This change proposal has changes to selected definitions and to the seismic chapter (formerly Chapter 21, now Chapter 20).

The system of identifying the source of each different code subsection is not followed generally, as most of the code language comes from the same or similar location in Chapter 21. The system is used where necessary to show the source of a seemingly new provision.

Where a section number is changed within Chapter 20, no indication of the change is noted. If a section reference outside Chapter 20 is changed, or if other additions are deletions are made, they are shown using the underline and strikethrough convention. We use << >> to convey instructions or requests for future action.

Changes since the accepted version are shown shaded.

CHAPTER 2 – NOTATION AND DEFINITIONS

$h_x$ = maximum center-to-center horizontal spacing of crossties or hoop legs on all faces of the column measured in the plane of a cross section, Chapter 20

$h_u$ = laterally unsupported height at extreme compression fiber of wall or wall pier, in., see 20.10.6.4. Equivalent to $r_u$ for compression members.

$k_n$ = confinement effectiveness factor

$k_f$ = concrete strength factor

$n_l$ = number of longitudinal bars around the perimeter of a column core with rectilinear hoops that are laterally supported by the corner of hoops or by seismic hooks. A bundle of bars is counted as a single bar.

$\sigma$ = wall boundary extreme fiber concrete nominal compressive stress, Fig. R.20.10.6.4.2.

$\ll <$ CH030 $>$

**Beam** — A structural member subjected primarily to flexure and shear, with or without axial load or torsion. Beams in a moment frame that forms part of the lateral-force-resisting system are predominantly horizontal members. A girder is a beam.

**Column** — Structural member, usually vertical or predominantly vertical, used primarily to support axial compressive load but can also resist moment, shear, or torsion, with a ratio of height-to-least lateral dimension exceeding 3 used primarily to support axial compressive load. For a tapered member, the least lateral dimension is the average of the top and bottom dimensions of the smaller side. Columns used as part of a lateral-force-resisting system resist combined axial load, moment, and shear.

**Moment frame** — Frame in which members beams, slabs, columns, and joints resist forces predominantly through flexure, shear, and axial force. Beams or slabs are predominantly horizontal or nearly horizontal. Columns are predominantly vertical or nearly vertical. $\ll <$Commentary to be drafted to suggest occasionally inclined column in an otherwise vertical-column frame is OK, and beams following moderately sloping floors and roofs are OK, too.$>$ Moment frames designated as part of the seismic-force-resisting system shall be categorized as follows:

**Ordinary moment frame** — A cast-in-place or precast concrete beam-column or slab-column frame complying with the requirements of Chapters 1 through 18 and 21 through 23.
Ordinary moment frames assigned to Seismic Design Category B, also complying with
20.3.

Intermediate moment frame — A cast-in-place beam-column frame or two-way slab-column
frame without beams complying with the requirements of 20.4 in addition to Chapters 1 through 19
and 20 through 23 the requirements for ordinary moment frames. <<Note that Chapter 20 currently
does not cover one-way slabs as part of intermediate moment frames. Application of 20.4.2 is
impractical because it requires hoops.>>

Special moment frame — A cast-in-place beam-column frame complying with the requirements of
20.2.3 through 20.2.7, 20.2.8, and 20.6 through 20.8, or a precast beam-column frame complying
with the requirements of 20.2.3 through 20.2.7, 20.2.8 and 20.6 through 20.9. In addition, the
requirements of Chapters 1 through 19 and 21 through 23 for ordinary moment frames shall be
satisfied. It shall be permitted for a beam to frame into a special structural wall if the supporting
portion of the wall is reinforced as a column satisfying requirements of 20.7 and 20.8. <<Note: This
last sentence was reviewed and approved during the Fall 2010 Sub H meeting.>>

CHAPTER 20 — EARTHQUAKE-RESISTANT STRUCTURES

20.1 — Scope
20.1.1 — The provisions of this chapter shall apply for design and construction of reinforced concrete
structures assigned to Seismic Design Categories B through F, including, where applicable:
(a) Structural systems designated as part of the seismic-force-resisting system, including
diaphragms, moment frames, structural walls, and foundations; and
(b) Members not designated as part of the seismic-force-resisting system but required to support
other loads while undergoing deformations associated with earthquake effects. <<~>

20.1.2 — Structures designed according to the provisions of this chapter are intended to resist
earthquake motions through ductile inelastic response of selected members. <20.1.1.1>

20.2 — General requirements
20.2.1 — Structural Systems
20.2.1.1 — Scope
20.2.1.1.1 — Chapter 20 contains requirements for design and construction of reinforced concrete
members of a structure for which the design forces, related to earthquake motions, have been
determined on the basis of energy dissipation in the nonlinear range of response.

20.2.1.1 — All structures shall be assigned to a seismic design category (SDC) in accordance with
4.1.4.1.4.5.1. <<Note: Whether this remains here depends on final resolution of 1.1.8 and Ch. 4.>>

20.2.1.2 — All members shall satisfy requirements of Chapters 1 to 19 and 22 Chapters 1 to 19, 21
to 23, and 25. Structures assigned to SDC B, C, D, E, or F also shall satisfy 20.2.1.3 through 20.2.1.7,
as applicable. Where requirements of Chapter 20 conflict with those of other chapters of this Code, the
requirements of Chapter 20 shall apply.

20.2.1.3 — Structures assigned to SDC B shall satisfy 20.2.2.

20.2.1.4 — Structures assigned to SDC C shall satisfy 20.2.2 and 20.2.3.
20.2.1.5 — Structures assigned to SDC D, E, or F shall satisfy 20.2.2 through 20.2.8, and 20.12 through 20.14.

