Chapter 9

CONCRETE STRUCTURE DESIGN REQUIREMENTS

9.1 REFERENCE DOCUMENTS: The quality and testing of concrete and steel materials and the design and construction of concrete *components* that resist *seismic forces* shall conform to the requirements of the reference listed in this section except as modified by the requirements of this chapter.

ACI 318	American Concrete Institute (ACI), *Building Code Requirements for Structural Concrete*, excluding Appendix A, ACI 318, 1999.
ACI ITG/T1.1	American Concrete Institute (ACI), *Acceptance Criteria for Moment Frames Based on Structural Testing (An ACI Provisional Standard),* 1999.
ASME B1.1	American Society of Mechanical Engineers (ASME), *Unified Inch Screw Threads UN and UNR Thread Form,* ASME B1.13, 1989.
ASME B18.2.1	American Society of Mechanical Engineers (ASME), *Square and Hex Bolts and Screws, Inch Series,* ASME B18.2.1, 1996
ASME B18.2.6.9	American Society of Mechanical Engineers (ASME),
ATC-24	Applied Technology Council (ATC), *Guidelines for Seismic Testing of Components of Steel Structures,* ATC-24, 1992

9.1.1 Modifications to ACI 318:

9.1.1.1: Insert the following notations in Sec. 21.0:

A_b	=	the area of the shank of the bolt or stud (in.^{2} or mm^2)
b_d	=	diaphragm width (ft)
c_u'	=	neutral axis depth at P_u' and M_n'.
ℓ_p	=	height of the plastic hinge above critical section; shall be established on the basis of substantiated test data or may be alternatively taken at $0.5\ell_w$.
P_u'	=	$1.2D + 0.5L + E$.
h	=	overall dimension of member in the direction of action considered.
h_s	=	story height (ft)
L_{eff}	=	length of diaphragm between inflection points (ft)
$S_{c\ Connection}$	=	*nominal strength* of connection cross section in flexural, shear, or axial load per Sec. 21.2.7.3.

$S_{e\ Connection}$ = moment, shear, or axial force at strong connection cross section corresponding to probable *strength* at the nonlinear action location, taking *gravity load* effects into consideration per Sec. 21.2.8.3.

$S_{n\ Connection}$ = *nominal strength* of connection cross section in flexural, shear, or axial load per Sec. 21.2.7.3.

S_{pr} = probable *strength* of connection cross section in flexural, shear, or axial load. See Sec. 21.11.5.

S_y = yield strength of connection cross section in flexural, shear, or axial load action as determined from physical experiments on the connection or the use of analytical models for the response of the connection that are based on the results of physical experiments on connections with characteristics similar to those being modeled. See Sec. 21.11.5.

Δ_E = elastic *design displacement* at the top of the wall using gross section properties and code-specified *seismic forces*.

Δ_i = inelastic deflection at top of wall = Δ_t - Δ_y.

Δ_m = $C_d\Delta_s$.

Δ_s = *design* level response *displacement*, which is the total drift or total *story* drift that occurs when the *structure* is subjected to the design *seismic forces*.

Δ_t = total deflection at the top of the wall equal to C_d times the elastic *design displacement* using cracked section properties or may be taken as $(I_g/I_{eff})C_d\Delta_E$. I_g is the gross moment of inertia of the wall and I_{eff} is the effective moment of inertia in the wall. I_{eff} may be taken as $0.5I_g$.

Δ_y = *displacement* at the top of the wall corresponding to yielding of the tension reinforcement at critical section or may be taken as $(M_n'/M_E)\Delta_E$ where M_E equals moment at critical section when top of wall is displaced Δ_E. M_n' is nominal flexural *strength* of critical section at P_u'.

ϕ_y = yield curvature which may be estimated as $0.003/\ell_w$.

ψ = dynamic amplification factor from Sec. 21.2.8.3 and 21.2.8.4.

9.1.1.2: Insert the following definitions in Sec. 21.1:

Anchorage – The means by which, for precast construction, the force in the connection is transferred into the precast or cast-in-place member.

Connection – An element that joins two precast members or a precast member and a cast-in-place member.

Connection Region – The portion of the precast or cast-in-place member through which the concentrated forces from the connection and anchorage are transferred to the concrete. Its extent from the connection is the distance for the forces to be distributed over the cross section and shall be permitted to not exceed the largest dimension of that cross section.

Design Displacement – Design story drift as specified in Sec. 5.2.2.4.3 of the 2000 *NEHRP Recommended Provisions*.

Design Load Combinations – Combinations of factored loads and forces specified in Sec. 5.2.7 of the the 2000 *NEHRP Recommended Provisions*.

Dry Connection – Connection used between precast members which does not qualify as a wet connection.

Joint – The geometric volume common to intersecting members.

Moment Frame

> **Special moment frame –** A cast-in-place frame complying with the requirements of Sec. 21.2 through 21.5 in addition to the requirements for ordinary moment frames or a precast concrete frame complying with the requirements for a cast-in-place frame and Sec. 21.11.
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> **Nonlinear Action Location –** Center of the region of yielding in flexure, shear, or axial action.

Nonlinear Action Region – The member length over which nonlinear action takes place. It shall be taken as extending a distance of no less than $h/2$ on either side of the nonlinear action location.

Strong Connection – A connection that remains elastic while the designated nonlinear action regions undergo inelastic response under the design basis ground motion.

Structural Walls

> **Ordinary precast concrete structural wall –** A wall incorporating precast concrete elements and complying with the requirements of Chapters 1 through 18 with the requirements of Chapter 16 superseding those of Chapter 14. Where connections between wall panels are required or anchorage of wall panels to foundations is required for resistance to overturning, Type Y or Type Z connections shall be provided as required by Sec. 21.11.6.
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> **Special precast concrete structural wall –** A wall complying with the requirements of Sec. 21.11 in addition to the requirements for ordinary reinforced concrete structural walls.

Wall Pier – A wall segment with a horizontal length-to-thickness ratio of at least 2.5, but not exceeding 6, whose clear height is at least two times its horizontal length.

Wet Connection – A connection that uses any of the splicing methods permitted by Sec. 21.3.2.3 or 21.3.2.4 to connect precast members and uses cast-in-place concrete or grout to fill the splicing closure.

Wet Connection – A connection in precast construction that uses any of the splicing methods permitted by Sec. 21.2.6, 21.2.7, or 21.3.2.3 to connect precast members and uses cast-in-place concrete or grout to fill the splicing closure.

9.1.1.3: Revise Sec. 21.2.1.2, 21.2.1.3, and 21.2.1.4 as follows:

> "**21.2.1.2** For *structures* assigned to *Seismic Design Categories* A and B, provisions of Chapters 1 through 18 and 22 shall apply except as modified by the requirements of

Chapter 9 of the 2000 *NEHRP Recommended Provisions*. Where the seismic design loads are computed using provisions for intermediate or special concrete systems, the requirements of Chapter 21 for intermediate or special systems, as applicable, shall be satisfied.

"**21.2.1.3** For *structures* assigned to *Seismic Design Category* C, intermediate or special moment frames or ordinary or special reinforced concrete structural walls shall be used to resist seismic forces induced by earthquake motions. Where the design seismic loads are computed using the provisions for special concrete systems, the requirements of Chapter 21 for special systems, as applicable, shall be satisfied.

"**21.2.1.4** For *structures* assigned to *Seismic Design Categories* D, E and F, special moment frames, special reinforced concrete structural walls, diaphragms and trusses, and foundations complying with Sec. 21.2 through 21.8 and 21.11 shall be used to resist forces induced by earthquake motions. Frame members not proportioned to resist earthquake forces shall comply with Sec. 21.9."

9.1.1.4: Insert the following new Sec. 21.2.1.6 and 21.2.1.7:

21.2.1.6 Precast concrete *seismic-force-resisting systems* used in regions of high seismic risk or for *structures* assigned to high seismic performance or design categories shall satisfy the requirements of Sec. 21.11 in addition to the requirements of Sec. 21.2 through 21.9.

"**21.2.1.7** In *structures* having precast *gravity load*-carrying systems, the *seismic-force-resisting system* shall be one of the systems listed in Table 5.2.2 of the 2000 *NEHRP Recommended Provisions* and shall be well distributed using one of the following methods:

"1. The diaphragm or diaphragm segment between *seismic-force-resisting systems* shall be designed to resist a force not less than Ω_0 times the force determined in accordance with Sec. 5.2.5.4 of the 2000 *NEHRP Recommended Provisions*. The chord force determined in accordance with Sec. 21.7.8.1 of ACI 318 shall be increased by a factor equal to:

$$b_d \frac{\left[1 + 0.4\left(\frac{L_{eff}}{b_d}\right)^2\right]}{12h_s}$$

but not less than unity where:

L_{eff} = length of diaphragm between inflection points, ft,

h_s = story height, ft, and

b_d = diaphragm width, ft

"Where the *seismic-force-resisting system* consists of moment resisting frames, at least (N_b/4) + 1 of the bays (rounded up to the nearest integer) along any frame line at any story shall be part of the *seismic-force-resisting system* where N_b is the total number of bays along that line at that story. This requirement applies to only the lower two thirds of the stories of buildings three stories or taller.

"2. All beam-to-column connections that are not part of the *seismic-force-resisting system* shall be designed in accordance with the following:

"**Connection Design Force.** The connection shall be designed to develop *strength M*. *M* is the moment developed at the connection when the frame is displaced by Δ_s assuming fixity at the connection and a beam flexural stiffness of no less than one half of the gross section of stiffness. *M* shall be sustained through a deformation of Δ_m.

