider the likely deformations of any proposed precast structure vis-a-vis those of the same structure composed of monolithic reinforced concrete before claiming that the precast form emulates monolithic construction. For example, the designer might propose a shear wall that is composed of multiple precast panels over its length and height that are connected vertically but not horizontally. Under lateral load that wall would have a deformed shape not emulating that for a solid cast-in-place monolithic wall. Therefore the wall could not be designed using this provision.

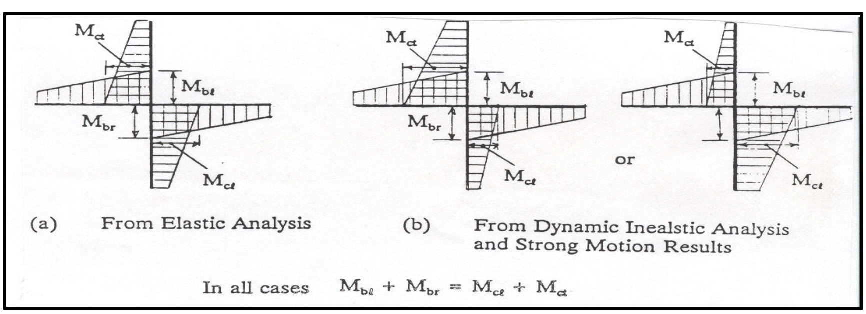

FIGURE C9.1.1-3 Moments at beam-to-column connections.

Sec. 21.11.3.1 in Sec. 9.1.1.12 recognizes that if the monolithic *wall* of Figure C9.1.1-4a, Part a, is composed of precast elements, as shown in Figure C9.1.1-4b, then the shear force acting on the connection at A-A can be limited by the shear capacity of the precast element above A-A, by the shear for slip along the connection, or by the probable connection moment capacity, M_{pr}. That moment corresponds to the value of H that causes a stress of $1.25f_y$ in the boundary reinforcement continuous across A-A. When the moment due to H causes a stress of $1.25f_y$ in the boundary reinforcement, the shear causing slip along the connection is less than if the steel stress was less than $1.25f_y$. The shear to cause slip decreases as the crack width increases. Only when the steel stress is limited to f_y can the shear *strength* be taken as that calculated by Sec. 11.7 of ACI 318. The probable shear *strength* is taken as that documented by Mueller (1989) and Wood (1990) for precast and monolithic *shear walls*, respectively.

The shear carrying mechanism of the monolithic *wall* of Figure C9.1.1-4a and that of the precast *wall* of Figure C9.1.1-4b are distinctly different when the overturning moment causes yielding of the boundary reinforcement and therefore opening of the horizontal connections. Lateral shears can then be transferred through compressed concrete only and the precast *wall* must be provided with horizontal reinforcement at the upper edge of the panel sufficient to balance the horizontal *component* of the force in the compression diagonal.

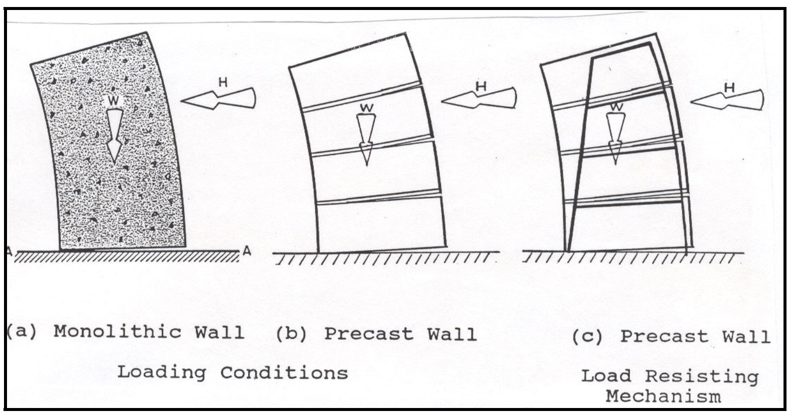

FIGURE C9.1.1-4 Conditions for walls.

Use of Prestressing Tendons: Sec. 9.1.1.5 defines conditions under which prestressing tendons can be used, in conjunction with deformed reinforcing bars, in frames resisting earthquake forces. As documented in Ishizuka and Hawkins (1987), if those conditions are met no modification is necessary to the R and C_d factors of Table 5.2.2 when prestressing is used. Satisfactory seismic performance can be obtained when prestressing amounts greater than those permitted by Sec. 9.1.1.5 are used. However, as documented by Park and Thompson (1977) and Thompson and Park (1980) and required by the combination of New Zealand Standards 3101:1982 and 4203:1992, ensuring that satisfactory performance requires modification of the R and C_d factors.

Structures Having Precast Concrete Gravity Load Carrying Systems: Sec. 9.1.1.4 defines conditions governing the design of structures, such as precast concrete parking garages, which have precast concrete gravity load carrying systems combined with either precast or cast-in-place seismic-force resisting systems. Further information on the background to the *Provisions* is provided in Ghosh et al., 1997. In the 1997 Provisions Sec. 21.2.1.7 in Sec. 9.1.1.5 required use of one of two methods with the first method differing from that specified in the 2000 Provisions and the second method being the same for both the 1997 and 2000 Provisions. The requirement in the first method of the 1997 Provisions that the span of the diaphragm or diaphragm segment between seismic-force resisting systems not exceed three times the width of the diaphragm or diaphragm segment has been deleted. The arbitrary 3:1 limit was imposed because of a lack of technical data. Based on analytical studies that requirement in the 2000 Provisions has been replaced by a requirement intended to ensure that the diaphragm remains elastic under the maximum design displacement and that there is sufficient chord reinforcement in the diaphragm to limit its maximum lateral deformation to 0.75 percent of the story height.

Structures Having Seismic-Force-Resisting Systems Utilizing Interconnected Precast Elements. Precast concrete seismic-force-resisting systems can be utilized only if: (1) substantiating experimental evidence of acceptable performance of that generic system has been demonstrated through cyclic tests on typical modules of that system; and (2) it is demonstrated through non-linear response history analysis using the evidence from those module tests that the system will perform satisfactorily under the Maximum Considered Earthquake Ground Motions. For special precast concrete moment frames substantiating experimental must satisfy the conditions specified in ACI Provisional Standard T1.1-99, (ACI Innovation Task Group 1 and Collaborators, 1999). Special precast concrete structural wall systems must satisfy similar conditions with the limiting drift ratio being a function of the height to width ratio for the wall as documented in Seo et al., 1998. The validity of the use of precast concrete seismic-force resisting systems has been demonstrated by the results of the recently completed PRESSS program (Priestley, 1991, 1996) and five story PRESSS building test (Priestley et al. 1999) and by analytical studies of precast/ prestressed concrete moment frames and structural walls (Cheok et al., 1998, El-Sheikh et al., 1999, Kurama et al. 1999).

Connections: Connections are classified into two types, X and Z in provision 21.11.6 in Sec. 9.1.1.12 in accordance with the ductilities achieved in acceptance tests on generic forms of those connections. Detailed information on performance of various connection types is contained in Schultz and Magana, 1996 and Pincheria et al., 1998.

9.2 ANCHORING TO CONCRETE:

9.2.1 Scope:

9.2.1.1: The *Provisions* are restricted in scope to structural anchors that transmit structural loads from attachments into concrete members. The levels of safety defined by the combinations of load factors and ϕ factors are appropriate for structural applications. Other standards can require more stringent safety levels during temporary handling.

9.2.1.2: The wide variety of shapes and configurations of specialty inserts makes it difficult to prescribe generalized tests and design equations for many insert types. Hence, they have been excluded from the scope of the Provisions. Bonded anchors, held in place by grout, epoxy, resins, or other chemicals are widely used and can perform adequately. However, at this time such anchors are outside the scope of the *Provisions*.

9.2.1.3: Typical cast-in headed studs and headed bolts with geometries consistent with ANSI/ASME B1.1 (1989), B18.2.1 (1996), and B18.2.6 (1996) have been tested and have 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.

9.2.1.4: Post-installed fasteners designed using the Provisions must first be qualified in accordance with a comprehensive set of tests. The tests shall include reference tests, reliability tests, and service-condition tests. The reference tests should establish basic anchor performance and capacity for failure modes, including concrete breakout, steel rupture, or pullout. The reliability tests should establish fastener performance under adverse installation conditions expected to be found under field conditions, and should provide the information necessary to establish the ϕ factors to be used in Sec. 9.2.4.4 or 9.2.4.5. Service-condition tests should determine if the fasteners are appropriate for use under these design provisions with respect to edge distance, fastener spacing, shear capacity, pryout, splitting near an edge, and seismic capacity.

Standards for qualification tests with these attributes are under preparation in ACI (ACI 355.2) and will be subsequently processed in ASTM (Z5819Z). These documents contain requirements for testing and certification of post-installed fasteners for both cracked and uncracked concrete applications including qualification for use in seismic applications. Anchor prequalification tests should require that anchors qualified for use in cracked concrete perform well in cracks whose width is consistent with that intended by the requirements of Sec. 10.6.4 of ACI 318.

9.2.1.5: The exclusion from the scope of load applications producing high cycle fatigue or extremely short duration impact (such as blast or shock wave) are not meant to exclude seismic load effects. Sec. 9.2.3.3 presents additional requirements for design when seismic loads are included.

9.2.2 Notations and Definitions:

9.2.2.1 Notations:

A_{se} = 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 (1989) defines A_{se} as:

$$A_{se} = \frac{\pi}{4}\left(d_0 - \frac{0.9743}{n_t}\right)^2 \qquad \text{(C9.2.2.1)}$$

where n_t =is the number of threads per inch.

e_n = Actual eccentricity of a normal force on an attachment

h_{ef} = Effective embedment depths for a variety of anchor types are shown in Figure C9.2.2.1.