20.2.1.6 — Structural systems designated as part of the seismic-force-resisting system shall be restricted to those designated by the legally adopted general building code General Building Code of which this Code forms a part, or determined by other authority having jurisdiction in areas without a legally adopted building code. Except for SDC A, for which Chapter 20 does not apply, the following provisions shall be satisfied for each structural system designated as part of the seismic-force-resisting system, regardless of the SDC, in addition to the requirements of 20.2.1.3 through 20.2.1.5:

(a) Ordinary moment frames shall satisfy 20.3.
(b) Ordinary reinforced concrete structural walls shall satisfy the provisions in Chapter 20 applicable to the assigned Seismic Design Category, but in need not satisfy any provisions detailing provisions in Chapter 20, unless required by 20.2.1.3 or 20.2.1.4. <<Clarification added by ACI 318H>>
(c) Intermediate moment frames shall satisfy 20.4.
(d) Intermediate precast walls shall satisfy 20.5.
(e) Special moment frames shall satisfy 20.2.3 through 20.2.8 and 20.6 through 20.8.
(f) Special moment frames constructed using precast concrete shall satisfy 20.2.3 through 20.2.8 and 20.9.
(g) Special structural walls shall satisfy 20.2.3 through 20.2.8 and 20.10.
(h) Special structural walls constructed using precast concrete shall satisfy 20.2.3 through 20.2.8 and 20.11.

All special moment frames and special structural walls shall also satisfy 20.2.3 through 20.2.7.

This requirement is now added to (e) through (h) above so that it is not overlooked. In reorganizing 20.2.2 through 20.2.8, and to keep the wording simple, Anchoring to Concrete, formerly 20.2.8 but now 20.2.3, is now required for all special frames and walls. This corresponds to a substantive technical change in that special systems in SDC B formerly did not have to satisfy Anchoring to Concrete. Though substantive, it is inconsequential because special systems are rarely used in SDC B.>>

20.2.1.7 — A reinforced concrete structural system not satisfying the requirements of this chapter shall be permitted if it is demonstrated by experimental evidence and analysis that the proposed system will have strength and toughness equal to or exceeding those provided by a comparable monolithic reinforced concrete structure satisfying this chapter.

20.2.2 — Analysis and proportioning of structural members

20.2.2.1 — The interaction of all structural and nonstructural members that affect the linear and nonlinear response of the structure to earthquake motions shall be considered in the analysis.

20.2.2.2 — Rigid members assumed not to be a part of the seismic-force-resisting system shall be permitted provided their effect on the response of the system is considered and accommodated in the structural design. Consequences of failure of structural and nonstructural members that are not a part of the seismic-force-resisting system shall be considered.

20.2.2.3 — Structural members extending below the base of the structure that are required to transmit forces resulting from earthquake effects to the foundation shall comply with the requirements of Chapter 20 that are consistent with the seismic-force-resisting system above the base of the structure.

20.2.3 — Anchoring to concrete

Anchors resisting earthquake-induced forces in structures assigned to SDC C, D, E, or F shall conform to the requirements of 19.3. <<Section references require updating to version 2. Sections do...
not exist at the time of this writing. Note: This section was moved from 20.2.8 so that all requirements for SDC B and C would come first, followed by requirements for D, E, and F.

20.2.4 — Strength reduction factors

Strength reduction factors shall be as given in 9.3.4.

20.2.4.1 — For structures that rely on intermediate precast structural walls in Seismic Design Category D, E, or F, special moment frames, or special structural walls to resist earthquake effects, $E, \phi$ shall be modified as given in (a) through (c): <9.3.4, 318-08>

(a) For any structural member that is designed to resist $E, \phi$ for shear shall be 0.60 if the nominal shear strength of the member is less than the shear corresponding to the development of the nominal flexural strength of the member. The nominal flexural strength shall be determined considering the most critical factored axial loads and including $E$;

(b) For diaphragms, $\phi$ for shear shall not exceed the minimum $\phi$ for shear used for the vertical components of the primary seismic-force-resisting system;

(c) For beam-column joints and diagonally reinforced coupling beams, $\phi$ for shear shall be 0.85.

20.2.5 — Concrete in special moment frames and special structural walls

Specified compressive strength of concrete in special moment frames and special structural walls shall be in accordance with the special seismic systems requirements of Table 5.2.1.1.

20.2.4.1 — Requirements of 20.2.4 apply to special moment frames, special structural walls, and all components of special structural walls including coupling beams and wall piers.

20.2.4.2 — Specified compressive strength of concrete, $f'_c$, shall be not less than 3000 psi.

20.2.4.3 — Specified compressive strength of lightweight concrete, $f'_c$, shall not exceed 5000 psi unless demonstrated by experimental evidence that structural members made with that lightweight concrete provide strength and toughness equal to or exceeding those of comparable members made with normalweight concrete of the same strength. Modification factor $\lambda$ for lightweight concrete in this Chapter shall be in accordance with 8.6.1—5.2.4 unless specifically noted otherwise.

20.2.6 — Reinforcement in special moment frames and special structural walls

Reinforcement in special moment frames and special structural walls shall be in accordance with the special seismic systems requirements of 6.3.4. <<Chapter 6 requires revisions before it can be correctly cited. Current version has errors in requirements for deformed reinforcement and does not adequately constrain prestressed steel.>>

20.2.5.1 — Requirements of 20.2.5 apply to special moment frames, special structural walls, and all components of special structural walls including coupling beams and wall piers.

20.2.5.2 — Longitudinal and diagonally oriented deformed Deformed reinforcement resisting earthquake-induced flexural and axial forces shall comply with ASTM A706 Grade 60. ASTM A615 Grades 40 and 60 reinforcement shall be permitted in these members if:

(a) The actual yield strength based on mill tests does not exceed $f_y$ by more than 18,000 psi; and

(b) The ratio of the actual tensile strength to the actual yield strength is not less than 1.25.

<<Add commentary to clarify longitudinal direction for walls.>>

20.2.5.3 — Prestressing steel resisting earthquake-induced flexural and axial loads forces in frame members and in precast structural walls shall comply with ASTM A416 or A722.