"**Connection Characteristics.** The connection shall be permitted to resist moment in one direction only, positive or negative. The connection at the opposite end of the member shall resist moment with the same positive or negative sign. The connection shall be permitted to have zero flexural stiffness up to a frame displacement of $\Delta_{m.}$

"In addition, complete calculations for the deformation compatibility of the gravity load carrying system shall be made in accordance with Sec. 5.2.2.4.3 of the 2000 NEHRP *Recommended Provisions* using cracked section stiffness in the *seismic-force-resisting system* and the *diaphragm.*

"Where gravity columns are not provided with lateral support on all sides, a positive connection shall be provided along each unsupported direction parallel to a principal plan axis of the structure. The connection shall be designed for a horizontal force equal to 4 percent of the axial load *strength*, $P_{o,}$ of the column.

"The bearing length shall be calculated to include end rotation, sliding, and other movements of precast ends at supports due to earthquake motions in addition to other movements and shall be at least 2 inches more than that required for bearing *strength*."

9.1.1.5: Change Sec. 21.2.5.1 as follows and insert the following new Sec. 21.2.5.2 and 21.2.5.3.

"**21.2.5.1** Except as permitted in 21.2.5.2 and 21.2.5.3, reinforcement resisting earthquake-induced flexural and axial forces in the frame members and in wall *boundary elements* shall comply with a ASTM A 706. ASTM A 615 Grades 40 and 60 reinforcement shall be permitted if (a) the actual yield strength based on mill tests does not exceed the specified yield *strength* by more than 18,000 psi (retests shall not exceed this value by more than an additional 3000 psi) and (b) the ratio of the actual ultimate tensile *strength* to the actual tensile yield *strength* is not less than 1.25.

"**21.2.5.2** Prestressing tendons shall be permitted in the flexural members of frames provided the average prestress, f_{pc}, calculated for an area equal to the member's shortest cross-sectional dimension multiplied by the perpendicular dimension shall not exceed the lesser of 700 psi or $f_c'/6$ at locations of nonlinear action where prestressing tendons are used in members of frames.

"**21.2.5.3** Unless the seismic-force-resisting frame is qualified for use through structural testing as required by 21.8.3.1, for members in which prestressing tendons are used together with mild steel reinforcement to resist earthquake-induced forces, prestressing tendons shall not provide more than one quarter of the strength for either positive or negative moment at the nonlinear action and be anchored at the exterior face of the joint or beyond.

"**21.2.5.4** Anchorages for tendons shall be demonstrated to perform satisfactorily for seismic loadings. Anchorage assemblies shall withstand, without failure, a minimum of 50 cycles of loading between 40 and 85 percent of the minimum tensile strength of the prestressing steel.

9.1.1.6: Add the following new Sec. 21.4.5.3:

"**21.4.5.3** At any section where the design strength, ϕP_n, of the column is less than the sum of the shear V_e computed in accordance with 21.4.5.1 for all the beams framing into the column above the level under consideration, special transverse reinforcement shall be provided. For beams framing into opposite sides of the column, the moment *components* may be assumed to be of opposite sign. For determination of the *nominal strength*, $P_{n,}$ of the column, these moments may be assumed to result from the *deformation* of the frame in any one principal axis."

9.1.1.7: In Sec. 21.6.3, change "factored load combinations" to "design load combinations."

9.1.1.8: Modify Sec. 21.6 by adding a new Sec. 21.6.10 to read as follows:

"21.6.10 Wall Piers and Wall Segments

"21.6.10.1 Wall piers not designed as part of a special moment frame shall have transverse reinforcement designed to satisfy the requirements of Sec. 21.6.10.2

"Exceptions:

"1. Wall piers that satisfy Sec. 21.9.

"2. Wall piers along a wall line within a story where other shear wall segments provide lateral support to the wall piers and such segments have a total stiffness of at least six times the sum of the stiffness of all the wall piers.

"**21.6.10.2** Transverse reinforcement shall be designed to resist the shear forces determined from Sec. 21.4.5.1 and 21.3.4.2. Where the axial compressive force, including earthquake effects , is less than $A_s f_c'/20$, transverse reinforcement in wall piers is permitted to have standard hooks at each free end and in lieu of hoops. Spacing of transverse reinforcement shall not exceed 6 inches (152 mm). Transverse reinforcement shall be extended beyond the pier clear height for at least the development length of the largest longitudinal reinforcement in the wall pier."

9.1.1.9: Change Sec. 21.8.1.1 to read as follows:

"**21.8.1.1** Foundations resisting earthquake-induced forces or transferring earthquake-induced forces between structure and ground shall comply with requirements of Sec. 21.8 and other applicable code provisions unless modified by Chapter 7 of the 2000 *NEHRP Recommended Provisions.*

9.1.1.10: Revise Sec. 21.9.3.3 to read as follows:

"**21.9.23.3** Members with factored gravity axial force exceeding $A_g f_c'/10$ shall satisfy Sec. 21.4.3.1, 21.4.4, 21.4.5, and 21.5.2.1. The maximum longitudinal spacing of ties shall be s_o for the full column height. The spacing, s_o, shall not be more than six diameters of the smallest longitudinal bar enclosed or 6 in. (152 mm), whichever is smaller. Lap splices of longitudinal reinforcement in such members need not satisfy Sec. 21.4.3.2 in structures where the seismic-force-resisting system does not include special moment frames."

9.1.1.11: Insert in Sec. 21.10 the following new Sec. 21.10.7:

"21.10.7 Precast concrete moment frames. For structure assigned to *Seismic Design Category* C, precast concrete seismic-force-resisting frames shall be permitted provided the frame conforms to 21.10.1 through 21.10.5 and Type Y or Type Z connections as defined in 21.11.5 are used."

9.1.1.12: Insert new Sec. 21.11 as follows:

"**21.11 Precast concrete special moment frames and special structural walls.**

"21.11.1 Scope.

Requirements of 21.11 apply to special moment frames, and to special reinforced concrete structural walls, resisting the earthquake induced forces and utilizing precast concrete elements."

"21.11.2 Seismic-force-resisting system requirements.

"21.11.2.1 A precast *seismic-force-resisting system* shall be permitted providing it satisfies either of the following criteria:

"1. The behavior of the system for forces up to and including those at the design displacement emulates the behavior of monolithic *reinforced concrete* construction and the system satisfies 21.11.3 or

"2. The system relies on its unique properties as a structure composed of interconnected precast elements and is demonstrated by experimental evidence and analysis to satisfy Sec. 21.2 through 21.7 of Chapter 21. Substantiating experimental evidence of acceptable performance of those elements required to sustain inelastic *deformations* shall be based upon cyclic testing of specimens representing those elements and shall satisfy 21.11.4.1 for special precast concrete moment frames and 21.11.4.2 for special precast concrete structural walls.

"**21.11.3 Emulation Design.** Precast structural systems emulating the behavior of monolithic reinforced concrete construction shall satisfy 21.11.3.1 where ductile connections are used and 21.11.3.2 where strong connections are used.

"21.11.3.1 Precast structural systems utilizing either wet or dry ductile connections at nonlinear action locations shall comply with all the applicable requirements of monolithic concrete construction for resisting *seismic forces* and satisfy the following:

"1. Where the moment acting on the connection is assumed equal to M_{pr}, the co-existing shear on the connection shall be no greater than $0.5S_{n\ Connection}$ and

"2. The nominal shear strength for the connection shall not be less than the shear strengths of the members immediately adjacent to that connection.

"**21.11.3.2** Precast structural systems not meeting the requirements of 21.11.2 shall utilize strong connections resulting in nonlinear response away from connections. Design shall satisfy the requirements of 21.11.5 in addition to the applicable requirements of monolithic concrete construction for resisting *seismic forces* except that the provisions of 21.3.1.2 shall apply to segments between nonlinear action locations.

"21.11.4 Interconnected element design.

"21.11.4.1 For special moment frames composed of interconnected precast elements, substantiating experimental evidence and analysis shall satisfy the requirements of ACI ITG/T1.1. It shall also be demonstrated by experimental evidence that the modules used for the validation testing of ACI ITG/T1.1 have the ability to carry, at 5 percent or greater drift ratios, the gravity loads that act on them in the generic frame.

"**21.11.4.2** For special structural wall systems composed of precast elements substantiating experimental evidence and analysis shall meet the requirements of Sec. 4.1, 4.2, 5.2, 5.3, 6.0, 7.0 and 8.0 of ACI ITG/T1.1 and:

2. The minimum stack module shall be a stack of wall panels at least two panels high.

2. For the third cycle at drift ratios equal to or exceeding the limiting drift ratio, the criteria for acceptance shall be that:

   a. Lateral load resisting capacity of the module shall be at least 80 percent of the peak lateral load;

   b. The relative energy dissipation ratio, as defined in ACI ITG/T1.1-99, shall equal or exceed 15 percent for that third cycle; and

   c. The stiffness at zero drift shall equal or exceed that required by ACI ITG/T1.1-99.

"3. The limiting drift ratio in percent shall satisfy the following criterion:

$$1.0 \leq 0.67\left[h_w / l_w\right] + 0.5 \leq 3.0$$

"21.11.4.3 Unless there is substantial experimental evidence obtained during a prior development program, the validation tests required in Sec. 21.11.4.1 and 21.11.4.2 shall:

1. Be conducted at full scale and

2. Be at least two in number for each characteristic configuration of intersecting beams and columns or structural walls.