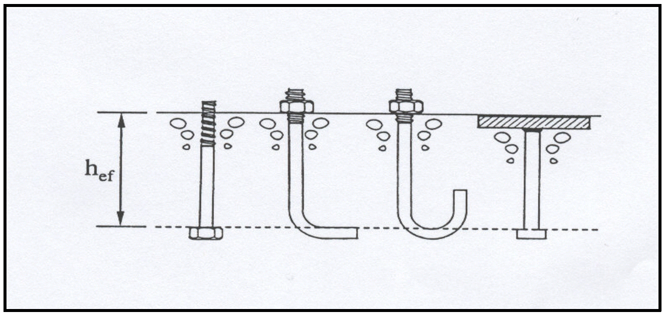

FIGURE C9.2.2.1 Types of cast-in-place anchors.

9.2.2.2 Definitions:

5 Percent Fractile – The determination of the coefficient K associated with the 5 percent fractile, $\bar{x} - \sigma$, $\bar{x} - K\sigma$, depends on the number of tests, n, used to compute $\bar{x}$ and σ. Values of K 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 Sec. 9.2.4.2 is the same as the characteristic strength in the anchor prequalification tests.

9.2.3 General Requirements:

9.2.3.1: When 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 assuming the theory of elasticity will be conservative. The works by Cook and Klingner (Feb. 1992), Cook and Klingner (June 1992), and Lotze and Klingner (1997) discuss non-linear analysis, using theory of plasticity, for the determination of the capacities of ductile anchor groups.

9.2.3.3: Post-installed structural anchors are required to be qualified for moderate or high seismic risk zone usage by passing anchor prequalification simulated seismic tests. In addition, the design of anchors in zones of moderate or high seismic risk is based on a more conservative approach by the introduction of a 0.75 factor on the design strength ϕN_n and ϕV_n, and by requiring ductile failures. Alternatively, a higher value of anchor strength can be used if the attachment being fastened is designed to ensure ductile yielding of the attachment at a load well below the minimum probable anchor strength.

For an anchor to be acceptable in seismic loading situations the system is required to have adequate ductility. The anchor is required to demonstrate the capacity to undergo large displacements through several cycles as specified in the anchor prequalification seismic simulation tests. If the anchor cannot meet these requirements, then the attachment is required to be designed so as to yield at a load well below the anchor capacity. In designing attachments for adequate ductility, the ratio of yield to ultimate load capacity should be considered. A connection element could yield only to result in a secondary failure as one or more elements strain harden if the ultimate load capacity is excessive when compared to the yield capacity.

Under seismic conditions, the direction of shear loading may not be predictable. The full shear load should be assumed in any direction for a safe design.

9.2.3.5: A limited number of tests of cast-in and post-installed anchors in high-strength concrete (see Primavera, Pinelli, and Kalajian (1997)) indicate that the design procedures contained in the Provisions become unconservative, particularly for cast-in anchors, at f'_c = 11,000 to 12,000 psi. Until further test results are available, an upper limit of f'_c = 10,000 psi was imposed in the design of cast-in anchors. This is consistent with Chapters 11 and 12 of ACI 318. The anchor prequalification standard does not require testing of post-installed anchors in concrete with f'_c > 8,000 psi since some post-installed anchors may have difficulty expanding in very high strength concretes. Because of this, f'_c is limited to 8000 psi in the design of post-installed anchors.

9.2.4 General Requirements for Strength of Structural Anchors:

9.2.4.1: This section provides 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 C9.2.4.1-1 and C9.2.4.1-2. Comprehensive discussions of anchor failure modes are included in *Design of Fastenings in Concrete* (1997), Fuchs, Eligehausen, Breen (1995), and Eligehausen and Balogh (1995). Any model that complies with the requirements of Sec. 9.2.4.2 and 9.2.4.3 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 method of Sec. 9.2.5.2 and 9.2.6.2 is acceptable. The anchor strength is also dependent on the pullout strength of Sec. 9.2.5.3, the side-face blowout strength of Sec. 9.2.5.4 and the minimum spacings and edge distances of Sec. 9.2.8. The design of anchors for tension recognizes that the strength of anchors is sensitive to appropriate installation; installation requirements are included in Sec. 9.2.9. 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 the anchor prequalification tests.

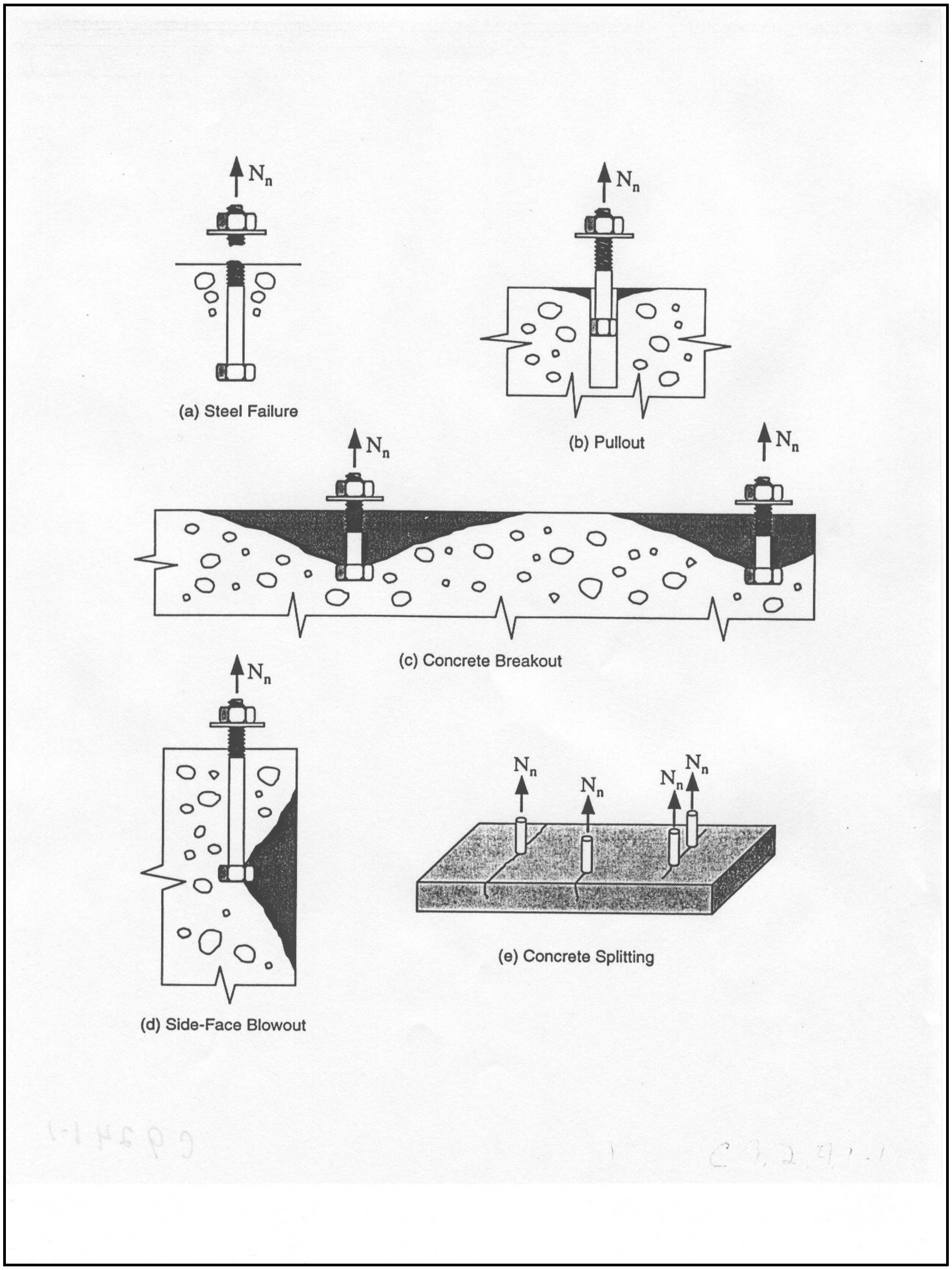

FIGURE C9.2.4.1-1 Failure modes for anchors under tensile loading.

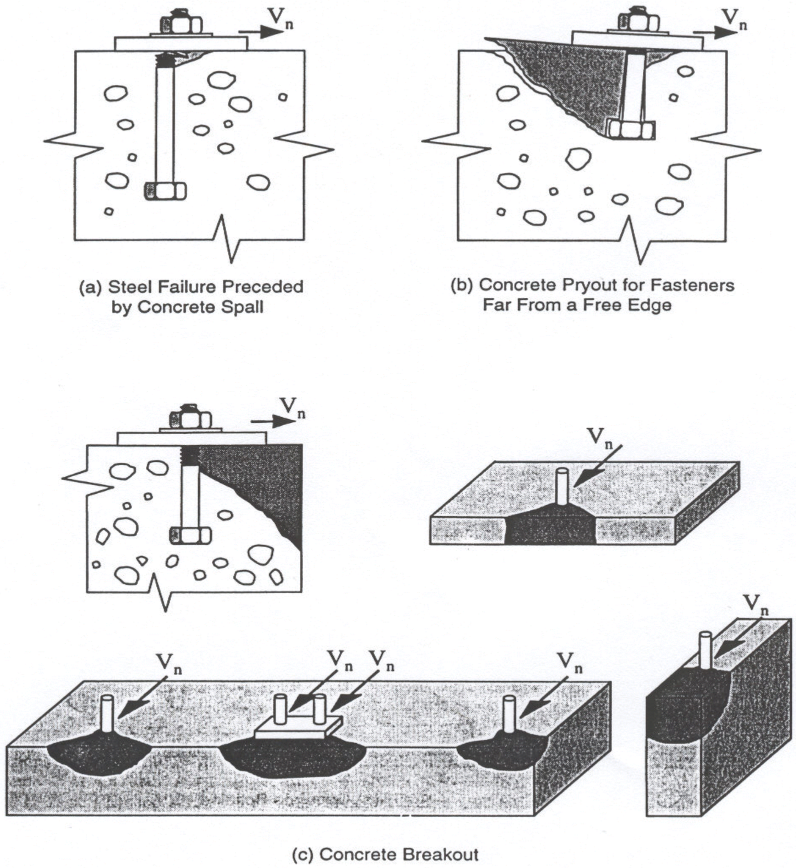

FIGURE C9.2.4.1-2 Failure modes for anchors under shear loading.