20.2.5.4 — The value of $f_y$ used to compute the amount of confinement reinforcement shall not exceed 100,000 psi.
20.2.6.5 — The value of $f_c$ or $f_{tu}$ used in design of shear reinforcement shall conform to 11.4.2.

6.3.1. — Note: posted version of chapter 5 is missing these sections. Check later.

20.2.7 — Mechanical splices in special moment frames and special structural walls

20.2.7.1 — Mechanical splices shall be classified as either Type 1 or Type 2 mechanical splices, as follows:

(a) Type 1 mechanical splices shall conform to 12.14.3.2 and shall develop the specified tensile strength of the spliced bar. <<CB035>>

(b) Type 2 mechanical splices shall conform to 12.14.3.2 and shall develop the specified tensile strength of the spliced bar. <<CB035>>

20.2.7.2 — Type 1 mechanical splices shall not be used within a distance equal to twice the member depth from the column or beam face for special moment frames or from critical sections where yielding of the reinforcement is likely to occur as a result of inelastic lateral displacements. Type 2 mechanical splices shall be permitted to be used at any location, except as noted in 20.9.2.1(b).

<<There is some sentiment to move much of the text of 20.2.7 to Chapter 21, but it is not yet in Chapter 21. Similarly, there is some sentiment to not move this from the seismic chapter.>>

20.2.8 — Welded splices in special moment frames and special structural walls

20.2.8.1 — Welded splices in reinforcement resisting earthquake-induced forces shall conform to 12.14.3.2 and shall not be used within a distance equal to twice the member depth from the column or beam face for special moment frames or from critical sections where yielding of the reinforcement is likely to occur as a result of inelastic lateral displacements beyond the elastic range of behavior. <<CB035>>

20.2.8.2 — Welding of stirrups, ties, inserts, or other similar elements to longitudinal reinforcement that is required by design shall not be permitted.

20.2.8 — Anchoring to concrete

Anchors resisting earthquake-induced forces in structures assigned to SDC C, D, E, or F shall conform to the requirements of D.3.3. <<This was moved to 20.2.3.>>

20.3 — Ordinary moment frames

20.3.1 — Scope

Requirements of 20.3 apply to ordinary moment frames forming part of the seismic-force-resisting system.

20.3.2 — For beam-column frames, beams shall have at least two of the longitudinal bars continuous at both the top and bottom faces. Continuous bottom bars shall have area not less than one-fourth the maximum area of bottom bars along the span. These bars shall be anchored to develop $f_y$, in tension developed at the face of support. <<Note: The requirement for one-fourth of the bottom bars being continuous has long been required for all continuous beams regardless of the seismic design category; the requirement is repeated here as additional guidance to the engineer. The requirement to anchor for $f_y$ is from 12.11 of ACI 318-08.>>

<<This is a placeholder section that Sub H provides in consideration of intent to submit a change proposal to introduce this requirement. This text is not part of the current ballot. ...>>

20.3.x — For slab-column frames, slabs shall have at least two of the column strip longitudinal bars continuous along both the top and bottom faces. Continuous bottom bars shall have an area not less than one-fourth the maximum area of column strip bottom bars along the span. Bars required to be
continuous shall pass through the column core and shall be anchored to develop $f_y$ in tension at the face of support.

20.3.3 — Columns having unsupported length clear height less than or equal to five times the dimension $t_u \leq 5c_1$ shall be designed for shear in accordance with 20.4.3. have $\phi V_n$ not be less than the smaller of (a) and (b):

(a) The shear associated with development of nominal moment strengths of the restrained end of the unsupported length due to reverse curvature bending. Column flexural strength shall be calculated for the factored axial force, consistent with the direction of the lateral forces considered, resulting in the highest flexural strength.

(b) The maximum shear obtained from design load combinations that include $E$, with $\Omega_0 E$ substituted for $E$.

20.4 — Intermediate moment frames

20.4.1 — Scope

Requirements of 20.4 apply to intermediate moment frames including two-way slabs without beams forming part of the seismic-force-resisting system. <<Note: Struck material below appears later in 20.4.2 and 20.4.3, though in different form.>>

21.3.2 — Reinforcement details in a frame member shall satisfy 21.3.4 if the factored axial compressive load, $P_u$ for the member does not exceed $A_g f_c'/10$. If $P_u$ is larger, frame reinforcement details shall satisfy 21.3.5. Where a two-way slab system without beams forms a part of the seismic-force-resisting system, reinforcement details in any span resisting moments caused by $E$ shall satisfy 21.3.6.

21.3.3 — $\phi V_n$ of beams and columns resisting earthquake effect, $E$, shall not be less than the smaller of (a) and (b):

(a) The sum of the shear associated with development of nominal moment strengths of the member at each restrained end of the clear span and the shear calculated for factored gravity loads;

(b) The maximum shear obtained from design load combinations that include $E$, with $E$ assumed to be twice that prescribed by the legally adopted general building code for earthquake-resistant design.

20.4.2 — Beams

20.4.2.1 — Beams shall have at least two continuous bars at both top and bottom faces.

Continuous bottom bars shall have area not less than one-fourth the maximum area of bottom bars along the span. These bars shall be anchored to develop $f_y$ in tension at the face of support.

<<Note: The requirement for one-fourth of the bottom bars being continuous is required for all continuous beams. It is repeated here as additional guidance to the engineer. The requirement to anchor for $f_y$ is from 12.11.>>

20.4.2.2 — The positive moment strength at the face of the joint shall be not less than one-third the negative moment strength provided at that face of the joint. Neither the negative nor the positive moment strength at any section along the length of the beam shall be less than one-fifth the maximum moment strength provided at the face of either joint.
20.4.2.3 — \( \phi V_o \) of beams resisting earthquake effect, \( E \), shall not be less than the smaller of (a) and (b):

(a) The sum of the shear associated with development of nominal moment strengths of the beam at each restrained end of the clear span due to reverse curvature bending and the shear calculated for factored gravity loads;

(b) The maximum shear obtained from design load combinations that include \( E \), with \( E \) assumed to be taken as twice that prescribed by the General Building Code for earthquake-resistant design.