"**21.11.4.4** The nonlinear response history analysis of Sec. 5.7 of the 2000 *NEHRP Recommended Provisions* shall be used to design the special precast concrete moment frame and structural wall systems using the force-deformation characteristics from the subassemblage tests required by Sec. 21.11.4.1 or Sec. 21.11.4.2.

"21.11.5 Emulation design of frames using strong connections.

"21.11.5.1 Location. Nonlinear action location shall be selected so that there is a strong column/weak beam *deformation* mechanism under seismic effects. The nonlinear action location shall be no closer to the near face of strong connection than *h/2.* For column-to-footing connections where nonlinear action may occur at the column *base* to complete the mechanism, the nonlinear action location shall be no closer to the near face of the connection than *h/2*.

"21.11.5.2 Anchorage and splices. Reinforcement in nonlinear action region shall be fully developed outside both the strong connection region and the nonlinear action region. Noncontinuous anchorage reinforcement of strong connection shall be fully developed between the connection and the beginning of nonlinear action region. Lap splices are prohibited within connections adjacent to a *joint.*

"21.11.5.3 Design forces. *Design strength* of strong connections shall be based on :

$$\phi S_{nConnection} \geq \psi S_{eConnection}$$

" The dynamic amplification factor, ψ, shall be taken as 1.0.

"21.11.5.4 Column-to-column connection. The *strength* of such connection shall comply with 21.2.7.3 with the ψ taken as 1.4. Where the column-to-column connections occur, the columns shall be provided with transverse reinforcement as specified in 21.4.4.1 through 21.4.4.3 over their full height in the factorial axial compressive force in these members, including seismic effects, exceeds $A_g f_c'/10$.

> **"Exception:** Where column-to-column connection is located within the middle third of the column clear height, the following shall apply: (a) the *design* moment *strength,* ϕM_n, the connection shall not be less than 0.4 times the maximum M_{pr} for the column within the *story* height and (b) the *design* shear strength, ϕV_n, of the connection shall not be less than that determined per 21.4.5.1.

"21.11.5.5 Column-face connection. Any strong connection located outside the middle half of a beam span shall be a wet connection unless a dry connection can be substantiated by approved cyclic test results. Any mechanical connector located within such a column-face strong connection shall develop in tension or compression, as required, at least 40 percent of the specified yield *strength*, f_y, of the bar.

"21.11.6 Connections.

"21.11.6.1 At dry connections that are nonlinear action locations, displacements both in the direction of the action of the connection and the transverse to it shall be controlled.

"21.11.6.2 Dry connections shall be mechanical splices that are Type 2 at nonlinear action locations. Type 1 mechanical splices and welded splices shall be permitted elsewhere.

"21.11.6.3 Based on the results of physical experiments and analytical modeling of connections, their anchorage and their connection regions, both dry and wet connections at nonlinear action locations shall be classified as either Type Y or Type Z connections, as follows:

1. Type Y connections shall conform to 21.11.6.4 and develop, for the specified loading, ductility ratios greater than 4.0.

2. Type Z connections shall conform to 21.11.6.5 and develop, for the specified loading ductility ratios greater than 8.0.

"Testing of connections and evaluation of results shall be made in accordance with the principles specified in ACI ITG/T1.1 and ATC-24.

"**21.11.6.4** Type Y connections shall develop under flexural, shear, and axial load actions, as required, a probable strength, S_{pr}, determined using a ϕ value of unity, that is not less than 125 percent or more of the yield strength, S_y, of the connection. The anchorage on either side of the connection shall be designed to develop $1.3S_{pr}$ in tension and shall be connected directly by mechanical splice, welded splice, or tension lap splice to the principal reinforcement of the precast or cast-in-place element.

"**21.11.6.5** Type Z connections shall develop under flexural, shear and axial load actions, as required, a probable strength, S_{pr}**,** determined using a ϕ value of unity, that is not less than 140 percent of the yield strength, S_y of the connection. The anchorage on either side of the connection shall be designed to satisfy the requirements for Type Y connections in tension and in compression, and the connection region shall be able to develop $1.3S_{pr}$. Equilibrium-based plasticity models (strut-and-tie models) shall be permitted to be used for design of the connection region. Confinement reinforcement in the form of closed hoops or spirals with a yield force equal to not less than 5 percent of the compressive force and having a spacing not greater than 3 in. shall be provided where the local compressive stress exceeds $0.7f_c'$.

> **"Exception:** Connections in nonlinear action regions in test modules used to qualify seismic-force resisting systems for use based on structural testing in accordance with 21.11.3 shall be deemed to satisfy the requirements of this provision.

"21.11.6.6 Connections at nonlinear action locations shall be Type Z connections. Type Y connections shall be permitted at cross sections other than nonlinear action locations.

"21.11.6.7 Connections and the structural *components* of which they are part shall have a quality assurance plan satisfying the requirements of the 2000 *NEHRP Recommended Provisions* Sec. 3.2.1 and 3.2.2."

9.2 ANCHORING TO CONCRETE:

9.2.1 Scope:

9.2.1.1: These provisions provide design requirements for structural anchors in concrete used to transmit structural loads from attachments into concrete members or from one connected member to another by means of tension, shear, or a combination of tension and shear. Safety levels specified are intended for in-service conditions rather than for short-term handling and construction conditions.

9.2.1.2: These provisions apply to both cast-in concrete anchors such as headed studs, headed bolts or hooked bolts, and post-installed anchors, such as expansion anchors and undercut anchors. Specialty inserts, through bolts, bolts anchored to embedded large steel plates, adhesive or grouted bounded anchors, and direct anchors, such as powder or pneumatic actuated nails or bolts are not included. Reinforcement used as a part of the embedment shall be designed in accordance with ACI 318.

9.2.1.3: Headed studs and headed bolts that have a geometry consistent with ASME B1.1, B18.2.1, and B18.2.6.9 shall be designed by Sec. 9.2.4. Hooked bolts that have a geometry that has been demonstrated to result in pullout strength without the benefit of friction in uncracked concrete equal to or exceeding 1.4 N_p (where N_p is given by Eq. 9.2.5.3.5) are included.

9.2.1.4: Post-installed anchors shall be tested before use to determine their nominal strength in uncracked concrete, cracked concrete, or both and also to verify their compliance with the requirements of these design provisions for reliability and performance under anticipated conditions of service. Such tests shall be conducted by an independent testing agency and shall be verified by a registered design professional with full description and details of the testing program, procedures, results, and conclusions. The test program shall be comprehensive and shall include tests, results, and analysis for the requirements of Sec. 9.2.1.4.1 and 9.2.1.4.3.

9.2.1.4.1 Reference Tests: Reference tests shall establish failure modes, technical data, and k factors to use with these design provisions for uncracked or cracked concrete or both.

9.2.1.4.2 Reliability Tests: Reliability tests shall establish the appropriate category for the fastener using tests of sensitivity to reduced installation effort, sensitivity to low strength concrete with larger tolerance drill bit in cracked or uncracked concrete, and sensitivity to high strength concrete with a lower tolerance drill bit in cracked or uncracked concrete. Tests shall also be performed in uncracked concrete with repeated loading and in cracked concrete with an opening and closing crack of at least 1,000 cycles to establish the fastener suitability.

9.2.1.4.3 Service-condition Tests: Service-condition tests shall establish requirements for edge distance, spacing between fasteners, splitting near an edge, shear capacity and pryout. For cracked concrete, simulated seismic qualification tests shall establish the ability of the fastener to perform in tension and shear under seismic conditions.

9.2.1.5: Load applications that are predominantly high cycle fatigue or impact are not covered by these provisions.

9.2.2 Notations and Definitions:

9.2.2.1 Notations:

A_b = bearing area of the head of the stud or anchor bolt (in^2).

A_{No} = projected concrete failure area of one anchor, for calculation of strength in tension, when not limited by edge distance or spacing, as defined in Sec. 9.2.5.2.1 (in^2). (see Figure C9.2.5.2.2-1).

A_N = projected concrete failure area of an anchor or group of anchors for calculation of strength in tension as defined in Sec. 9.2.5.2.1, in^2. A_N shall not be taken greater than nA_{No} (see Figure C9.2.5.2.1-2).

A_{se} = effective cross-sectional area of anchor (in^2).

A_{sl} = effective cross-sectional area of expansion or undercut anchor sleeve, if sleeve is within shear plane (in^2).

A_{Vo} = projected concrete failure area of one anchor, for calculation of strength in shear, when not limited by corner influences, spacing, or member thickness, as defined in Sec. 9.2.6.2.1 (in^2) (See Figure C9.2.6.2.1-1).