Test procedures can also be used to determine the single-anchor breakout strength in tension and in shear. However, the test results 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. 9.2.4.2. 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.

9.2.4.2 and 9.2.4.3: These sections 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. 9.2.4.2 as long as sufficient data are available to verify the model.

9.2.4.2.1: The addition of supplementary reinforcement in the direction of the load, confining reinforcement, or both, can greatly enhance the strength and ductility of the anchor connection. Such enhancement is practical with cast-in anchors such as those used in precast sections.

The shear strength of headed anchors located near the edge of a member can be significantly increased with appropriate supplementary reinforcement. *Design of Fastenings in Concrete* (1997), *Fastenings in Concrete and Masonry Structures, State of the Art Report* (1994)and Klingner, Mendonca, and Malik (1982) provide substantial information on design of such reinforcement. The effect of such supplementary reinforcement is not included in the anchor prequalification tests or in the concrete breakout calculation method of Sec. 9.2.5.2 and 9.2.6.2. The designer has to rely on other test data and design theories in order to include the effects of supplementary reinforcement.

For anchors exceeding the limitations of Sec. 9.2.4.2.2, for situations where geometric restrictions limit breakout capacity, or both, reinforcement proportioned to resist the total load, oriented in the direction of load, within the breakout prism and fully anchored on both sides of the breakout planes, may be provided instead of calculating breakout capacity.

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 Sec. 9.2.6.2.1)

9.2.4.2.2: The method for concrete breakout design included as "considered to satisfy" Sec. 9.2.4.2 was developed from the Concrete Capacity Design (CCD) Method (see Fuchs, Eligehausen and Breen (1995), and Eligehausen and Balogh (1995), which was an adaptation of the κ Method (see Eligehausen, Fuchs and Mayer (1987), and (Eligehausen and Fuchs (1988), and is considered to be accurate, relatively easy to apply, and capable of extension to irregular layouts. The CCD Method predicts the load-bearing capacity 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 multiplying by factors which 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 degrees (Figures C.9.2.4.2.2-1and C9.2.4.2.2-2).

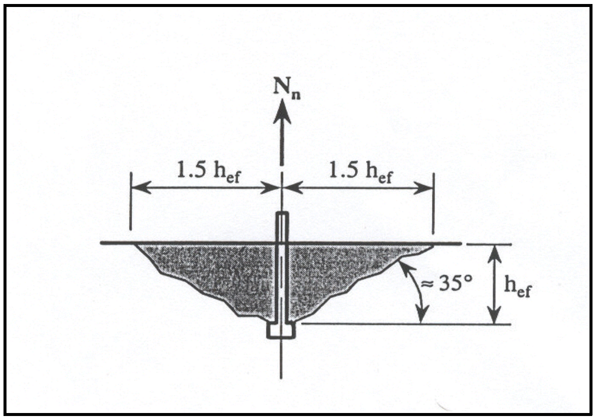

FIGURE C9.2.4.2.2-1 Breakout cone for tension.

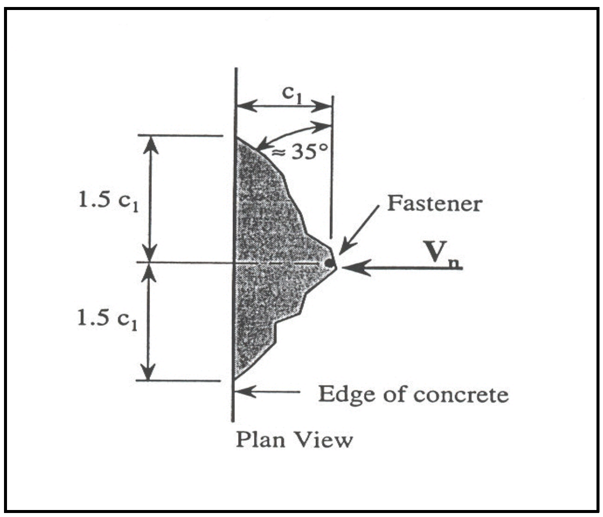

FIGURE C9.2.4.2.2-2 Breakout cone for shear.

9.2.4.4: The ϕ factor for failure of ductile elements is indicative of less variability in steel tension failures than concrete breakout failures, and the greater amount of warning with a ductile failure. 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 no greater than 75 percent of the minimum anchor design strength (See Sec. 9.2.3.3.4). For anchors governed by the more brittle concrete breakout or blowout failure, two conditions are recognized. If supplementary reinforcement is provided to tie the failure prism into the structural member (Condition A), more ductility is present than in the case where such supplementary reinforcement is not present (Condition B). Design of supplementary reinforcement is discussed in Sec. 9.2.4.2.1 and the References by Primavera, Pinelli, and Kalajian (1997), Cook and Klingner (June 1992), and ACI Committee 349-85. Even though the ϕ factor for plain concrete uses a value of 0.65, the basic factor for brittle failures (ϕ = 0.75) has been chosen based on the results of probabilistic studies (Farrow and Klingner (1995)) that indicated that for anchoring to concrete the use of ϕ = 0.65 with mean values of concrete-controlled failures produced adequate safety levels. However, the nominal resistance expressions used in the *Provisions* and in the test requirements are the 5 percent fractiles. Thus, the ϕ = 0.65 value would be overly conservative. Comparison with other design procedures and probabilistic studies by Farrow and Klingner (1995) indicated that the choice of ϕ = 0.75 was justified. For applications with supplementary reinforcement and more ductile failures (Condition A), the ϕ factors are increased. The value of ϕ = 0.85 is compatible with the level of safety for shear failures in concrete beams, and has been recommended by the *PCI Design Handbook* (1992) and ACI 349-85 .

The anchor prequalification tests for sensitivity to installation procedures determine the category appropriate for a particular anchoring device. In the prequalification tests, the effects of variability in anchor torque during installation, tolerance on drilled hole size, energy level used in setting anchors, and lateral contact with reinforcement are considered. The three categories of acceptable post-installed anchors are:

Category 1 - systems with high installation safety

Category 2 - systems with medium installation safety

Category 3 - systems with lower but still acceptable installation safety

The capacities of anchors under shear loads are not as sensitive to installation errors and tolerances. Therefore, for shear calculations of all anchors ϕ = 0.85 for Condition A and ϕ = 0.75 for Condition B.

9.2.5 Design Requirements for Tensile Loading:

9.2.5.2 Concrete Breakout Strength of Anchor in Tension:

9.2.5.2.1: 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_N / A_{No} and y_2 in Eq. 9.2.5.2.1-1 or -2.

Figure C.9.2.5.2.1-1 shows A_{No} and the development of Eq. 9.2.5.2.1-3. A_{No} is the maximum projected area for a single anchor. Figure C9.2.5.2.1-2 shows examples of the projected areas for various single anchor and multiple anchor arrangements. Because A_N is the total projected area for a group of anchors, and A_{No} is the area for a single anchor, there is no need to include *n*, the number of anchors, in Eq. 9.2.5.2.1-1 or 9.2.5.2.1-2. If anchor groups are positioned in such a way that their projected areas overlap, the value of A_N is required to be reduced accordingly.

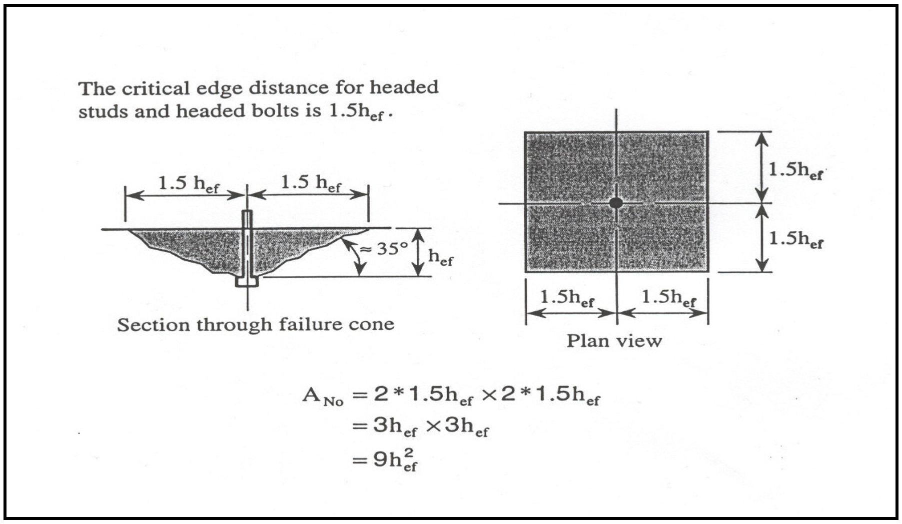

FIGURE C9.2.5.2.1-1 Calculation of A_{N0}.