20.4.2.4 — At both ends of the beam, hoops shall be provided over lengths not less than 2h measured from the face of the supporting member toward midspan. The first hoop shall be located not more than 2 in. from the face of the supporting member. Spacing of hoops shall not exceed the smallest of (a), (b), (c), and (d):

(a) \( d / 4 \);

(b) Eight \( \Phi \) times the diameter of the smallest longitudinal bar enclosed;

(c) 24 times the diameter of the hoop bar;

(d) 12 in.

20.4.2.5 — Stirrups shall be Beams shall have transverse reinforcement spaced not more than \( d / 2 \) throughout the length of the beam.

20.4.2.6 — In beams having factored axial compressive force exceeding \( A_g f_c / 10 \), transverse reinforcement required by 20.4.2.5 shall conform to 7.10.5.1 and either 7.10.5.3 or 7.10.5.4. <<Note: Change proposals approved for ACI 318-11 resulted in modifications to Section 7.10. The section numbers called out here refer to those new section numbers. Section numbers will require updating for 318-13. Reorganized code sections available at the time of this writing do not contain all the required provisions.>>

20.4.3 — Columns

20.4.3.1 — \( \phi V_o \) of columns resisting earthquake effect, \( E \), shall not be less than the smaller of (a) and (b):

(a) The shear associated with development of nominal moment strengths of the column at each restrained end of the unsupported length due to reverse curvature bending. Column flexural strength shall be calculated for the factored axial force, consistent with the direction of the lateral forces considered, resulting in the highest flexural strength.

(b) The maximum shear obtained from design factored load combinations that include \( E \), with \( \Omega E \) substituted for \( E \).<<Note: Preceding code language is the same as provided in ACI 318-11, except as noted.>>

20.4.3.2 — Columns shall be spirally reinforced in accordance with 7.10.4 Chapter 14 or shall conform with 20.4.3.3 through 20.4.3.5. Section 20.4.3.6 shall apply to all columns, and 20.4.5.6 shall apply to all columns supporting discontinuous stiff members. <<Note: The generic reference to Chapter 14 was done because new section 21.7.6 does not contain all the details that were formerly in Chapter 14. It is similarly unclear whether all of them are in Chapter 14.>>

20.4.3.3 — At both ends of the column, hoops shall be provided at spacing \( s_o \) over a length \( l_o \) measured from the joint face. Spacing \( s_o \) shall not exceed the smallest of (a), (b), (c), and (d):

(a) Eight \( \Phi \) times the diameter of the smallest longitudinal bar enclosed;

(b) 24 times the diameter of the hoop bar;

(c) One-half of the smallest cross-sectional dimension of the column;
(d) 12 in.

Length $\ell_0$ shall not be less than the largest of (e), (f), and (g):

(e) One-sixth of the clear span of the column;

(f) Maximum cross-sectional dimension of the column;

(g) 18 in.

20.4.3.4 — The first hoop shall be located not more than $s_o / 2$ from the joint face.

20.4.3.5 — Outside the length $\ell_0$, spacing of transverse reinforcement shall conform to 7.10 and 14.7.4.

11.4.5.4 Joint transverse reinforcement shall conform to 11.10.<<Moved to 20.4.4.>>

20.4.3.6 — Columns supporting reactions from discontinuous stiff members, such as walls, shall be provided with transverse reinforcement at the spacing, $s_o$, as defined in 20.4.3.3 over the full height beneath the level at which the discontinuity occurs if the portion of factored axial compressive force in these members related to earthquake effects exceeds $A_g f_c / 10$. Where design forces have been magnified to account for the overstrength of the vertical elements of the seismic-force-resisting system, the limit of $A_g f_c / 10$ shall be increased to $A_g f_c / 4$. This transverse reinforcement shall extend above and below the column as required in 20.7.5.6(b).

20.4.4 — Joints

20.4.4.1 — Beam-column joints shall have transverse reinforcement conforming to 11.10.<<Insert appropriate Chapter 17 reference when ready>>

20.4.5 — Two-way slabs without beams

20.4.5.1 — Factored slab moment at support including earthquake effects, $E$, shall be determined for load combinations given in Eq. (9-5 7.3.1e) and (9-7 7.3.1g). Reinforcement provided to resist $M_{slab}$ shall be placed within the column strip defined in 13.2.1 8.3.2.3. <<Insert appropriate definition of column strip when it resurfaces>>.

20.4.5.2 — Reinforcement placed within the effective width specified in 13.5.3.2 12.4.3.3 shall be proportioned to resist $\gamma_M M_{slab}$. Effective slab width for exterior and corner connections shall not extend beyond the column face a distance greater than $c_I$ measured perpendicular to the slab span.

20.4.5.3 — Not less than one-half of the reinforcement in the column strip at support shall be placed within the effective slab width given in 13.5.3.2 12.4.3.3.

20.4.5.4 — Not less than one-quarter of the top reinforcement at the support in the column strip shall be continuous throughout the span.

20.4.5.5 — Continuous bottom reinforcement in the column strip shall be not less than one-third of the top reinforcement at the support in the column strip.

20.4.5.6 — Not less than one-half of all bottom middle strip reinforcement and all bottom column strip reinforcement at midspan shall be continuous and shall develop $f_y$ at face of support as defined in 13.6.2.5 8.6.2.3.3.

20.4.5.7 — At discontinuous edges of the slab, all top and bottom reinforcement at support shall be developed at the face of support as defined in 13.6.2.5 8.6.2.3.3.