A_V = projected concrete failure area of an anchor or group of anchors, for calculation of strength in shear, as defined in Sec. 9.2.6.2.1 (in^2). A_V shall not be taken greater than $nA_{Vo.}$ (see Figure C9.2.6.2.1-2).

c = distance from center of an anchor shaft to the edge of concrete (in.).

c_1 = distance from the center of an anchor shaft to the edge of the concrete in one direction (in). Where shear force is applied to an anchor, c_1 is in the direction of the shear force (see Figure C9.2.6.2.1-2).

c_2 = distance from center of an anchor shaft to the edge of the concrete in the direction orthogonal to c_1 (in.).

c_{max} = the largest of the edge distances that are less than or equal to 1.5 h_{ef} (in.) used only for the case of 3 or 4 edges.

c_{min} = the smallest of the edge distances that are less than or equal to 1.5 h_{ef} (in.)

d_o = outside diameter of anchor or shaft diameter of headed stud, headed anchor bolt, or hooked anchor (in.) (see also Sec. 9.2.8.4).

d_u = diameter of head of stud or anchor bolt or equivalent diameter of effective perimeter of an added plate or washer at the head of the anchor (in.).

e_h = distance from the inner surface of the shaft of a J-bolt to the outer tip of the J- or L-bolt (in.).

e_N' = eccentricity of normal force on a group of anchors; the distance between the resultant tension load on a group of anchors in tension and the centroid of the group of anchors loaded in tension (in.) (see Figures C9.2.5.2.4-1 and -2).

e_v' = eccentricity of shear force on a group of anchors; the distance between the point of shear force application and the centroid of the group of anchors resisting shear in the direction of the applied shear (in.).

f_c' = specified compressive strength of concrete (psi).

f_{ct} = specified tensile strength of concrete (psi).

f_r = modules of rapture of concrete (psi).

f_t = calculated tensile stress in a region of a member (psi).

f_y = specified yield strength of anchor steel (psi).

f_{ut} = specified tensile strength of anchor steel (psi).

f_{utsl} = specified tensile strength of anchor sleeve (psi).

h = thickness of member in which an anchor is anchored measured parallel to the anchor axis (in.).

h_{ef} = effective anchor embedment depth (in.) (see Sec. 9.2.8.5 and Figure C9.2.2.2).

k = coefficient for basic concrete breakout strength in tension.

k_{cp} = coefficient for pryout strength.

l = load bearing length of anchor for shear, not to exceed $8d_o$ (in.).

l = h_{ef} for anchors with a constant stiffness over the full length of the embedded section, such as headed studs or post-installed anchors with one tubular shell over the full length of the embedment depth.

l = $2d_o$ for torque-controlled expansion anchors with a distance sleeve separated from the expansion sleeve.

n = number of anchors in a group.

N_b = basic concrete breakout strength in tension of a single anchor in cracked concrete, as defined in Sec. 9.2.5.2.2 (lb).

N_{cb} = nominal concrete breakout strength in tension of a single anchor, as defined in Sec. 9.2.5.2.1 (lb).

N_{cbg} = nominal concrete breakout strength in tension of a group of anchors, as defined in Sec. 9.2.5.2.1 (lb).

N_n = nominal strength in tension (lb).

N_p = pullout strength in tension of a single anchor in cracked concrete, as defined in Sec. 9.2.5.3.4 or 9.2.5.3.5 (lb).

N_{pn} = nominal pullout strength in tension of a single anchor, as defined in Sec. 9.2.5.3.1 (lb).

N_{sb} = side-face blowout strength of a single anchor (lb).

N_{sbg} = side-face blowout strength of a group of anchors (lb).

N_s = nominal strength of a single anchor in tension as governed by the steel strength, as defined in Sec. 9.2.5.1.2 (lb).

N_u = factored tensile load (lb).

s = anchor center-to center spacing (in).

s_o = spacing of the outer anchors along the edge in a group (in).

t = thickness of washer or plate (in).

V_b = basic concrete breakout strength in shear of a single anchor in cracked concrete, as defined in Sec. 9.2.6.2.2 or 9.2.6.2.3 (lb).

V_{cb} = nominal concrete breakout strength in shear of a single anchor as defined in Sec. 9.2.6.2.1 (lb).

V_{cbg} = nominal concrete breakout strength in a shear group of anchors as defined in Sec. 9.2.6.2.1 (lb).

V_{cp} = nominal concrete pryout strength, as defined in Sec. 9.2.6.3 (lb).

V_n = nominal shear strength (lb).

V_s = nominal strength in shear of a single anchor as governed by the steel strength as defined in Sec. 9.2.6.1.1 (lb).

V_u = factored shear load (lb).

ϕ = strength reduction factor (see Sec. 9.2.4.4 and 9.2.4.5).

Ψ_1 = modification factor, for strength in tension, to account for anchor groups loaded eccentrically as defined in Sec. 9.2.5.2.4.

Ψ_2 = modification factor, for strength in tension, to account for edge distances smaller than $1.5h_{ef}$ as defined in Sec. 9.2.5.2.5

Ψ_3 = modification factor, for strength in tension, to account for cracking as defined in Sec. 9.2.5.2.6 and 9.2.5.2.7.

Ψ_4 = modification factor, for pullout strength, to account for cracking as defined in Sec. 9.2.5.3.1 and Sec. 9.2.5.3.6.

Ψ_5 = modification factor, for strength in shear, to account for the anchor groups loaded eccentrically as defined in Sec. 9.2.6.2.5.

Ψ_6 = modification factor, for strength in shear, to account for edge distances smaller than $1.5c_1$ as defined in Sec. 9.2.6.2.6.

Ψ- = modification factor, for strength in shear, to account for cracking, as defined in Sec. 9.2.6.2.7

9.2.2.2 Definitions:

Anchor: A metallic element either cast into concrete or post-installed into a hardened concrete member and used to transmit applied loads including straight bolts, hooked bolts (J-or L-bolt), headed studs, expansion anchors, undercut anchors, or inserts.

Anchor Group: A number of anchors of approximately equal effective embedment depth with each anchor spaced as less than three times its embedment depth from one or more adjacent anchors.

Anchor Pullout Strength: The strength corresponding to the anchoring device or a major *component* of the device sliding out from concrete without breaking out a substantial portion of the surrounding concrete.

Attachment: The structural assemble, external to the surface of the concrete, that transmits loads to the anchor.

Brittle Steel Element: An element with a tensile test elongation of less than 14 percent over a 2 in. gage length reduction in area of less than 40 percent, or both.

Concrete Breakout Strength: The strength corresponding to a volume of concrete surrounding the anchor or group of anchors separating from the memeber.

Concrete Pryout Strength: The strength corresponding to formation of a concrete spall behind a short, stiff anchor with an embedded base that is 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 but does not expand.

Ductile Steel Element: An element with a tensile test elongation of at least 14 percent over a 2 in. gage length and reduction in area of at least 40 percent.

Edge Distance: The distance from the edge of the concrete surface to the center of the nearest anchor.

Effective Embedment Depth: The overall depth through which the anchor transfers force to the surrounding concrete. The effective embedment depth normally will be the depth of the 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. (See Figure C9.2.2.1.)

Expansion Anchor: A post-installed anchor inserted into hardened concrete that transfers loads into the concrete by direct bearing and/or friction. 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 a plug and the anchorage 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 pre-drilled hole.

5 Percent Fractile: A statistical term meaning 90 percent confidence that 95 percent of the actual strengths will exceed the nominal strength. Determination shall include the number of tests when evaluating data.

Hooked Bolt: A cast-in anchor anchored mainly by mechanical interlock from the 90-degree bend (L-bolt) or 180-degree bend (J-bolt) at its lower end.

Insert (specialty insert): Predesigned and pre-fabricated cast-in anchors specifically designed for attachment of bolted or slotted connections. Inserts often are used for handling, transportation, and erection but are used also for anchoring structural elements.

Post-Installed Anchor: An anchor installed in hardened concrete. 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 the 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.

Undercut Anchor: A post-installed anchor anchored mainly by mechanical interlock provided by an undercutting in the anchoring concrete. The undercutting is achieved with a special drill before installing the anchor or, alternatively, by the anchor itself during its installation.

9.2.3 General Requirements:

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

9.2.3.2: Except for load combinations that include earthquake forces or effects, anchors shall be designed for all load combinations outlined in Sec. 9.2 of ACI 318. Where resistance to specified earthquake loads or forces, E, are included in the design, the load combinations Sec. 5.2.7 of this document shall apply.

9.2.3.3: When anchor design includes seismic loads, the following additional requirements shall apply.

9.2.3.3.1: In structures assigned to *Seismic Design Categories* C, D, E, or F post-installed structural anchors for use under Sec. 9.2.3.2 shall have passed the simulated seismic tests in accordance with Sec. 9.2.1.4.

9.2.3.3.2: In structures assigned to *Seismic Design Categories* C, D, E or F, the design strength of anchors shall be taken as 0.75 ϕV_n, where ϕ is given in Sec. 9.2.4.4 or 9.2.4.5 and N_n and V_n are determined in accordance with Sec. 9.2.4.1.

9.2.3.3.3: In structures assigned to *Seismic Design Categories* C, D, E or F, anchors shall be designed to be governed by tensile or sheer strength of a ductile steel element unless Sec. 9.2.3.3.4 is satisfied.

9.2.3.3.4: In lieu of Sec. 9.2.3.3.3, the attachment that the anchor is connecting to the structure shall be designed so that the member being attached will undergo ductile yielding at a load level no greater than 75 percent of the minimum anchor design strength or the minimum anchor design strength is at least Ω_0 times the attachment force determined from design loads of the attached structure or 2.5 times the attachment force determined from the design loads of the attached nonstructural *component*.

9.2.3.4: All provisions for anchor axial tension and shear strength apply to normal-weight concrete. When lightweight aggregate concrete is used, provisions for N_n and V_n shall be modified by multiplying all values of $\sqrt{f_c'}$ affecting N_n and V_n by 0.75 for “all lightweight” concrete and 0.85 for “sand-lightweight” concrete. Linear interpolation shall be permitted when partial sand replacement is used.

9.2.3.5: The values of f_c' used for calculations in these provisions shall not exceed 10,000 psi for cast-in anchors and 8000 psi for post-installed anchors.