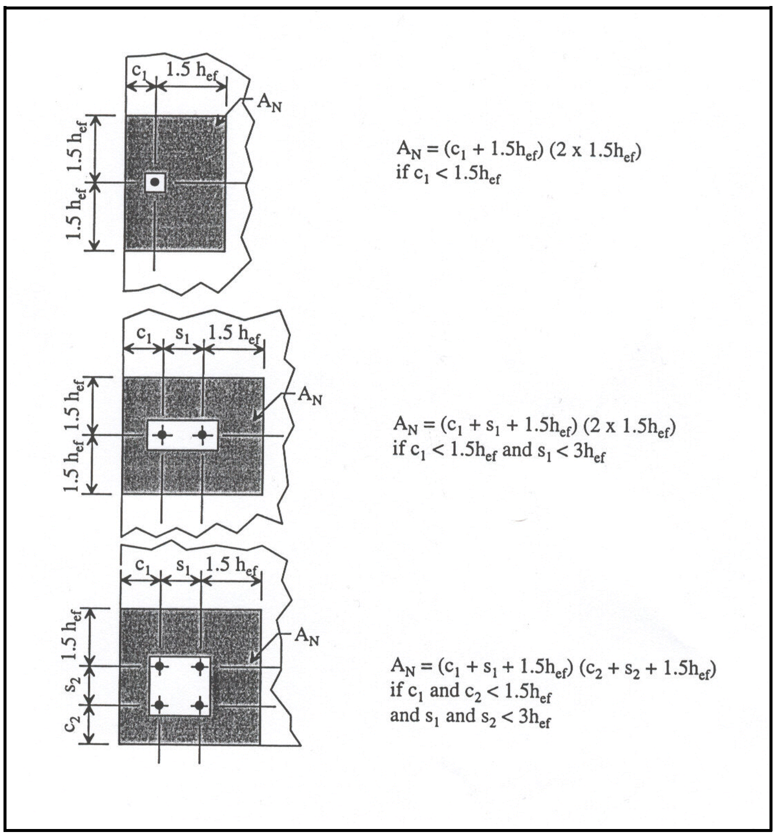

FIGURE C9.2.5.2.1-2 Projected areas for single anchors and groups of anchors.

9.2.5.2.2: The basic equation for anchor capacity was derived (Fuchs, Eligehausen, and Breen (1991), Eligehausen and Balogh (1995), Fastenings to Concrete and Masonry Structures (1994), and Eligehausen and Fuchs (1998)) assuming a concrete failure prism with an angle of about 35 degrees, and considering fracture mechanics concepts.

The values of k were determined from a large database of test results in uncracked concrete (Fuchs, Eligehausen and Breen (1995) as the 5 percent fractile. The values were adjusted to corresponding k values for cracked concrete (Eligehausen and Balogh (1995) and Zhang (1997)). For anchors with a deep embedment ($h_{ef} > 11$ in.) some 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 alternate expression (Eq. 9.2.5.2.2-2) is provided using $h_{ef}^{5/3}$ for evaluation of cast-in anchors with 11 in. $< h_{ef} <$ 25 in. The limit of 25 in. corresponds to the upper range of test data. This expression can also be appropriate for some undercut post-installed anchors. However, Sec. 9.2.4.2 should be used with test results to justify such applications.

9.2.5.2.3: For anchors influenced by three or more edges where any edge distance is less than 1.5 h_{ef}, the tensile breakout strength computed by the ordinary CCD method, which is the basis for Eq. 9.2.5.2.2-1, gives misleading results. This occurs because the ordinary definitions of A_N/A_{No} do not correctly reflect the edge effects. However, if the value of h_{ef} is limited to $c_{max}/1.5$, where c_{max} is the largest of the influencing edge distances that are less than or equal to the actual $1.5h_{ef}$, this problem is corrected. As shown by Lutz (1995), this limiting value of h_{ef} is to be used in Eq. 9.2.5.2.1-3, 9.2.5.2.2-1, 9.2.5.2.4, and 9.2.5.2.5-1 or -2. This approach is best understood when applied to an actual case. Figure C9.2.5.2.3 shows how the failure surface has the same area for any embedment beyond the proposed limit on h_{ef} (taken as h'_{ef} in the figure). In this example, the proposed limit on the value of h_{ef} to be used in the computations where $h_{ef} = c_{max}/1.5$, results in $h_{ef} = h'_{ef} = 4$ in./1.5 = 2.67 in. This would be the proper value to be used for h_{ef} in computing the resistance, for this example, even if the actual embedment depth is larger.

9.2.5.2.4: Figure C9.2.5.2.4-1 shows dimension $e'_N = e_N$ for a group of anchors that are all in tension but that have a resultant force eccentric with respect to the centroid of the anchor group. Groups of anchors can be loaded in such a way that only some of the anchors are in tension (Figure C9.2.5.2.4-2). 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. Eq. 9.2.5.2.4 is limited to cases where $e'_N < s/2$ to alert the designer that all anchors may not be in tension.

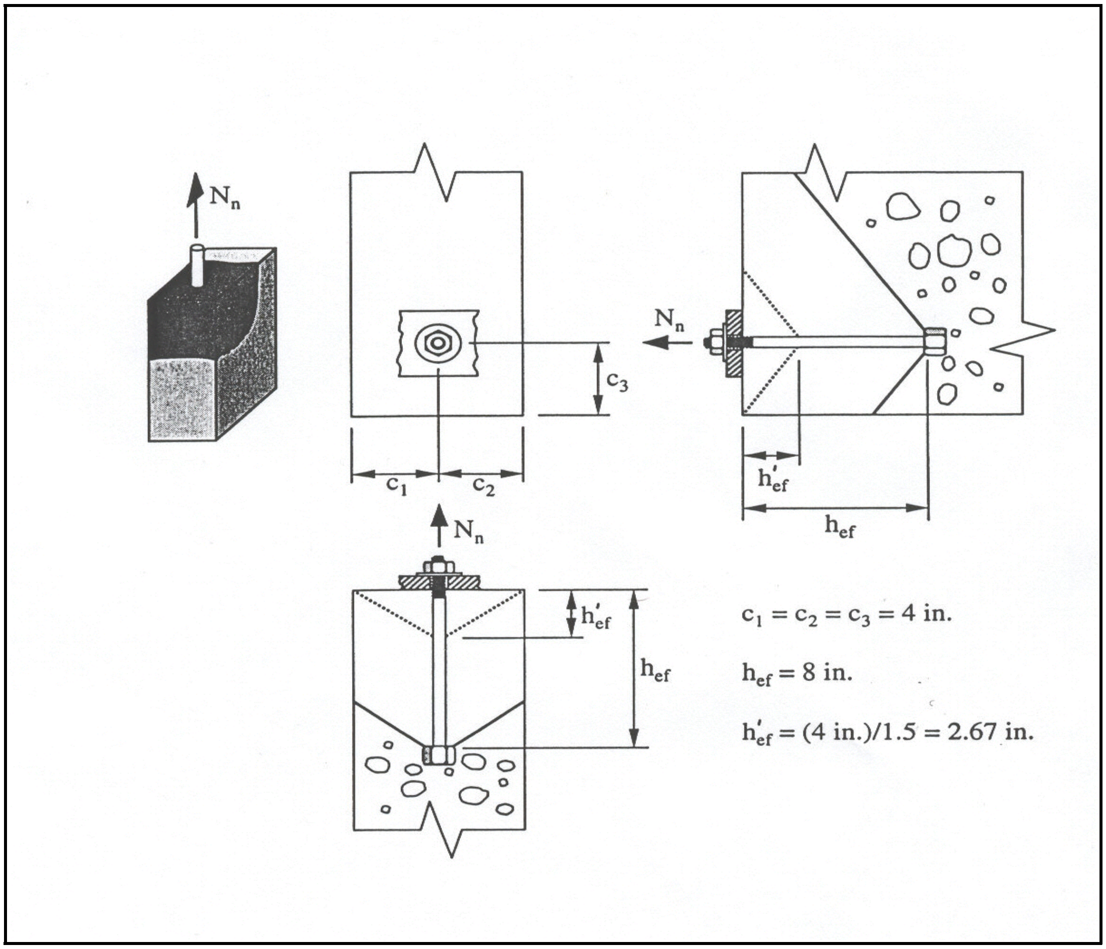

FIGURE C9.2.5.2.3 Failure surfaces in narrow members for different embedment depths.

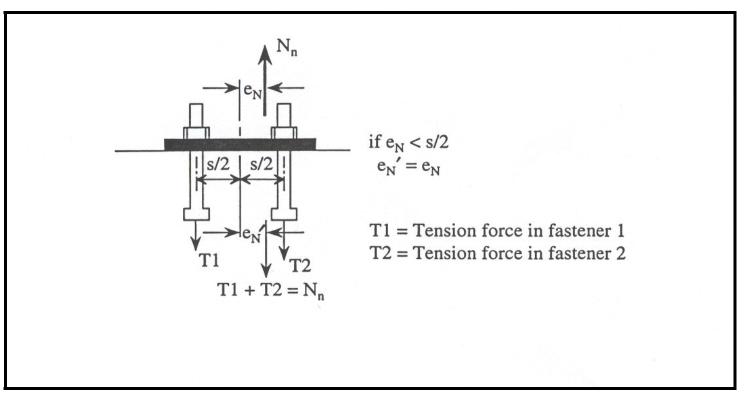

FIGURE C9.2.5.2.4-1 Definition of dimension e_N' when all anchors in a group are in tension.