20.4.5.8 — At the critical sections for columns defined in 11.11.2 9.6.5.1, two-way shear caused by factored gravity loads shall not exceed $0.4 V_c$, where $V_c$ shall be calculated in accordance with as defined in 11.11.2.1 9.6.8 for nonprestressed slabs and in 11.11.2.1 for prestressed slabs. It shall be permitted to waive this requirement if the slab design satisfies requirements of 20.14.6.
20.5 — Intermediate precast structural walls

20.5.1 — Scope
Requirements of 20.5 apply to intermediate precast structural walls forming part of the seismic-force-resisting system.

20.5.2 — Requirements
20.5.2.1 - In connections between wall panels, or between wall panels and the foundation, yielding shall be restricted to steel elements or reinforcement.
20.5.2.2 - Elements of the connection that are not designed to yield shall develop at least 1.5$S_y$.
20.5.2.3 – In structures assigned to SDC D, E, or F, wall piers shall be designed in accordance with 20.10.8 or 20.14.

<<Note the requirements of 20.5 have been organized under slightly different headings.>>

20.6 — Flexural members Beams of special moment frames

20.6.1 — Scope
(a) Requirements of 20.6 apply to beams of special moment frames that form part of the seismic-force-resisting system and are proportioned primarily to resist flexure and shear.
(b) Beams of special moment frames shall frame into columns of special moment frames satisfying columns and joints. Also, note that a special moment frame beam can frame into a wall if the supporting portion of the wall is reinforced as a column.>>

20.6.2 — Dimensional Limits
These frame members Beams shall also satisfy (a) through (c) the conditions of 20.6.1.1 through 20.6.1.4.

20.6.1.1 — (a) Factored axial compressive force on the member, $P_u$, shall not exceed $A_y f'_y / 10$.

<<See new 20.6.4.7 for the axial load clause.>>

20.6.1.2 — (a) Clear span for member, $l_n$, shall not be less than 4$d$ four times its effective depth.

20.6.1.3 — (b) Width of member, $b_w$, shall not be less than the smaller of 0.3$h$ and 10 in.

20.6.1.4 — (c) Projection of the beam width beyond the width of the supporting column on each side shall not exceed the smaller of $c_2$ and 0.75$c_1$.

20.6.1.5 — Width of member, $b_w$, shall not exceed width of supporting member, $c_2$, plus a distance on each side of supporting member equal to the smaller of (a) and (b):
(a) Width of supporting member, $c_2$, and
(b) 0.75 times the overall dimension of supporting member, $c_2$.

20.6.3 — Longitudinal reinforcement
20.6.3.1 — Beams shall have at least two continuous bars at both top and bottom faces. At any section, of a flexural member, except as provided in 10.6.3.13.6.2.2, for top as well as for bottom reinforcement, the amount of reinforcement shall not be less than that given by Eq. (10.3) required by 13.6.2.1 but not less than $200b_w d / f_y$, and the reinforcement ratio, $\rho$, shall not exceed 0.025. At least two bars shall be provided continuously at both top and bottom. <<Note that 13.6.2.1 covers both Eq. (10.3) and the 200bwdfy requirement. Note that the exception, for 13.6.2.2 has been eliminated;
though it is a substantive change, it never affects beams and, if it did, we would not want 13.6.2.2 to govern.>>

20.6.3.2 — Positive moment strength at joint face shall be not less than one-half the negative moment strength provided at that face of the joint. Neither the negative nor the positive moment strength at any section along member length shall be less than one-fourth the maximum moment strength provided at face of either joint.

20.6.3.3 — Lap splices of flexural deformed longitudinal reinforcement shall be permitted only if hoop or spiral reinforcement is provided over the lap length. Spacing of the transverse reinforcement enclosing the lap-spliced bars shall not exceed the smaller of $d/4$ and 4 in. Lap splices shall not be used in locations identified in (a) through (c):

(a) Within the joints;

(b) Within a distance of twice the member beam depth from the face of the joint; and

(c) Within a distance of twice the beam depth from critical sections where analysis indicates flexural yielding is likely to occur as a result of inelastic lateral displacements beyond the elastic range of behavior of the frame. <<Sub H presents this as a clarification of the intent of the original language, approved at the Fall 2010 meeting of Sub H.>>

20.6.3.4 — Mechanical splices shall conform to 20.2.7 and welded splices shall conform to 20.2.8.

20.6.3.5 — Prestressing, where used, shall satisfy (a) through (d), unless used in a special moment frame as permitted by 20.9.2.3:

(a) The average prestress, $f_{pc}$, calculated for an area equal to the smallest cross-sectional dimension of the member beam multiplied by the perpendicular cross-sectional dimension shall not exceed the smaller of 500 psi and $f_{pc} / 10$.

(b) Prestressing steel shall be unbonded in potential plastic hinge regions, and the calculated strains in prestressing steel under the design displacement shall be less than 1 percent.

(c) Prestressing steel shall not contribute to more than one-quarter of the positive or negative flexural strength at the critical section in a plastic hinge region and shall be anchored at or beyond the exterior face of the joint.

(d) Anchorages of the post-tensioning tendons resisting earthquake-induced forces shall be capable of allowing tendons to withstand 50 cycles of loading, bounded by 40 and 85 percent of the specified tensile strength of the prestressing steel.

20.6.4 — Transverse reinforcement <<This section has been reorganized to group provisions together more logically.>>

20.6.4.1 — Hoops shall be provided in the following regions of frame members beams:

(a) Over a length equal to twice the member beam depth measured from the face of the supporting member column toward midspan, at both ends of the flexural member beam;

(b) Over lengths equal to twice the member beam depth on both sides of a section where flexural yielding is likely to occur in connection with inelastic lateral displacements of the frame, as a result of lateral displacements beyond the elastic range of behavior.