9.2.4 General Requirements for Strength of Structural Anchors:

9.2.4.1: Strength design of structural anchors shall be based on the computation or test evaluation of the following:

a. Steel strength of anchor in tension Sec. 9.2.5.1,

b. Steel strength of anchor is shear Sec. 9.2.6.1,

c. Concrete breakout strength of anchor in tension Sec. 9.2.4.2 and 9.2.5.2

d. Concrete breakout strength of anchor in shear Sec. 9.2.4.2 and 9.2.6.2,

e. Pullout strength of anchor in tension Sec. 9.2.4.2 and 9.2.5.3,

f. Concrete side-face blowout strength of anchor in tension Sec. 9.2.4.2 and 9.2.5.4,

g. Concrete pryout strength of anchor in shear Sec. 9.2.4.2 and 9.2.6.3,

h. Required edge distances, spacings and thickness to preclude splitting failure Sec. 9.2.4.2 and 9.2.8.

9.2.4.1.1: For the design of anchors, except as required in Sec. 9.2.3.3:

$$\phi N_n \geq N_u \qquad (9.2.4.1.1\text{-}1)$$

$$\phi V_n \geq V_u \qquad (9.2.4.1.1\text{-}2)$$

9.2.4.1.2: When both N_u and V_u are present, interaction effects shall be considered in accordance with 9.2.4.3.

9.2.4.1.3: In Eq. 9.2.4.1.1-1 and 9.2.4.1.1-2, 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_s, ϕN_{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_s, either ϕV_{cb} or ϕV_{cbg}, and ϕV_{cp}.

9.2.4.2: The nominal strength for any anchor and group of anchors 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 with the modifications made for 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. Limits on edge distances and anchor spacing in the design models shall be consistent with the tests that verified the model.

9.2.4.2.1: The effect of supplementary reinforcement provided to confine or restrain the concrete breakout, or both, shall be permitted to be included in the design models of Sec. 9.2.4.2.

9.2.4.2.2: For anchors with diameters not exceeding 2 in. and with tensile embedment not exceeding 25 in. in depth, the concrete breakout strength requirements of Sec. 9.2.4.2 shall be considered satisfied by the design procedure of Sec. 9.2.5.2 and 9.2.6.2.

9.2.4.3: 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. 9.2.7.

9.2.4.4: Strength reduction factor ϕ for anchoring to concrete shall be as follows when the load combinations of Sec. 9.2 of ACI 318 and Sec. 5.2.7 of this document are used:

Anchor governed by tensile or shear strength of a ductile steel element	0.90
Anchor governed by tensile or shear strength of a brittle steel element	0.75

Anchor governed by concrete breakout, blowout, pullout, or pryout

	Condition A	**Condition B**
i. Shear loads	0.85	0.75
ii. Tension loads		
Cast-in headed studs, headed bolts, or hooked bolts	0.85	0.75
Post-installed anchors with category as determined from anchor pre-qualification tests of Sec. 9.2.1.4		
Category 1 (low sensitivity to installation and high reliability)	0.85	0.75
Category 2 (medium sensitivity to installation and medium reliability)	0.75	0.65
Category 3 (high sensitivity to installation and lower reliability)	0.65	0.55

Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportional to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided to where pullout or pryout strength governs.

9.2.4.5: Strength reduction factor ϕ for fastening to concrete shall be as follows when the load combinations referenced in ASCE 7 are used:

Anchor governed by tensile or shear strength of a ductile steel element	0.80

Anchor governed by tensile or shear strength of a brittle steel element		0.70
Anchor governed by concrete breakout, blowout, pullout, or pryout		
	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 anchor prequalification tests of Sec. 9.2.1.4		
Category 1 (low sensitivity to installation and high reliability)	0.75	0.65
Category 2 (medium sensitivity to installation and medium reliability)	0.65	0.55
Category 3 (high sensitivity to installation and lower reliability)	0.55	0.45

Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided where pullout or pryout strength governs.

9.2.5 Design Requirements for Tensile Loading:

9.2.5.1 Steel Strength of Anchor in Tension:

9.2.5.1.1: The nominal strength of an anchor in tension as governed by the steel, N_s, shall be evaluated by calculations based on the properties of the anchor material and the physical dimensions of the anchor. Alternatively, it shall be permitted to use values based on the 5 percent fractile of the test results to establish values of N_s

9.2.5.1.2: Unless determined by the 5 percent fractile of test results, nominal strength of an anchor or group of anchors in tension shall not exceed the following:

For anchor material with a well-defined yield point

$$N_s = nA_{se} f_y$$

For anchor material without a well-defined yield point where f_{ut} shall not be taken greater than 125,000 psi

$$N_s = nA_{se}\left(0.8 f_{ut}\right)$$

9.2.5.2 Concrete Breakout Strength of Anchor in Tension:

9.2.5.2.1: Unless determined in accordance with Sec. 9.2.4.2, nominal concrete breakout strength of an anchor or group of anchors in tension shall not exceed the following:

For a single anchor

$$N_{cb} = \frac{A_N}{A_{No}}\psi_2\psi_3 N_b \qquad (9.2.5.2.1\text{-}1)$$

For a group of anchors

$$N_{cbg} = \frac{A_n}{A_{No}}\psi_1\psi_2\psi_3 N_b \qquad (9.2.5.2.1\text{-}2)$$

N_b is the basic concrete breakout strength value for a single anchor in tension in cracked concrete. A_n is the projected area of the failure surface for the 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 center lines of the anchor or, in the case of a group of anchors, from a line through a row of adjacent anchors. A_n shall not exceed nA_{No}, where n is the number of tensioned anchors in the group. A_{No} is the projected area of the failure surface of the single anchor in remote from edges:

$$A_{No} = 9h_{ef}^{\ 2}$$

9.2.5.2.2: Unless determined in accordance with Sec. 9.2.4.2, the basic concrete breakout strength of a single anchor in tension in cracked concrete shall not exceed:

$$N_b = \sqrt[k]{f_c'}h_{ef}^{\ 1.5} \qquad (9.2.5.2.2\text{-}1)$$

where

k = 24 for cast-in headed studs, headed bolts and hooked bolts and

k = 17 for post-installed anchors.

Alternatively, for cast-in headed studs and headed bolts with 11 in. < h_{ef} < 25 in., the basic concrete breakout strength of a single anchor in tension in cracked concrete shall not exceed:

$$N_b = \sqrt[k]{f_c'}\,h_{ef}^{\ 5/3} \qquad (9.2.5.2.2\text{-}2)$$

where k = 16.

9.2.5.2.3: For the special case of anchors in an application with three or four edges and the largest edg e distance c_{max} < 1.5 h_{ef}, the embedment depth h_{ef} 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 and -2 shall be limited to c_{max}/1.5.

9.2.5.2.4: The modification factor for eccentrically loaded anchor groups is:

$$\psi_1 = \frac{1}{\left(1+\frac{2e_N'}{3h_{ef}}\right)} \leq 1 \qquad (9.2.5.2.4)$$

Eq. 9.2.5.2.4 is valid for $e_n' \leq s/2$.

If the loading of 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. 9.2.5.2.4.

Where eccentric loading exists about two axes, the modification factor, y_l, shall be computed for each axis individually and the product of these factors used as y_l in Eq. 9.2.5.2.4

9.2.5.2.5: The modification factor for edge effects is:

$$\psi_2 = 1 \, if \; c_{\min} \geq 1.5h_{ef} \qquad (9.2.5.2.5\text{-}1)$$

$$\psi_2 = 0.7 + 0.3\frac{c_{\min}}{1.5h_{ef}} \, if \; c_{\min} < 1.5h_{ef} \qquad (9.2.5.2.5\text{-}1)$$

9.2.5.2.6: When an anchor is located in a region of a concrete member where analysis indicates no cracking ($f_t < f_r$) at service load levels, the following modification factor shall be permitted:

ψ_3 = 1.25 for cast-in headed studs, headed bolts, and hooked bolts and

ψ_3 = 0.4 for post-installed anchors.

9.2.5.2.7: When analysis indicates cracking at service load levels, y_3 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 Sec. 9.2.1.4. The cracking in the concrete shall be controlled by flexural reinforcement distributed in accordance with Sec. 10.6.4 of ACI 318 or equivalent crack control shall be provided by confining reinforcement.

9.2.5.2.8: When 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 *t* from the outer edge of the head of the anchor where *t* is the thickness of the washer or plate.

9.2.5.3 Pullout Strength of Anchor in Tension:

9.2.5.3.1: Unless determined in accordance with Sec. 9.2.4.2 the nominal pullout strength of an anchor in tension shall not exceed:

$$N_{pn} = \psi_4 N_p \qquad (9.2.5.3.1)$$

9.2.5.3.2: For post-installed expansion and undercut anchors, it is not permissible to calculate the pullout strength of tension. Values of N_p shall be based on 5 percent fractile of tests performed and evaluated according to Sec. 9.2.1.4.

9.2.5.3.3: For single cast-in headed studs and headed bolts, it shall be permitted to evaluate the pullout strength in tension using Sec. 9.2.5.3.4. For single J-bolts and L-bolts, it shall be permitted to evaluate the pullout strength in tension using Sec. 9.2.5.3.5. 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 test procedures of Sec. 9.2.1.4 but without the benefit of friction.

9.2.5.3.4: Unless determined in accordance with Sec. 9.2.4.2, the pullout strength in tension of a single headed stud or headed bolt, N_p for use in Eq. 9.2.5.3.1 shall not exceed:

$$N_p = A_b 8 f'_c \tag{9.2.5.3.4}$$

9.2.5.3.5: Unless determined in accordance with Sec. 9.2.4.2 the pullout strength in tension of a single J-bolt or L-bolt, N_p for use in Eq. 9.2.5.3.1 shall not exceed:

$$N_p = 0.9 f'_c e_h d_o \tag{9.2.5.3.5}$$

where $3d_o < e_h < 4.5d_o$.