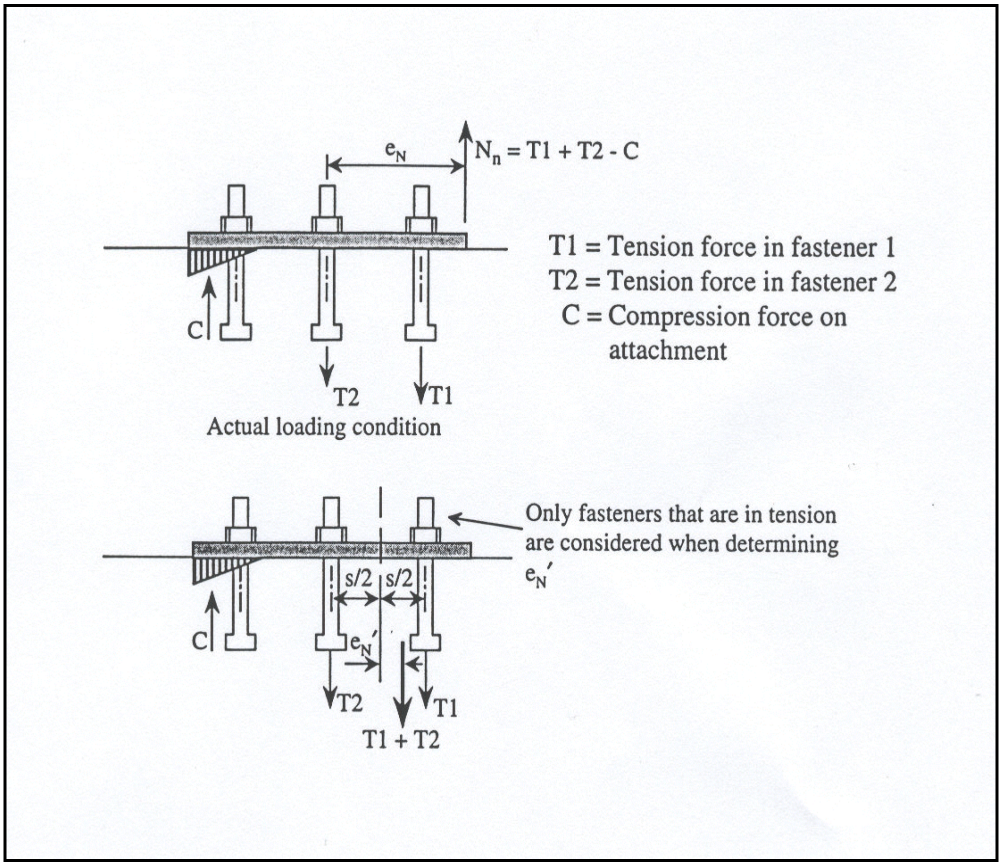

FIGURE C9.2.5.2.4-2 Determination of e_N' for anchor group with only some anchors in tension.

9.2.5.2.5: If anchors are located close to an edge so that there is not enough space for a complete breakout prism to develop, the load bearing capacity of the anchor is further reduced beyond that reflected in A_N/A_{No}. If the smallest side cover distance is greater than 1.5 h_{ef}, a complete prism can form and there is no reduction (Y_2 = 1). If the side cover is less than 1.5 h_{ef}, the factor, Y_2, is required to adjust for the edge effect (Lotze and Klingner (1997).

9.2.5.2.6: Post-installed and cast-in anchors that have not met the requirements for use in cracked concrete according to the anchor prequalification tests should be used in uncracked regions only. The analysis for the determination of crack formation should include the effects of restrained shrinkage.

9.2.5.2.7: The anchor prequalification tests require that anchors in cracked concrete zones perform well in a crack that is 0.012 in. wide. If wider cracks are expected, confining reinforcement to control the crack width to about 0.012 in. should be provided.

9.2.5.3 Pullout Strength of Anchors in Tension:

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

9.2.5.3.4: Eq. 9.2.5.3.4 corresponds to the load at which the concrete under the anchor head begins to crush. (*Design of Fastenings in Concrete* (1997) and ACI 349-85). 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. The designer should be aware that local crushing under the head bearing region will greatly reduce the stiffness of the connection, and generally will be the beginning of a pullout failure.

9.2.5.3.5: Eq. 9.2.5.3.5 for J-bolts and L-bolts was developed by Lutz based on the results of work by Kuhn and Shaikh (1996). Reliance is placed on the bearing *component* only, neglecting any frictional *component* since local crushing under the head will greatly reduce the stiffness of the connection, and generally will be the beginning of pullout failure.

9.2.5.4 Concrete Side-Face Blowout Strength of Anchor in Tension: The design requirements for side-face blowout are based on the recommendations of Furche and Eligehausen (1991). These requirements are applicable to headed anchors that usually are cast-in anchors. Splitting during installation rather than sideface blowout generally governs post-installed anchors, and is evaluated by the anchor prequalification tests.

9.2.6 Design Requirements for Shear Loading:

9.2.6.2 Concrete Breakout Strength of Anchors in Shear:

9.2.6.2.1: The shear strength equations were developed from the CCD method. They assume a breakout cone angle of approximately 35 degrees Figure C9.2.4.2.2-2, 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 A_V/A_{V_o} and ι_5 in Eq. 9.2.6.2.1-1 or -2. For anchors far from the edge, Sec. 9.2.6.2 usually will not govern. For these cases, Sec. 9.2.6.1 and Sec. 9.2.6.3 often govern.

Figure C9.2.6.2.1-1 shows A_{V_o} and the development of Eq. 9.2.6.2.1-3. A_{V_o} 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 C9.2.6.2.1-2 shows examples of the projected areas for various single anchor and multiple anchor arrangements. A_V approximates the full surface area of the breakout cone for the particular arrangement of anchors. Since A_V is the total projected area for a group of anchors, and A_{V_o} is the area for a single anchor, there is no need to include the number of anchors in the equation.

The assumption shown in Figure C9.2.6.2.1-2 with the case for two anchors perpendicular to the edge is a conservative interpretation of the distribution of the shear force on an elastic basis. 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 cases where nominal strength is not controlled by ductile steel elements, ACI Committee 318 has specified in Sec. 9.2.3.1 that load effects be determined by elastic analysis. It has been suggested in the *PCI Design Handbook* approach (1992) that the increased capacity 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 (*Fastenings to Concrete and Masonry Structures* (1994)), Sec. 9.2.6.2 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 $1.5c_1$, then after formation of the near-edge failure surface, the higher capacity of the farther anchor would resist most of the load. As shown in the bottom example in Figure C9.2.6.2.1-2, it would be appropriate to consider the full shear capacity to be provided by this anchor with its much larger resisting failure surface. No contribution of the anchor near the edge is then considered. It would be advisable to check the near-edge anchor condition to preclude undesirable cracking at service load conditions. Further discussion of design for multiple anchors is given *in Design of Fastenings in Concrete* (1997).

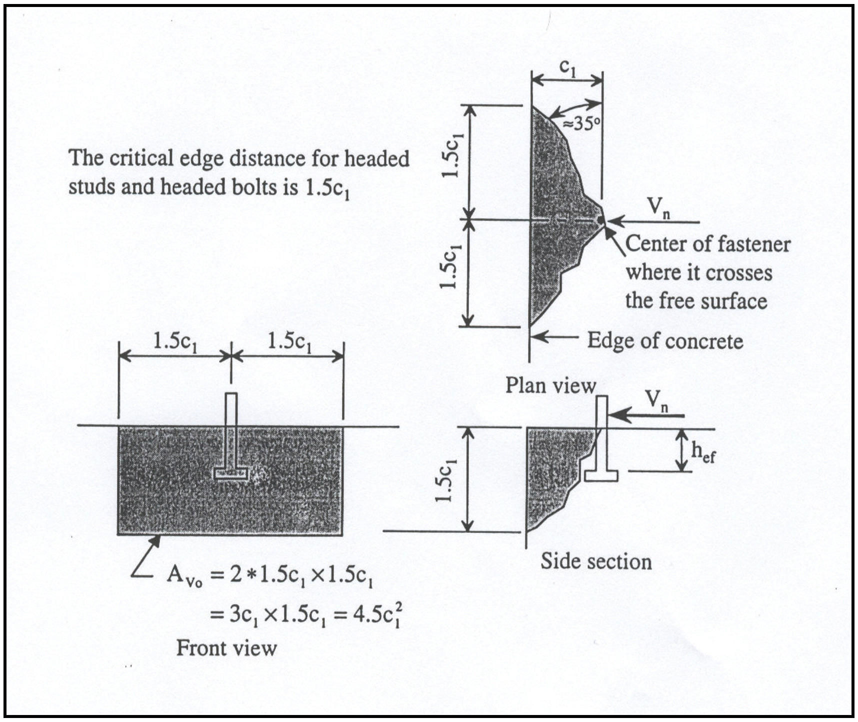

FIGURE C9.2.6.2.1-1 Calculation of A_{V0}.

For the case of anchors near a corner subjected to a shear force with *components* normal to each edge, a satisfactory solution is to independently check 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 *Fastenings to Concrete and Masonry Structures* (1994).