20.6.4.2 — Where hoops are required, primary flexural longitudinal reinforcing bars closest to the tension and compression faces shall have lateral support conforming to 7.10.5.3 or 7.10.5.4

21.7.5 <<7.10.5.4 of ACI 318-11 is currently in 14.7.4.3. Not sure how this relates to beams. This note is kept as a placeholder to be certain it is not lost in the new code.>>. The spacing of laterally supported flexural reinforcing bars shall not exceed 14 in. Skin reinforcement required by 10.6.7 13.7.3.1.5 need not be laterally supported. <<Note this is language adopted for ACI 318-11.>>
20.6.4.6 — Hoops in flexural members shall be permitted to be made up of two pieces of reinforcement: a stirrup having seismic hooks at both ends and closed by a crosstie. Consecutive crossties engaging the same longitudinal bar shall have their 90-degree hooks at opposite sides of the flexural member. If the longitudinal reinforcing bars secured by the crossties are confined by a slab on only one side of the flexural frame member, the 90-degree hooks of the crossties shall be placed on that side.

20.6.4.2 — The first hoop shall be located not more than 2 in. from the face of a supporting member. Spacing of the hoops shall not exceed the smallest of (a), (b), and (c):

(a) \( \frac{d}{4} \);

(b) Six times the diameter of the smallest primary flexural reinforcing bars excluding longitudinal skin reinforcement required by 10.6.7.3.1.5; and

(c) 6 in.

Where hoops are required, they shall be proportioned to resist shear according to 20.6.5. Stirrups or ties required to resist shear shall be hoops over lengths of members identified in 20.6.4.1.

20.6.4.6 — Where hoops are not required, stirrups with seismic hooks at both ends shall be spaced at a distance not more than \( \frac{d}{2} \) throughout the length of the member.

20.6.4.5 — Stirrups or ties Transverse reinforcement required to resist shear shall be hoops over lengths of members identified in 20.6.4.1.

20.6.4.7 — In beams having factored axial compressive force exceeding \( \frac{A_g f'_c}{10} \), hoops satisfying 20.7.5.2 through 20.7.5.4 shall be provided along lengths identified in 20.6.4.1. Along the remaining length, hoops satisfying 20.7.5.2 shall have center-to-center spacing, \( s \), not exceeding the smaller of six times the diameter of the smallest longitudinal beam bars and 6 in. Where concrete cover over transverse reinforcement exceeds 4 in., additional transverse reinforcement having cover not exceeding 4 in. and spacing not exceeding 12 in. shall be provided. <<Some Sub H members feel additional reinforcement for members with large cover should be required for beams with small axial load as well. This, however, would be a substantive change. We provide this note as a reminder to consider this in future committee work.>>

20.6.5 — Shear strength requirements

20.6.5.1 — Design forces

The design shear force, \( V_e \), shall be determined from consideration of the statical forces on the portion of the member between faces of the joints. It shall be assumed that moments of opposite sign corresponding to probable flexural moment strength, \( M_{pr} \), act at the joint faces and that the member is loaded with the factored tributary gravity load along its span.

20.6.5.2 — Transverse reinforcement

Transverse reinforcement over the lengths identified in 20.6.4.1 shall be proportioned to resist shear assuming \( V_e = 0 \) when both (a) and (b) occur:

(a) The earthquake-induced shear force calculated in accordance with 20.6.5.1 represents one-half or more of the maximum required shear strength within those lengths;

(b) The factored axial compressive force, \( P_u \), including earthquake effects is less than \( \frac{A_g f'_c}{20} \).
20.7 — **Columns of special moment frames** Special moment frame members
subjected to bending and axial load

20.7.1 — **Scope**

Requirements of this section 20.7 apply to columns of special moment frames that form part of the seismic-force-resisting system and are proportioned primarily to resist flexure, shear, and axial forces, that resist a factored axial compressive force $P_u$ under any load combination exceeding $\frac{A_g f'c}{10}$. 

20.7.2 — **Dimensional Limits**

These frame members shall also satisfy (a) and (b) the conditions of 20.7.1.1 and 20.7.1.2.

20.7.1.1 — **(a)** The shortest cross-sectional dimension, measured on a straight line passing through the geometric centroid, shall not be less than 12 in.

20.7.1.2 — **(b)** The ratio of the shortest cross-sectional dimension to the perpendicular dimension shall not be less than 0.4.

20.7.3 — **Minimum flexural strength of columns**

20.7.3.1 — Columns shall satisfy 20.7.3.2 or 20.7.3.3.

20.7.3.2 — The flexural strengths of the columns shall satisfy Eq. (20.7.3.2)

$$\sum M_{nc} \geq (6/5)\sum M_{nb} $$

(20.7.3.2)

$\sum M_{nc}$ = sum of nominal flexural strengths of columns framing into the joint, evaluated at the faces of the joint. Column flexural strength shall be calculated for the factored axial force, consistent with the direction of the lateral forces considered, resulting in the lowest flexural strength.

$\sum M_{nb}$ = sum of nominal flexural strengths of the beams framing into the joint, evaluated at the faces of the joint. In T-beam construction, where the slab is in tension under moments at the face of the joint, slab reinforcement within an effective slab width defined in 8.12.8.3.1.1 shall be assumed to contribute to $M_{nb}$ if the slab reinforcement is developed at the critical section for flexure.

Flexural strengths shall be summed such that the column moments oppose the beam moments. Equation (20.7.3.2) shall be satisfied for beam moments acting in both directions in the vertical plane of the frame considered.

20.7.3.3 — If 20.7.3.2 is not satisfied at a joint, the lateral strength and stiffness of the columns framing into that joint shall be ignored when determining the calculated strength and stiffness of the structure. These columns shall conform to 20.14.

20.7.4 — **Longitudinal reinforcement**

20.7.4.1 — Area of longitudinal reinforcement, $A_{sl}$, shall not be less than 0.01$A_g$ or more than 0.06$A_g$.