9.2.5.3.6: For an anchor located in a region of concrete member where analysis indicates no cracking ($f_t < f_r$) at service load levels, the following modification factor shall be permitted:

$$\psi_4 = 1.4$$

Otherwise, Ψ, shall be taken as 1.0.

9.2.5.4 Concrete Side-Face Blowout Strength of a Headed Anchor in Tension:

9.2.5.4.1: For a single headed anchor with deep embedment close to an edge $c < 0.4h_{ef}$), unless determined in accordance with Sec. 9.2.4.2, the nominal side-face blowout strength N_{sb} shall not exceed:

$$N_{sb} = 160_c \sqrt{A_b} \sqrt{f'_c}$$

If the single anchor is located at a perpendicular distance, c_2, less than 3c from an edge, the value of N_{sb} shall be modified by multiplying it by the factor $(1 + c_2/c)/4$ where $1 < c_2/c < 3$.

9.2.5.4.2: For multiple headed anchors with deep embedment close to an edge $c < 0.4\ h_{ef}$) and spacing between anchors less than 6c, unless determined in accordance with Sec. 9.2.4.2, the nominal strength of the group of anchors for a side-face blowout failure, N_{sbg}, shall not exceed:

$$N_{sbg} = \left(1 + \frac{S_o}{6c}\right) N_{sb} \tag{9.2.5.4.2}$$

where s_o is the spacing of the outer anchors along the edge in the group and N_{sb} is obtained from Eq. 9.2.5.4.1 without the modification for a perpendicular edge distance.

9.2.6 Design Requirements for Shear Loading:

9.2.6.1 Steel Strength of Anchor in Shear:

9.2.6.1.1: The nominal strength of anchor in shear as governed by steel, V_s, shall be evaluated by calculations based on the properties of the anchor material and the physical dimensions of the anchor. Alternatively, it shall be permitted to use values based on the 5 percent fractile of test results to establish values of V_s.

9.2.6.1.2: Unless determined by the 5 percent fractile of test results, nominal strength of an anchor or group of anchors in shear shall not exceed the following:

a. For anchor material with a well-defined yield point:

$$V_s = nA_{se} f_y \tag{9.2.6.1.2-1}$$

b. For cast-in anchors without a well defined yield point:

$$V_s = n0.6A_{se} f_{ut} \tag{9.2.6.1.2-2}$$

where f_{ut} shall not be taken greater than 125,000 psi.

c. For post-installed anchors without a well-defined yield point:

$$V_s = n\left(0.6A_{se} f_{ut} + 0.4A_{sl} f_{utsl}\right) \tag{9.2.6.1.2-3}$$

where f_{ut} shall be taken greater than 125,000 psi.

9.2.6.1.3: Where anchors are used with built-up grout pads, the nominal strengths of Sec. 9.2.6.1.2 shall be reduced by 20 percent.

9.2.6.2: Concrete Breakout Strength of Anchors in Shear:

9.2.6.2.1: Unless determined in accordance with Sec. 9.2.4.2, nominal concrete breakout strength in shear of an anchor or group of anchors shall not exceed the following:

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

$$V_b = \frac{A_V}{A_{Vo}} \psi_6 \psi_7 V_b \tag{9.2.6.2.1-1}$$

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

$$V_{cbg} = \frac{A_v}{A_{vo}} \psi_5 \psi_6 \psi_7 V_b \tag{9.2.6.2.1-2}$$

For shear force parallel to an edge, V_{ch} or V_{cbg} shall be permitted to be twice the value for the force determined from Eq. 9.2.6.2.1-1 or 9.2.6.2.1-2- respectively, with ψ_6 taken equal to 1.

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.

V_b is the basic concrete breakout strength value for a single anchor. A_v 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 this area 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_1 shall be taken as the distance from the edge to this axis, A_y, and shall not exceed nA_{vo} where n is the number of anchors in the group

A_{vo} is the projected area for a single anchor in a deep member and remote from edges in the direction perpendicular to the shear force. It shall be permitted to evaluate this area as the base of a half pyramid with a side length parallel to the edge of $3c_1$ and the depth of $1.5\ c_1$.

$$A_{vo} = 4.5c_1^{\,2} \qquad (9.2.6.2.1\text{-}3)$$

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_1 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.

9.2.6.2.2: Unless determined in accordance with Sec. 9.2.4.2, the basic concrete breakout strength in shear of a single anchor in cracked concrete shall not exceed:

$$V_b = 7\left(\frac{l}{d_o}\right)^{0.2}\sqrt{d_o}\sqrt{f_c'}\,c_1^{1.5} \qquad (9.2.6.2.2)$$

9.2.6.2.3: For cast-in headed studs, headed bolts, or hooked bolts that are rigidly welded to steel attachments having a minimum thickness equal to the greater of 3/8 in. or half of the anchor diameter, unless determined in accordance with Sec. 9.2.4.2, the basic concrete breakout strength in shear of a single anchor in cracked concrete shall not exceed:

$$V_b = 8\left(\frac{1}{d_o}\right)^{0.2}\sqrt{d_o}\sqrt{f_c'}\,c_1^{1.5} \qquad (9.2.6.2.3)$$

provided that:

a. For groups of anchors, the strength is determined based on the strength of the row of anchors farthest from the edge,

b. The center-to-center spacing of the anchors is not less than 2.5 in., and

c. Supplementary reinforcement is provided at the corners if $c_2 \leq 1.5h_{ef}$.

9.2.6.2.4: For the special case of anchors in a thin member influenced by three or more edges, the edge distance c_1 used in Eq. 9.2.6.2.1-3, 9.2.6.2.2, 9.2.6.2.3, 9.2.6.2.5, 9.2.6.2.6-1 or -2 shall be limited to *h*/1.5.

9.2.6.2.5: The modification factor for eccentrically loaded anchor groups is:

$$\psi_5 = \frac{1}{1+\frac{2e_v'}{3c_1}} \leq 1 \qquad (9.2.6.2.5)$$

Eq. 9.2.6.2.5 is valid for $e_v' \leq s/2$.

9.2.6.2.6: The modification factor edge effects is:

$$\psi_6 = 1 \; if \; c_2 \geq 1.5c_1 \qquad (9.2.6.2.6\text{-}1)$$

$$\psi_6 = 0.7 + 0.3\frac{c_2}{1.5c_1} \; if \; c_2 < 1.5c_1 \qquad (9.2.6.2.6\text{-}2)$$

9.2.6.2.7: For anchors located in a region of a concrete member where analysis indicates no cracking ($f_t < f_r$) at the service loads, the following modification factor shall be permitted:

$$\psi_7 = 1.4$$

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. In order to be considered as edge reinforcement, the reinforcement shall be designed to resist the concrete breakout:

ψ = 1.0 for anchors in cracked concrete with no edge reinforcement or edge reinforcement smaller that a No. 4 bar

ψ = 1.2 for anchors in cracked concrete with edge reinforcement of a No. 4 bar or greater between the anchor and the edge

ψ = 1.4 for anchors in cracked concrete with edge reinforcement of a No. 4 bar or greater between the anchor and the edge and with the edge reinforcement enclosed within stirrups spaced at not more than 4 in.

9.2.6.3 Concrete Pryout Strength of Anchor in Shear:

9.2.6.3.1: Unless determined in accordance with Sec. 9.2.4.2, the nominal pryout strength, V_{cp}, shall not exceed:

$$V_{cp} = k_{cp} N_{cb} \qquad (9.2.6.3.1)$$

where

k_{cp} = 1.0 for $h_{ef} < 2.5$ in.,

k_{cp} = 2.0 for $h_{ef} > 2.5$ in., and N_{cb} shall be determined from Eq. 9.2.5.2-1 (lb).

9.2.7 Interaction of Tensile and Shear Forces: Unless determined in accordance with Sec. 9.2.4.3, anchors or groups of anchors that are subjugated to both shear and axial loads shall be designed to satisfy the requirements of Sec. 9.2.7.1 through 9.2.7.3. The value of ϕN_n shall be the smallest of shear strength of the anchor in tension, concrete breakout strength of anchor in tension, pullout strength of anchor in tension, and side-face blowout strength. The value of ϕV_n shall be the smallest of the steel strength of an anchor and shear, the concrete breakout strength of anchor in shear, and the pryout strength.

9.2.7.1: If $V_u \leq 0.2\ \phi V_n$, then full strength in tension shall be permitted: $\phi N_n > N_u$

9.2.7.2: If $N_u \leq 0.2\ \phi N_n$, then full strength in shear shall be permitted: $\phi V_n > V_u$

9.2.7.3: If $V_u > 0.2\ \phi V_n$ and $N_u > 0.2\ \phi N_n$, then:

$$\frac{N_u}{\phi N_n} + \frac{V_u}{\phi V_n} \leq 1.2 \qquad (9.2.7.3)$$

9.2.8 Required Edge Distances, Spacings, and Thicknesses to Preclude Splitting Failure: Minimum spacings and edge distances for anchors and minimum thickness of members shall conform to Sec. 9.2.8.1 through Sec. 9.2.8.5, unless reinforcement is provided to control splitting. Lesser values from product-specific tests performed in accordance with prequalification test in accordance with Sec. 9.2.1.4 shall be permitted.

9.2.8.1: Unless determined in accordance with Sec. 9.2.8.4, minimum center-to-center spacing of anchors will be $4d_o$ for untorqued cast-in anchors that will not be torqued or otherwise pretensioned and $6d_o$ for torqued or otherwise pretensioned cast-in anchors and all post-installed anchors.