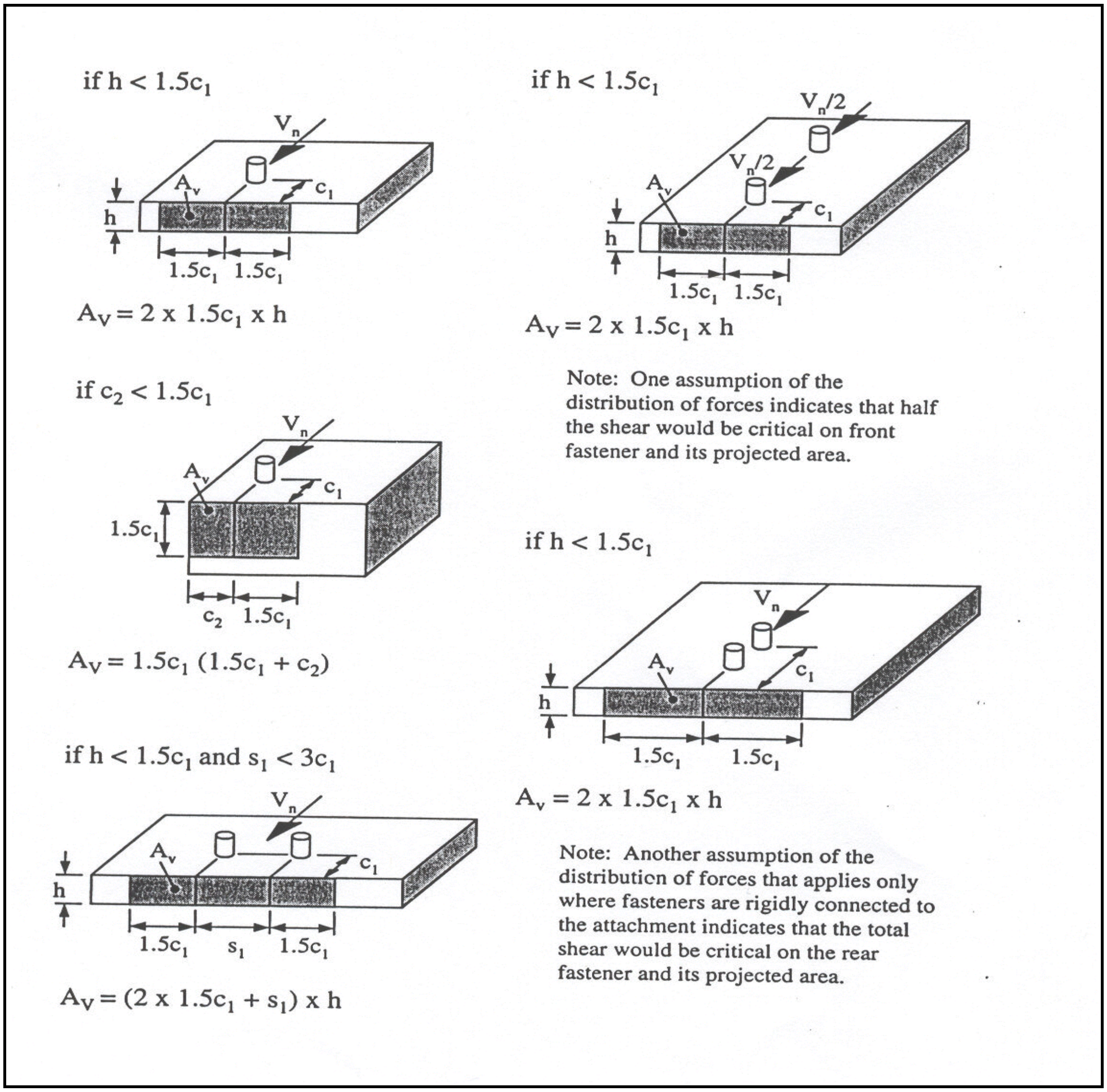

FIGURE C9.2.6.2.1-2 Projected areas for single anchor and groups of anchors.

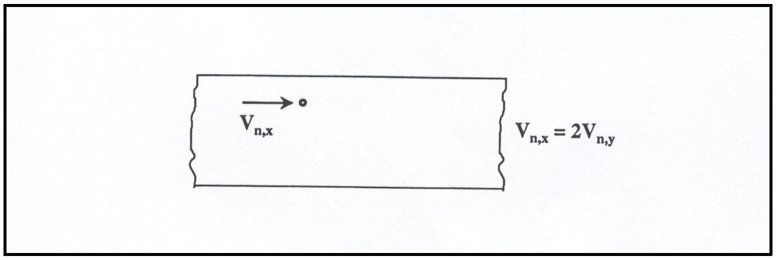

FIGURE C9.2.6.2.1-3 Shear force parallel to an edge.

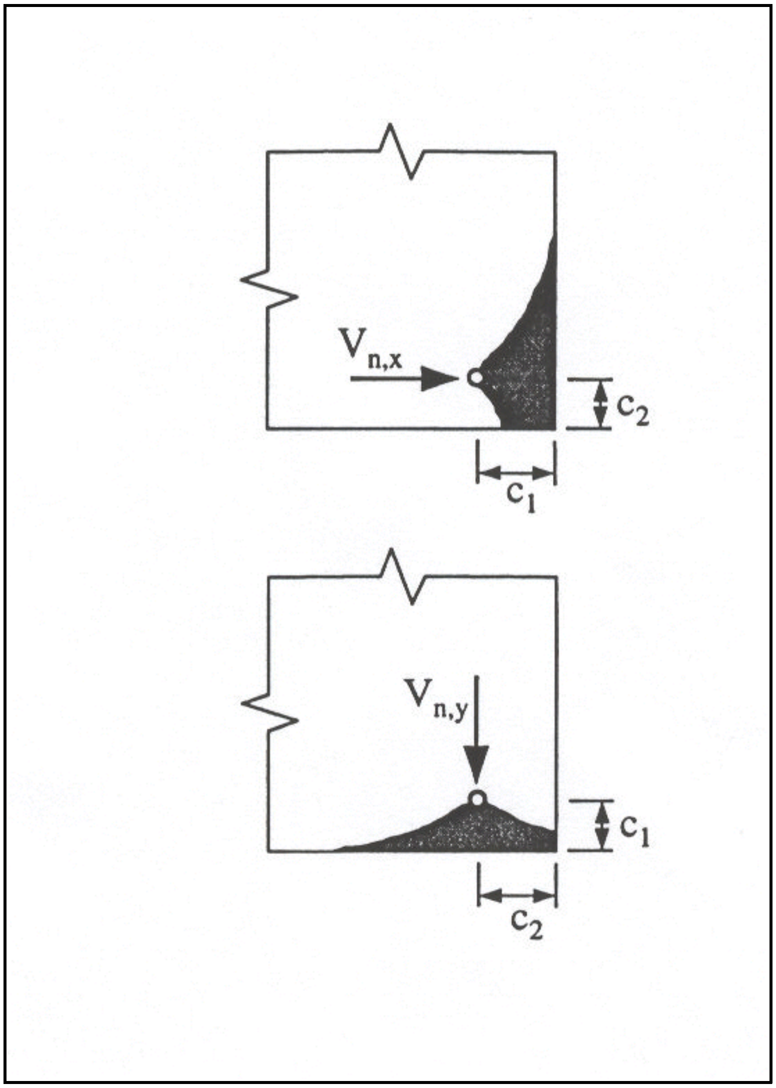

FIGURE C9.2.6.2.1-4 Anchors near a corner.

The detailed provisions of Sec. 9.2.6.2.1 (a) apply to the case of shear force directed towards an edge. When the shear force is directed away from the edge, the strength will usually be governed by Sec. 9.2.6.1 or 9.2.6.3.

The case of shear force parallel to an edge (Sec. 9.2.6.2.1b) is shown in Figure C9.2.6.2.1-3. A special case can arise with shear force parallel to the edge near a corner. Take the example of a single anchor near a corner (Figure C9.2.6.2.1-4). If the edge distance to the side c_2 is 40 percent or more of the distance c_1 in the direction of the load, the shear strength parallel to that edge can be computed directly from Eq. 9.2.6.2.1-1 or -2 using c_1 in the direction of the load.

9.2.6.2.2: Like the concrete breakout tensile capacity, the concrete breakout shear capacity does not increase with the failure surface, which is proportional to c_l^2. Instead the capacity increases proportionally to $c_l^{1.5}$, due to the size effect. The capacity is also influenced by the anchor stiffness and the anchor diameter D. (see diameter. (See Fuchs, Eligehausen, and Breen (1995), Eligehausen and Balogh (1995), *Fastenings to Concrete and Masonry Structures* (1994), and Eligehausen and Fuchs (1988).

The constant 7 in the shear strength equation was determined from test data reported in the article by Fuchs, Eligehausen, and Breen (1995) as the 5 percent fractile adjusted for cracking.

9.2.6.2.3: For the special case of cast-in headed bolts rigidly welded to an attachment, test data (Wong (1988) [1988] and Shaikh and Yi (1985) [1985]) show that somewhat higher shear capacity exists, possibly due to the stiff welding connection clamping the bolt more effectively than an attachment with a 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 *Design of Fastenings in Concrete* (1997*), Fastenings to Concrete and Masonry Structures* (1994) and Klingner, Mendonca, and Malik (1982).

9.2.6.2.4: For anchors influenced by three or more edges where any edge distance is less than $1.5c_l$, the shear breakout strength computed by the basic CCD Method, which is the basis for Eq. 9.2.6.2.2 or 9.2.6.2.3, gives safe but misleading results. These special cases were studied for the κ method (Eligehausen and Fuchs (1988)) and the problem was pointed out by Lutz (1995).

Similar to the approach used for tensile breakouts in Sec. 9.2.5.2.3, a correct evaluation of the

capacity is determined if the value of c_1 to be used in Eq. 9.2.6.2.1-3, 9.2.6.2.2 or 9.2.6.2.3, 9.2.6.2.5 and 9.2.6.2.6-1 or -2 is limited to h/1.5.