20.7.4.2 — In columns with circular hoops, the minimum number of longitudinal bars shall be 6.

20.7.4.3 — Mechanical splices shall conform to 20.2.6 20.2.7, and welded splices shall conform to 20.2.7 20.2.8. Lap splices shall be permitted only within the center half of the member length, shall be designed as tension lap splices, and shall be enclosed within transverse reinforcement conforming to 20.7.5.2 and 20.7.5.3.<<CB035>>

20.7.5 — **Transverse reinforcement**

20.7.5.1 — Transverse reinforcement required in 20.7.5.2 through 20.7.5.4 shall be provided over a length $\ell_o$ from each joint face and on both sides of any section where flexural yielding is likely to occur.
as a result of inelastic lateral displacements beyond the elastic range of behavior of the frame. Length $\ell_0$ shall not be less than the largest of (a), (b), and (c):

(a) The depth of the member column at the joint face or at the section where flexural yielding is likely to occur;
(b) One-sixth of the clear span of the member column; and
(c) 18 in.

**20.7.5.2** — Transverse reinforcement shall be provided by either single or overlapping spirals satisfying 7.10.4, circular hoops, or rectilinear hoops with or without crossties. Crossties of the same or smaller bar size as the hoops shall be permitted. Each end of the crosstie shall engage a peripheral longitudinal reinforcing bar. Consecutive crossties shall be alternated end for end along the longitudinal reinforcement. Spacing of crossties or legs of rectilinear hoops, $h_x$, within a cross section of the member column shall not exceed 14 in. on center. <<CH031>>

**20.7.5.2** — Transverse reinforcement shall be in accordance with (a) through (f):

(a) Transverse reinforcement shall comprise either single or overlapping spirals, circular hoops, or rectilinear hoops with or without crossties.
(b) Bends of rectilinear hoops and crossties shall engage peripheral longitudinal reinforcing bars.
(c) Crossties of the same or smaller bar size as the hoops shall be permitted, subject to the limitation of 21.8.2.2 <<former 7.10.5.1>>. Consecutive crossties shall be alternated end for end along the longitudinal reinforcement and around the perimeter of the cross section.
(d) Where rectilinear hoops or crossties are used, they shall provide lateral support to longitudinal reinforcement in accordance with 21.8.2.2 and 21.8.2.3.
(e) Spacing of crossties or legs of rectilinear hoops, $h_x$, within a cross section of the column shall not exceed 14 in. on center.
(f) Where $P_u > 0.3A_g f_c$ or $f_y > 10,000$ psi in columns with rectilinear hoops, every longitudinal bar or bundle of bars around the perimeter of the column core shall have lateral support provided by the corner of a hoop or by a seismic hook, and the maximum value of $h_x$ shall not exceed 8 in. $P_u$ shall be the largest value in compression consistent with factored load combinations including $E$. <<CH031>>

**20.7.5.3** — Spacing of transverse reinforcement along the length $\ell_0$ of the member shall not exceed the smallest of (a), (b), and (c):

(a) One-quarter of the minimum member column dimension;
(b) Six times the diameter of the smallest longitudinal bar; and
(c) $s_0$, as defined by Eq. (20.7.5.3)

$$s_0 = 4 + \left( \frac{14 - h_x}{3} \right)$$  \hspace{1cm} (20.7.5.3)

The value of $s_0$ from Eq. (20.7.5.3) shall not exceed 6 in. and need not be taken less than 4 in. <<At the beginning of 20.7.5.3 the reference to $\ell_0$ is removed to avoid confusion when other sections, notably on special structural walls, refer to this section. For columns, the fact that this refers to the length $\ell_0$ is clear from 20.7.5.1.>>

**20.7.5.4** — Amount of transverse reinforcement required in (a) or (b) shall be provided unless a larger amount is required by 20.7.6.
(a) The volumetric ratio of spiral or circular hoop reinforcement, $\rho_s$, shall not be less than required by Eq. (20.7.5.4a)

$$\rho_s = \frac{0.12 f'_c / f_{yt}}{s \rho c}$$

(20.7.5.4a)

and shall not be less than required by Eq. (10.5-14.6.3.1).

(b) The total cross-sectional area of rectangular hoop reinforcement, $A_{sh}$, shall not be less than required by Eq. (20.7.5.4b) and (20.7.5.4c)

$$A_{sh} = 0.3 \frac{sb c f'_c}{f_{yt}} \left[ \left( \frac{A_g}{A_{ch}} \right) - 1 \right]$$

(20.7.5.4b)

$$A_{sh} = 0.09 \frac{sb f'_c}{f_{yt}}$$

(20.7.5.4c) <<CH031>>

20.7.5.4 — Amount of transverse reinforcement shall be as required in Table 20.7.5.4.

Table 20.7.5.4 – Transverse reinforcement for columns of special moment frames <<Table entirely new, Underline not shown.>>