9.2.8.2: Unless determined in accordance with Sec. 9.2.8.4, minimum edge distances for cast-in headed anchors that will not be torqued or otherwise pre-tensioned shall be based on minimum cover requirements for reinforcement in Sec. 7.7 of ACI 318. For cast-in headed anchors that will be torqued or otherwise pretensioned, the minimum edge distances shall be $6d_o$.

9.2.8.3: Unless determined in accordance with Sec. 9.2.8.4, minimum edge distances for post-installed anchors shall be based on the greater of the minimum cover requirements for reinforcement in Sec. 7.7 of ACI 318 or the minimum edge distance requirements for the products as determined by tests in accordance with Sec. 9.2.1.4 and shall not be less than 2.0 times the maximum aggregate size. In the absence of such product-specific test information, the minimum edge distance shall be taken as not less than:

$6d_o$ for undercut anchors

$8d_o$ for torque-controlled anchors

$10d_o$ for displacement-controlled anchors

9.2.8.4: For anchors where installation does not produce a splitting force and the anchors will remain untorqued, if the edge distance or spacing is less than that specified in Sec. 9.2.8.1 to 9.2.8.3, calculations shall be performed using a fictitious value of d_o that meets the requirements of Sec.9.2.8.1 to 9.2.8.3. Calculated forces applied to the anchor shall be limited to values corresponding to an anchor having the fictitious diameter.

9.2.8.5: The value of h_{ef} for an expansion or undercut post-installed anchor shall not exceed the greater of either two thirds of the member thickness or the member thickness less than 4 in.

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

9.2.9 Installation of Anchors: Anchors shall be installed in accordance with the project drawings and specifications.

9.3 CLASSIFICATION OF SHEAR WALLS: Structural concrete *shear walls* that resist *seismic forces* shall be classified in accordance with Sec. 9.3.1 through 9.3.4.

9.3.1 Ordinary Plain Concrete Shear Walls: Ordinary *plain concrete shear walls* are *walls* conforming to the requirements of Chapter 22 of ACI 318.

9.3.2 Detailed Plain Concrete Shear Walls: Detailed *plain concrete shear walls* are *walls* above the *base* conforming to the requirements of Chapter 22 of ACI 318 and containing reinforcement as follows:

Vertical reinforcement of at least 0.20 $in.^2$ (129 mm^2) in cross-sectional area shall be provided continuously from support to support at each corner, at each side of each opening, and at the ends of *walls*. The continuous vertical bar required by Sec. 22.6.6.5 of ACI 318 shall be provided.

Horizontal reinforcement at least 0.20 $in.^2$ (129 mm^2) in cross-sectional area shall be provided:

a. Continuously at structurally connected roof and floor levels and at the top of *walls*,

b. At the bottom of load-*bearing walls* or in the top of foundations when doweled to the wall, and

c. At a maximum spacing of 120 inches (3050 mm).

Reinforcement at the top and bottom of openings, when used in determining the maximum spacing specified in Item c above, shall be continuous in the wall.

Basement, foundation, or other *walls* below the *base* shall be reinforced as required by Sec. 22.6.6.5 of ACI 318.

9.3.2.1 Ordinary Reinforced Concrete Shear Walls: *Ordinary reinforced concrete shear walls* are *walls* conforming to the requirements of ACI 318 exclusive of Chapters 21 and 22.

9.3.2.2 Special Reinforced Concrete Shear Walls: *Special reinforced concrete shear walls* are *walls* conforming to the requirements of ACI 318 in addition to the requirements for *ordinary reinforced concrete shear walls*.

9.4 SEISMIC DESIGN CATEGORY A: *Structures* assigned to *Seismic Design Category A* may be of any construction permitted in ACI 318 and the *Provisions*.

9.5 SEISMIC DESIGN CATEGORY B: *Structures* assigned to *Seismic Design Category B* shall conform to all the requirements for *Seismic Design Category A* and the additional requirements for *Seismic Design Category B* of this section and in other chapters of the *Provisions.*

9.5.1 Ordinary Moment Frames: Flexural members of *ordinary moment frames* forming part of the *seismic-force-resisting system* shall be designed in accordance with Sec. 7.13.2 of ACI 318 and at least two main flexural reinforcing bars shall be provided continuously top and bottom throughout the beams, through or developed within exterior columns or *boundary elements*.

Columns of *ordinary moment frames* having a clear height to maximum plan dimension ratio of 5 or less shall be designed for shear in accordance with Sec. 21.10.3 of ACI 318.

9.6 SEISMIC DESIGN CATEGORY C: *Buildings* assigned to *Seismic Design Category* C shall conform to all requirements for *Seismic Design Category* B and to the additional requirements for *Seismic Design Category* C of this section an in other chapters of the Provisions.

9.6.1 Seismic-Force-Resisting Systems: *Seismic-force-resisting systems* shall conform to Sec. 9.6.1.1 and Sec. 9.6.1.2.

9.6.1.1 Moment Frames: All *moment frames* that are part of the *seismic-force-resisting system* shall be *intermediate moment frames* or *special moment frames.*

9.6.1.2 Shear Walls: All *shear walls* that are part of the *seismic-force-resisting system* shall be *ordinary reinforced concrete shear walls* conforming to Sec. 9.3.3 or *special reinforced concrete shear walls* conforming to Sec. 9.3.4.

9.6.2 Discontinuous Members: Columns supporting reactions from discontinuous stiff members such as *walls* shall be designed for special load combinations in Sec. 5.2.7.1 and shall be provided with transverse reinforcement at the spacing s_o as defined in Sec. 21.10.5.1 of ACI 318 over their full height beneath the level at which the discontinuity occurs. This transverse reinforcement shall be extended above and below the column as required in Sec. 21.4.4.5 of ACI 318.

9.6.3 Plain Concrete: Structural *plain concrete* members in *buildings* assigned to *Seismic Design Category* C shall conform to ACI 318 and the additional requirements and limitations of this section.

9.6.3.1 Walls: Structural plain concrete walls are not permitted in structures assigned to *Seismic Design Category* C.

> **Exception:** Structural plain concrete basement, foundation, or other *walls* below the base are permitted in detached one- and two-family dwellings three stories or less in height constructed with stud bearing *walls*. Such *walls* shall have reinforcement in accordance with Sec. 22.6.6.5 of ACI 318.

9.6.3.2 Footings: Isolated footings of *plain concrete* supporting pedestals or columns are permitted provided the projection of the footing beyond the face of the supported member does not exceed the footing thickness.

> **Exception:** In detached one- and two-family dwellings three stories or less in height of light-frame construction, structural plain concrete basement walls, foundation, or other

walls below the basement shall be permitted. Such concrete walls shall have reinforcement in accordance with Sec. 22.6.6.5 of ACI 318.

Plain concrete footings supporting *walls* shall be provided with no less than two continuous longitudinal reinforcing bars. Bars shall not be smaller than No. 4 (#13) and shall have a total area of not less than 0.002 times the gross cross-sectional area of the footing. For footings that exceed 8 in. in thickness, a minimum of one bar shall be provided at the top and bottom of the footing. For foundation systems consisting of plain concrete footing and plain concrete stemwall, a minimum of one bar shall be provided at the top of the stemwall and at the bottom the footing. Continuity of reinforcement shall be provided at corners and intersections.

Exceptions:

1. In detached one- and two-family dwellings three stories or less in height and constructed with stud *bearing walls*, *plain concrete* footings supporting *walls* shall be permitted without longitudinal reinforcement.
2. Where a slab-on-ground is cast monolithically with the footing, one No. 5 (#16) bar is permitted to be located at either the top or bottom of the footing.

9.6.3.3 Pedestals: *Plain concrete* pedestals shall not be used to resist lateral *seismic forces.*

9.6.4 Anchor Bolts in the Tops of Columns: Anchor bolts that are set in the top of a column shall be provided with ties that completely enclose at least four longitudinal column bars. There shall be at least two No. 4 (#13) or three No. 3 (#10) ties within 5 inches of the top of the column. The ties shall have hooks on each free end that comply with Sec. 7.1.3 of ACI 318.

9.7 SEISMIC DESIGN CATEGORIES D, E, OR F: *Structures* assigned to *Seismic Design Category* D, E, or F shall conform to all of the requirements for *Seismic Design Category* C and to the additional requirements in this section.

9.7.1 Seismic-Force-Resisting Systems: *Seismic-force-resisting systems* shall conform to Sec. 9.7.1.1 and Sec. 9.7.1.2.

9.7.1.1 Moment Frames: All *moment frames* that are part of the *seismic-force-resisting system,* regardless of height, shall be *special moment frames.*

9.7.1.2 Shear Walls: All *shear walls* that are part of the *seismic-force-resisting system* shall be *spacial reinforced concrete shear walls* conforming to Sec. 9.3.4.

9.7.2 Frame Members Not Proportioned to Resist Forces Induced by Earthquake Motions: All frame members assumed not to contribute to lateral forces resistance shall conform to Sec. 21.9 of ACI 318 is modified by Sec. 9.1.1.11 of this chapter.

Appendix to Chapter 9

REINFORCED CONCRETE DIAPHRAGMS CONSTRUCTED USING UNTOPPED PRECAST CONCRETE ELEMENTS

Preface: Reinforced concrete diaphragms constructed using untopped precast concrete elements are permitted in the body of the 2000 *NEHRP Recommended Provisions* for Seismic Design Categories A, B, and C but not for Categories D, E, and F. For the latter the precast elements must be topped and the topping designed as the diaphragm. For resisting seismic forces, a composite-topping slab cast in place on precast concrete elements must have a thickness no less than 2 in. and a topping slab not relying on composite action with the precast elements must have a thickness not less than 2-1/2 in.