9.2.6.2.5: This section provides a modification factor for an eccentric shear force towards an edge on a group of anchors. If the shear load 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 can or cannot also cause tension in the anchors, depending on the normal force. Figure C9.2.6.2.5 defines the term e'_v for calculating the Ψ_5 modification factor that accounts for the fact that more shear is applied on one anchor than the other, tending to split the concrete near an edge. If $e'_v > s/2$, the CCD procedure is not applicable.

9.2.6.2.6: Figure C9.2.6.2.6 shows the dimension c_2 for the Ψ_6 calculation.

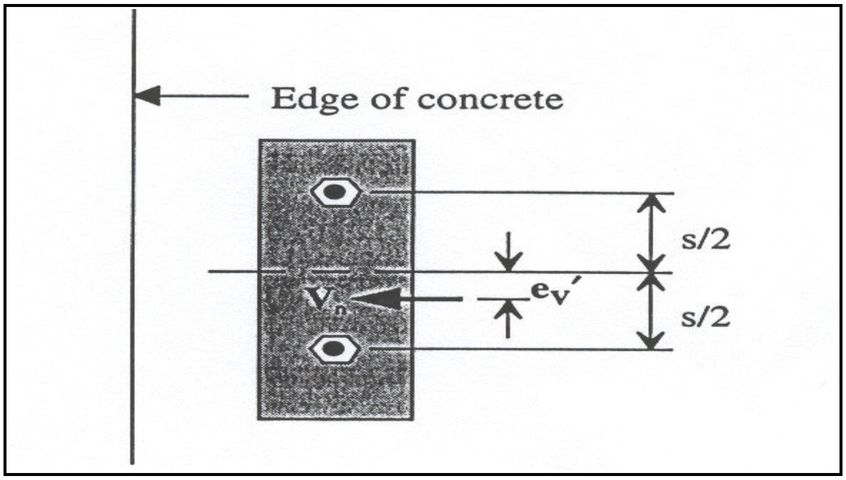

FIGURE C9.2.6.2.5 Definition of dimension e_V'.

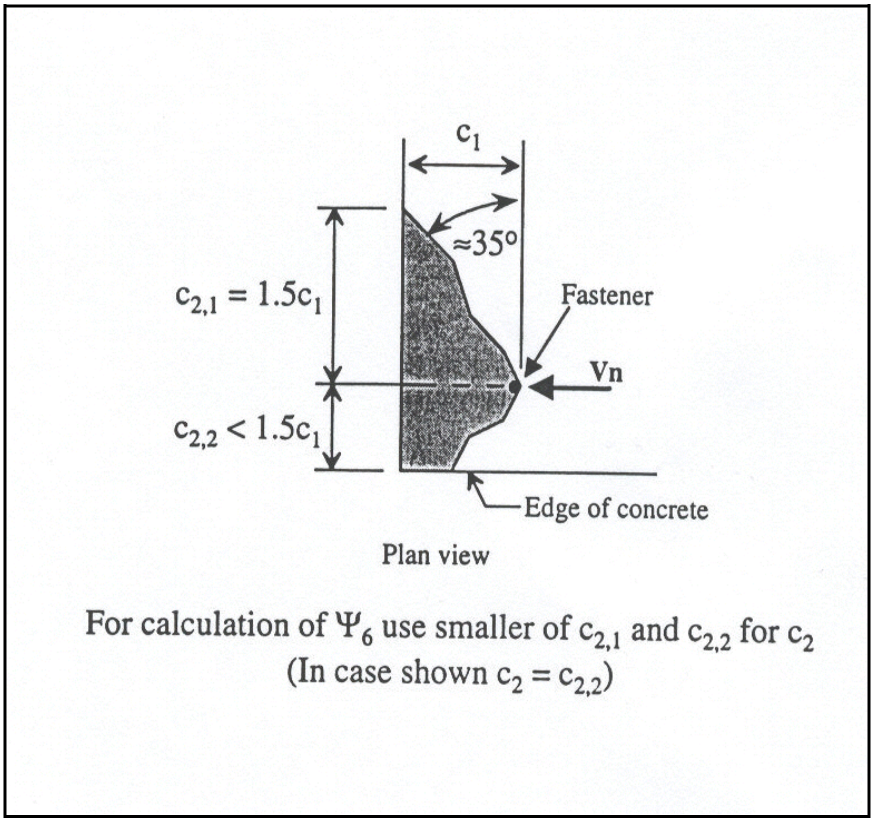

FIGURE C9.2.6.2.6 Dimension c_2 for edge proximity modification factor.

9.2.6.2.7: Torque-controlled and displacement-controlled expansion anchors are permitted in cracked concrete under pure shear loadings.

9.2.6.3 Concrete Pryout Strength:

9.2.6.3.1: The article by Fuchs, Eligehausen, and Breen (1995) indicates that the pryout shear resistance can be approximated as 1 to 2 times the anchor tensile resistance with the lower value appropriate for h_{ef} less than 2.5 in.

9.2.7 Interaction of Tensile and Shear Forces: The shear-tension interaction expression has traditionally been expressed as:

$$\left(\frac{N}{N_n}\right)^{a} + \left(\frac{V}{V_n}\right)^{a} \le 1.0 \qquad \text{(C9.2.7)}$$

where α varies from 1 to 2. The current tri-linear recommendation is a simplification of the expression where α = 5/3 (Figure C9.2.7). The limits were chosen to eliminate the requirement for computation of interaction effects where very small values of the second force are present. However, any other interaction expression that is verified by test data can be used under Sec. 9.2.4.3.

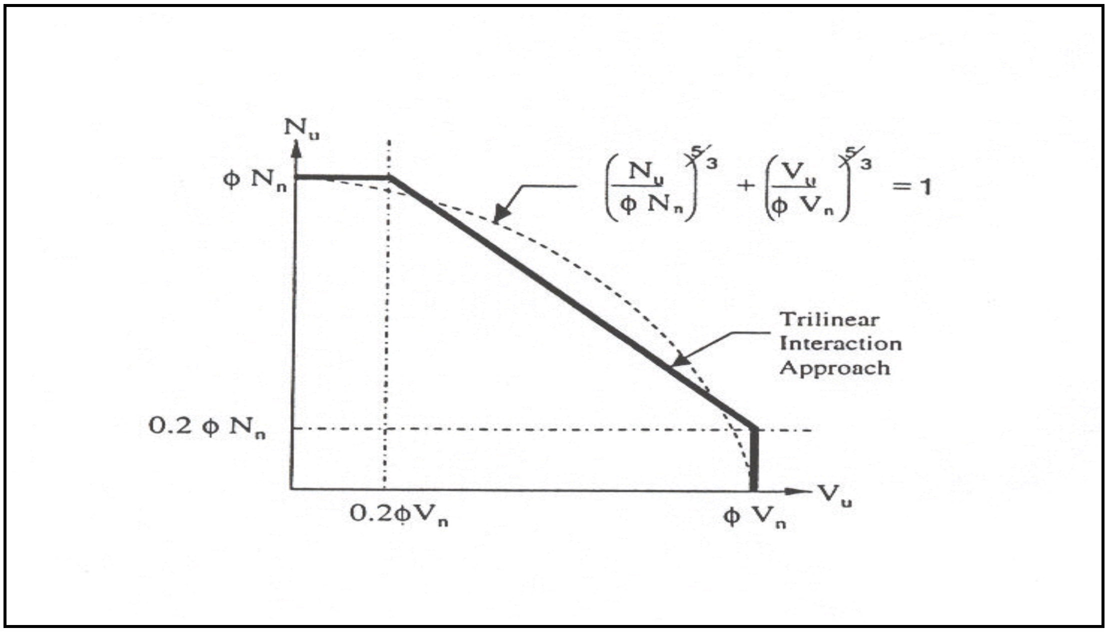

FIGURE C9.2.7 Shear and tensile load interaction equation.

9.2.8 Required Edge Distances, Spacings, and Thicknesses to Preclude Splitting Failure: The minimum spacings, 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 spacings, edge distances, and thicknesses of post-installed anchors should be the product-specific prequalification tests of Sec. 9.2.1.4. However, in some cases specific products are not known in the design stage. Approximate values are provided for use in design.

9.2.8.2: Since the edge cover over a deep embedment close to the edge can have a significant effect on the side-face blowout strength of Sec. 9.2.5.4, in addition to the normal concrete cover requirements, the designer may wish to use larger cover to increase the side-face blowout strength.

9.2.8.3: Drilling holes for post-installed anchors can cause microcracking. The requirement for a minimum edge distance 2 times the maximum aggregate size is to minimize the effects of such microcracking.

9.2.8.5: This minimum thickness requirement is not applicable to through-bolts because they are outside the scope of the Provisions. 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.

9.2.9 Installation of Anchors: Many anchor performance characteristics depend on proper installation of the anchor. Anchor capacity and deformations can be assessed by the anchor prequalification tests. These tests are carried out assuming that the manufacturer's installation directions will be followed. Certain types of anchors can be sensitive to variation in hole diameter, cleaning conditions related to embedment depth, orientation of the axis, magnitude of the installation torque, proximity of reinforcement, and other variables. Some of this 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 reliability tests. Gross deviations from the prequalification 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.

9.3 CLASSIFICATION OF SHEAR WALLS: In the 2000 *Provisions*, shearwalls have been classified by the amount and type of detailing required. This classification was developed to facilitate assigning shearwalls to seismic design categories.

9.4 SEISMIC PERFORMANCE CATEGORY A: Construction qualifying under Category A may be built with no special detailing requirements for earthquake resistance. Special details for ductility and *toughness* are not required in Category A.