There are two principal reasons why a framework for the design of untopped diaphragms for Seismic Design Categories D, E, and F may be desirable. One relates to the performance of topping slab diaphragms in recent earthquakes and the other to durability considerations. The 1997 *Provisions* incorporated ACI 318-95 for which the provisions for topping slab diaphragms on precast elements were essentially the same as those in ACI 318-89. In the 1994 Northridge earthquake, performance was poor for structures where demands on the topping slab diaphragms on precast elements were maximized and the structures had been designed using ACI 318-89. The topping cracked along the edges of the precast elements and the welded wire reinforcement crossing those cracks fractured. The diaphragms became the equivalent of an untopped diaphragm with the connections between precast concrete elements, the connectors, and the chords not detailed for that condition. Another problem found with topping slab diaphragms was that the chords often took the large diameter bars, grouped closely together at the topping slab edge. Under severe loading, these unconfined chord bars lost bond with the concrete and with it, ability to transfer seismic forces.

The 2000 *NEHRP* incorporates ACI 318-99 which recognizes that for topping slab diaphragms a controlling condition is the in-plane shear in concrete along the edges of the precast elements. Ductility is provided by requiring that the topping slab reinforcement crossing the edges be spaced at not less than 10 in. on center. While those requirements are based on best available engineering judgement and evidence, they have not yet been proven to provide adequate safety either by laboratory testing or field performance. Due to the dimensions of the precast element relative to the thickness of the topping slab, it may well be prudent to have seismic provisions for diaphragms incorporating precast elements controlled by untopped diaphragm considerations and to have those provisions modified for topped diaphragms. Further, in geographic areas where corrosive environments are a significant concern, the construction of un-topped diaphragms using "pretopped" precast elements rather than topped elements is desirable.

This appendix provides a compilation of current engineering judgment on a framework for seismic provisions for untopped diaphragms. That framework does not, however, adequately address all the concerns needed for its incorporation into the body of the *Provisions.* This appendix proposes that a diaphragm composed of untopped elements be designed to remain elastic, and that the connectors be designed for limited ductility in the event that design forces are exceeded during earthquake response and some inelastic action occurs when the demands on the diaphragm are maximized. By contrast, for all other systems for Seismic Design Categories D, E and F, the philosophy of the 2000 *NEHRP* is to require significant ductility. For the approach of this appendix, critical issues are how best to define:

1. **The design forces for the diaphragm so that they are large enough to result in essentially elastic behavior when the demands on the diaphragm are maximized or whether that criterion is even achievable;**
2. **The relation between the response of the diaphragm, its dimensions, and the ductility demands on the connectors;**
3. **The ductility changes that occur for connectors under various combinations of in-plane and out-of-plane shear forces and tensile and compressive forces;**
4. **The boundary conditions necessary for testing and for application of the loading for the validation testing of connectors; and**
5. **The constraints on connector performance imposed by their size relative to the size of the diaphragm elements.**

The use of this appendix as a framework for laboratory testing, analyses of the performance of diaphragms in past earthquakes, analytical studies, and trial designs, is encouraged. Users should also consult the *Commentary* for guidance and references. Please direct all feedback on this appendix and its commentary to the BSSC.

9A.1 Background: ACI 318-99 was significantly revised for structural diaphragms to add new detailing provisions in response to the poor performance of some cast-in-place composite topping slab diaphragms during the 1994 Northridge earthquake. New code and commentary Sec. 21.7 and R21.7 were inserted into Chapter 21. In those provisions, cast-in-place composite topping slabs and cast-in-place topping slab diaphragms are permitted but no mention is made of untopped precast diaphragms. The evidence from the recently completed PRESS 5-story building test (*PCI Journal*, November-December 1999), from Italian and English tests (M. J. NM. Priestley, D. Sritharan, J. R. Connley, and S. Pampanin, "Preliminary Results and Conclusions from the PRESSS Five-Story Precast Concrete Test Building," *PCI Journal*, Vol. 44, No. 6, November-December 1999; K. S. Elliott, G. Davies, and W. Omar, "Experimental Hollow-cored Slabs Used as Horizontal Floor Diaphragms," *The Structural Engineer*, Vol. 70, No. 10, May 1922, pp. 175-187; M. Menegotto, Seismic Diaphragm Behavior of Untopped Hollow-Core Floors, *Proceedings*, FIP Congress, Washington, D. C., May 1994), and from the 1999 Turkey earthquake is that such diaphragms can perform satisfactorily if they are properly detailed and if they and their connections remain elastic under the force levels the diaphragms experience. However, further additions are needed to the ACI 318 requirements to make that possible both in terms of the forces for which diaphragms should be designed and the ductilities that should be inherent in connections as a second line of defense.

In this appendix, the untopped precast diaphragm is designed to remain elastic by requiring that its design forces be based on Eq. 5.2.5.4 and be not less than a minimum value dependent upon the seismic response coefficient, with both values multiplied by the overstrength and redundancy factors associated with the seismic-force-resisting system. In addition, the connections are required to be able to perform in a ductile manner in the unlikely event that the diaphragm is forced to deform inelastically.

9A.2 References: The following references are to be considered part of this appendix to the extent referred to in this document:

ACI 318-99/ ACI 318-99R	American Concrete Institute (ACI), *Building Code Requirements for Structural Concrete,* 1999
ACI ITG/T1.1	American Concrete Institute (ACI), *Acceptance Criteria For Moment Frames, Based on Structural Testing* (An ACI Provisional Standard), ITG/T1.1, 1999
ATC-24	Applied Technology Council (ATC), *Guidelines for Seismic Testing of Components of Steel Structures,* ATC-24, 1992.

9A.3 Untopped Precast Diaphragms:

9A.3.1: An untopped precast floor or roof shall be permitted as a structural diaphragm provided Sec. 9A.3.2 through 9A.3.3 are satisfied. Untopped diaphragms shall not be permitted in structures having plan irregularity Type 4 as defined in *Provisions* Table 5.2.3.2.

9A.3.2: Rational elastic models shall be used to determine the in-plane shear and tension/compression forces acting on connections that cross joints. For any given joint, the connections shall resist the total shear and total moment acting on the joint according to an elastic distribution of stresses

9A.3.3: The diaphragm design force shall be not less than the force calculated from either of the following two criteria:

1. $\rho'\Omega_o$ times the F_px value calculated from Eq. 5.2.5.4 but not less than $\rho'\Omega_o C_s w_p x$.
2. A shear force corresponding to 1.25 times that for yielding of the *seismic-force-resisting system* calculated using a Φ value of unity.

The overstrength factor, $'\Omega_o$, shall be that for the *seismic-force-resisting system* specified in *Provisions* Table 5.2.2 unless derived by analysis of the probable strength of the *seismic-force-resisting system* and shall not be taken as less than 1.25 times the yield strength of that system. The redundancy factor, ρ, shall be as specified in Sec. 5.2.4 and the seismic response coefficient ,$C_{s,}$ shall be that determined in accordance with *Provisions* Sec. 5.3.2.1.

9A.3.4: For diaphragms in buildings having plan irregularities Type 1a, 1b, 2 or 5 as defined in *Provisions* Table 5.2.3.2, the analysis required by Sec. 9A.3.3 shall explicitly include the effect of such irregularities as required by *Provisions* Sec. 5.2.6.

9A.3.5: Mechanical connections shall have design strength for the body of the connector greater than the factored forces determined in accordance with Sec. 9A.3.3 and 9A.3.4.

9A.3.6: Mechanical connections used at joints shall be shown by analysis and test results to develop, under reversed cyclic loading, the capacity in shear, tension, compression, or a combination as required by the analysis specified in Sec. 9A.3.2. Testing of connections and evaluation of results shall be made in accordance with the principles specified in ACI ITG/T1.1 and ATC-24. Connections shall develop for the specified loading ductility ratios equal to or grater than 2.0. Embedments for connections shall be governed by steel yielding and not by fracture of concrete or welds.

9A.3.7: Connections shall be designed using the strength reduction factors ϕ specified in ACI 318-99 and ACI 318-99R. When the ϕ factor is modified by Sec. 9.3.4 of ACI 318-99 and ACI 318-99R, the modified value shall be used for the diaphragm connections.

9A.3.8: Where the design relies on friction in grouted joints for shear transfer across the joints, shear friction resistance shall be provided by mechanical connectors or reinforcement.

9A.3.9: Cast-in place strips shall be permitted in the end or edge regions of precast *components* as chords or collectors. These strips shall meet the requirements for topping slab diaphragms. The reinforcement in the strips shall conform to 21.7.8.2 and 21.7.8.3 of ACI 318-99 and ACI 318-99R.

9A.3.10: In satisfying the compatibility requirement of *Provisions* Sec. 5.2.2.4.3, the additional deformation that results from the diaphragm flexibility shall be considered. The assumed flexural and shear stiffness properties of the elements that are part of the seismic-force-resisting system shall not exceed one-half of the gross-section properties, unless a rational cracked-section analysis is performed.

9A.3.11: Diaphragms shall have a quality assurance plan satisfying the requirements of *Provisions* Sec. 3.2.1.

9A.3.12: Ties to supporting members and bearing lengths shall satisfy the requirements for design force and geometry characteristics specified for the connections in ACI 318, Sec. 21.2.1.7 as modified by *Provisions* Sec. 9.1.1.4.