9.5 SEISMIC PERFORMANCE CATEGORY B: Special details for ductility and *toughness* are not required in Category B.

9.5.1 Ordinary Moment Frames: Since ordinary *frames* are permitted only in Categories A and B, they are not required to meet any particular seismic requirements. Attention should be paid to the often overlooked requirement for *joint* reinforcement in Sec.11.11.2 of ACI 318.

9.6 SEISMIC PERFORMANCE CATEGORY C: A *frame* used as part of the lateral force resisting system in Category C is required to have certain details that are intended to help sustain integrity of the *frame* when subjected to *deformation* reversals into the nonlinear range of response. Such *frames* must have attributes of *intermediate moment frames*. Structural *(shear) walls* of *buildings* in Category C are to be built in accordance with the requirements of ACI 318.

9.6.2 Intermediate Moment Frames and 9.6.3 Special Moment Frames: The concept of *moment frames* for various levels of hazard zones and of performance is changed somewhat from the provisions of ACI 318. Two sets of *moment frame* detailing requirements are defined in ACI 318, one for "regions of high seismic risk" and the other for "regions of moderate seismic risk." For the purposes of this document, the "regions" are made equivalent to Seismic Performance Categories in which "high risk" means Categories D and E and "moderate risk" means Category C. This document labels these two *frames* the "*special moment frame*" and the "*intermediate moment frame*," respectively.

The level of inelastic energy absorption of the two *frames* is not the same. The *Provisions* introduce the concept that the *R* factors for these two *frames* should not be the same. The preliminary version of the Provisions (ATC 3-06) assigned the *R* for ordinary *frames* to what is now called the intermediate *frame*. In spite of the fact that the *R* factor for the intermediate *frame* is less than the *R* factor for the special *frame*, use of the intermediate *frame* is not permitted in the higher Performance Categories (D and E). On the other hand, this arrangement of the provisions encourages consideration of the more stringent detailing practices for the special *frame* in Category C because the reward for use of the higher *R* factor can be weighed against the higher cost of the detailing requirements. The *Provisions* also introduce the concept that an intermediate *frame* may be a part of a Dual System in Category C.

The differences in the performance basis of the requirements for the two types of *frames* might be briefly summarized as follows (see the commentary of ACI 318 for a fuller discussion of the requirement for the special *frame*):

1. The shear *strength* of beams and columns shall not be less than that required when the member has yielded at each end in flexure. For the special *frame*, strain hardening and other factors are considered by raising the effective tensile *strength* of the bars to 125 percent of specified yield. For the intermediate *frame*, an escape clause is provided in that the calculated shear using double the prescribed seismic force may be substituted. Both types require the same minimum amount and maximum spacing of transverse reinforcement throughout the member.

2. The shear *strength* of *joints* is limited and special provisions for anchoring bars in *joints* exist for *special moment frames* but not intermediate *frames*. Both *frames* require transverse reinforcement in *joints* although less is required for the intermediate *frame*.

3. Closely spaced transverse reinforcement is required in regions of potential hinging (typically the ends of beams and columns) to control lateral buckling of longitudinal bars after the cover has spalled. The spacing limit is slightly more stringent for columns in the special *frame*.

4. The amount of transverse reinforcement in regions of hinging for special *frames* is empirically tied to the concept of providing enough confinement of the concrete core to preserve a ductile response. These amounts are not required in the intermediate *frame* and, in fact, stirrups in lieu of hoops may be used in beams.

5. The special *frame* must follow the strong column/weak beam rule. Although this is not required for the intermediate *frame*, it is highly recommended for multistory construction.

6. The maximum and minimum amounts of reinforcement are limited to prevent rebar congestion and assure a nonbrittle flexural response. Although the precise limits are different for the two types of *frames*, a great portion of practical, buildable designs will satisfy either.

7. Minimum amounts of continuous reinforcement to account for moment reversals are required by placing lower limits on the flexural *strength* at any cross section. Requirements for the two types of *frames* are similar.

8. Locations for splices of reinforcement are more tightly controlled for the special *frame*.

9. In *addition*, the special *frame* must satisfy numerous other requirements beyond the intermediate *frame* to assure that member proportions are within the scope of the present research experience on seismic resistance and that the analysis, the design procedures, the qualities of the materials, and the inspection procedures are at the highest level of the state of the art.

9.7 SEISMIC PERFORMANCE CATEGORIES D, E, or F: The requirements conform to current practice in the areas of highest seismic hazard.

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Appendix to Chapter 9

REINFORCED CONCRETE DIAPHRAGMS CONSTRUCTED USING UNTOPPED PRECAST CONCRETE ELEMENTS

C9A.1 BACKGROUND: Although not directly addressed in the code, untopped precast *components* have been used as diaphragms in high seismic regions. Untopped hollow core with grouted joints and end chords have performed successfully both in earthquakes and in laboratory tests, (Elliot et al., 1992) (Menegotto,1994), (Priestley et al. 1999). Experience has also demonstrated the unsuccessful use of cast-in-place concrete topping as diaphragms (Iverson and Hawkins, 1994). Where problems have occurred, they have not been inherently with the precast construction, but the result of a failure to address fundamental requirements of structural mechanics.

This section provides conditions that are intended to ensure that diaphragms composed of precast *components* are designed with attention to the principles required for satisfactory behavior. Each condition addresses requirements that should be considered for all diaphragms, but which are particularly important in jointed construction. Specific attention should be paid to providing a complete load path that considers force transfer across all joints and connections.

C9A.3 Untopped Precast Diaphragms:

C9A.3.1: Out-of-plane offsets in the vertical elements of the seismic-force-resisting system place particularly high demands on the diaphragm in providing a continuous load path. Untopped precast diaphragms are not suitable for this condition.

C9A.3.2: Following the principle that the diaphragm is not generally an appropriate location for inelastic behavior and, in particular, for untopped precast diaphragms, specific direction is provided that elastic models should be used for diaphragm analysis. Connections are subject to a combination of load effects (Fleischman et al. 1998). The distribution of loads may change after yielding, and therefore the design of the diaphragm should avoid yielding.

C9A.3.3: Since the diaphragm is not generally an appropriate location for inelastic behavior, it should be designed to a level of strength that is intended to ensure the ductility and yield strength of the seismic-force-resisting system can be mobilized before the diaphragm yields. While research (Fleischman et al. 1998) suggests that the diaphragm demand will not exceed twice the equivalent lateral forces used for the vertical system design, Table 5.2.2. prescribes an overstrength factor, Ω_o, and Sec. 5.2.4 prescribes a redundancy factor, ρ, for the systems that should be used. If an analysis of the probable strength of the seismic-force-resisting system is made to determine a lower demand on the diaphragm, the design force used should still be sufficient to attempt to ensure that the diaphragm remains elastic. For that reason a 1.25 factor is specified.

C9A.3.4: It must be recognized that the demand on diaphragms in buildings with these plan irregularities requires special attention. In accordance with Sec 5.2.6.4.3 the design force for the diaphragm should be increased by at least 25 percent when such irregularities are present in structures assigned to SDC D, E and F.

C9A.3.5: Although the design procedures prescribed in these sections are intended to ensure elastic behavior at the level of the code design forces, it is recognized that catastrophic events may exceed code requirements. Under such circumstances, it is important that the connections possess ductility under reversed cyclic loading. The intent, in these sections, is for the connection capacity to be limited by steel yielding of the connector and not by brittle concrete failure or weld fracture.

C9A.3.6: Substantiating experimental evidence to demonstrate through testing and evaluation that mechanical connections satisfy the principles specified in ITG/T1.1 and ATC-24, and can develop the required capacity and ductility, should meet the following criteria:

Test Procedures:

1. Prior to testing, a design procedure should have been developed for prototype connections having the generic form that is to be tested for acceptance.
2. That design procedure should be used to proportion the test specimens.
3. Specimens should not be less than two-thirds scale.
4. Test specimens should be subject to a sequence of reversing cycles having increasing limiting displacements.
5. Three fully reversed cycles should be applied at each limiting displacement.
6. The maximum load for the first sequence of three cycles should be 75 percent of the calculated nominal strength of the connection, E_n.
7. The stiffness of the connection should be defined as 75 percent of the calculated nominal strength of the connection divided by the corresponding measured displacement, δ_m.
8. Subsequent to the first sequence of three cycles, limiting displacements should be incremented by values not less than one, and not more than one and one quarter times δ_m.

Acceptance Criteria:

1. The connection should develop a strength, E_{max}, greater than its calculated nominal strength, E_n..
2. The strength, E_{max}, should be developed at a displacement not greater than $3\delta_m$.
3. For cycling between limiting displacements not less than $3\delta_m$, the peak force for the third loading cycle for a given loading direction should not be less than $0.8\ E_{max}$ for the same loading direction.

Results of reversed cyclic loading tests on typical connections are reported in Spencer (1986) and Pincheira et al. (1998).

C9A.3.9: Successful designs may include a combination of untopped precast *components* with areas of concrete topping in locations of high force demand or concentration. Such topping can allow for continuity of reinforcement across joints. For such designs, the requirements for topping slab diaphragms apply to the topped portions.

C9A.3.10: .An important element in the *Provisions* is attention to deformation compatibility requirements. Reduction in effective shear and flexural stiffness for the diaphragm is appropriate in evaluating the overall effects of drift on elements that are not part of the seismic-force-resisting system. This approach should encourage the use of more vertical elements to achieve shorter spans in the diaphragm and result in improved system redundancy and diaphragm continuity. Redundancy will also improve the overall behavior should any part of the diaphragm yield in a catastrophic event.

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