<table>
<thead>
<tr>
<th>Transverse reinforcement</th>
<th>Conditions</th>
<th>Applicable expressions</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\frac{A_{sh}}{sb c}$ for rectilinear hoop</td>
<td>$P_u \leq 0.3 A_g f_c$ and $f_c \leq 10,000$ psi</td>
<td>Greater of (a) and (b)</td>
</tr>
<tr>
<td>$P_u &gt; 0.3 A_g f_c$ or $f_c &gt; 10,000$ psi</td>
<td>Greater of (a), (b), and (c)</td>
<td></td>
</tr>
<tr>
<td>$0.3 \frac{A_g}{A_{ch}} - 1 \frac{f_c'}{f_{yt}}$ a</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$0.09 \frac{f_c'}{f_{yt}}$ b</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$0.2 k_f k_n \frac{P_u}{f_{yt} A_{ch}}$ c</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\rho_s$ for spiral or circular hoop</td>
<td>$P_u \leq 0.3 A_g f_c$ and $f_c \leq 10,000$ psi</td>
<td>Greater of (d) and (e)</td>
</tr>
<tr>
<td>$P_u &gt; 0.3 A_g f_c$ or $f_c &gt; 10,000$ psi</td>
<td>Greater of (d), (e), and (f)</td>
<td></td>
</tr>
<tr>
<td>$0.45 \frac{A_g}{A_{ch}} - 1 \frac{f_c'}{f_{yt}}$ d</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$0.12 \frac{f_c'}{f_{yt}}$ e</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$0.35 k_f \frac{P_u}{f_{yt} A_{ch}}$ f</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The concrete strength factor, $k_f$, and confinement effectiveness factor, $k_n$, are calculated according to Eqs. (20.7.5.4a) and (20.7.5.4b). <<both equations are new, but not shown underlined.>>

$$k_f = \frac{f_c'}{25,000} + 0.6 \geq 1.0$$

(20.7.5.4a)

$$k_n = \frac{n_l}{n_{l-2}}$$

(20.7.5.4b)
in which \( n_l = \) number of longitudinal bars or bar bundles around the perimeter of a column core with rectilinear hoops that are laterally supported by the corner of hoops or by seismic hooks.

20.7.5.5 — Beyond the length \( \ell_d \) specified in 20.7.5.1, the column shall contain spiral or hoop reinforcement satisfying 2.10, 14.2.4 and 14.7.4 with center-to-center spacing, \( s \), not exceeding the smaller of six times the diameter of the smallest longitudinal column bars and 6 in., unless a larger amount of transverse reinforcement is required by 20.7.4.3 or 20.7.6.

20.7.5.6 — Columns supporting reactions from discontinued stiff members, such as walls, shall satisfy (a) and (b):

(a) Transverse reinforcement as required in 20.7.5.2 through 20.7.5.4 shall be provided over their full height at all levels beneath the discontinuity if the factored axial compressive force in these members columns, related to earthquake effect, exceeds \( A_g f'_c / 10 \). Where design forces have been magnified to account for the overstrength of the vertical elements of the seismic-force-resisting system, the limit of \( A_g f'_c / 10 \) shall be increased to \( A_g f'_c / 4 \).

(b) The transverse reinforcement shall extend into the discontinued member at least \( \ell_d \) of the largest longitudinal column bar, where \( \ell_d \) is determined in accordance with 20.8.5. Where the lower end of the column terminates on a wall, the required transverse reinforcement shall extend into the wall at least \( \ell_d \) of the largest longitudinal column bar at the point of termination. Where the column terminates on a footing or mat, the required transverse reinforcement shall extend at least 12 in. into the footing or mat.

20.7.5.7 — If the concrete cover outside the confining transverse reinforcement specified in 20.7.5.5, 20.7.5.6 exceeds 4 in., additional transverse reinforcement having cover not exceeding 4 in. and spacing not exceeding 12 in. shall be provided. Concrete cover for additional transverse reinforcement shall not exceed 4 in. and spacing of additional transverse reinforcement shall not exceed 12 in.

20.7.6 — Shear strength requirements

20.7.6.1 — Design forces

The design shear force, \( V_e \), shall be determined from consideration of the maximum forces that can be generated at the faces of the joints at each end of the member column. These joint forces shall be determined using the maximum probable moment flexural strengths, \( M_{pr} \), at each end of the member column associated with the range of factored axial forces loads, \( P_u \), acting on the member column. The member column shears need not exceed those determined from joint strengths based on \( M_{pr} \) of the transverse member beams framing into the joint. In no case shall \( V_e \) be less than the factored shear determined by analysis of the structure.

20.7.6.2 — Transverse reinforcement

Transverse reinforcement over the lengths \( \ell_o \), identified in 20.7.5.1, shall be proportioned to resist shear assuming \( V_e = 0 \) when both (a) and (b) occur:

(a) The earthquake-induced shear force, calculated in accordance with 20.7.6.1, represents one-half or more of the maximum required shear strength within \( \ell_o \);
(b) The factored axial compressive force, $P_u$, including earthquake effects is less than $A_2 f'_c / 20$.

20.8 — Joints of special moment frames

20.8.1 — Scope

Requirements of 20.8 apply to beam-column joints of special moment frames forming part of the seismic-force-resisting system.

20.8.2 — General requirements

20.8.2.1 — Forces in longitudinal beam reinforcement at the joint face shall be determined by assuming that the stress in the flexural tensile reinforcement is $1.25 f_y$.

20.8.2.2 — Beam longitudinal reinforcement terminated in a column shall be extended to the far face of the confined column core and anchored shall be developed in tension according to in accordance with 20.8.5 and in compression according to Chapter 12 in accordance with 21.4.9.

20.8.2.3 — Where longitudinal beam reinforcement extends through a beam-column joint, the column dimension parallel to the beam reinforcement shall not be less than 20 times the diameter of the largest longitudinal beam bar for normalweight concrete or 26 times the diameter of the largest longitudinal bar for lightweight concrete. For lightweight concrete, the dimension shall be not less than 26 times the bar diameter.

20.8.3 — Transverse reinforcement

20.8.3.1 — Joint transverse reinforcement shall satisfy either 20.7.5.4(a) or 20.7.5.4(b), and shall also satisfy 20.7.5.2, 20.7.5.3, 20.7.5.4, and 20.7.5.7, except as permitted in 20.8.3.2.

20.8.3.2 — Where members beams frame into all four sides of the joint and where each member beam width is at least three-fourths the column width, the amount of reinforcement specified in 20.7.5.4(a) or 20.7.5.4(b) shall be permitted to be reduced by one-half, and the spacing required in 20.7.5.3 shall be permitted to be increased to 6 in. within the overall depth $h$ of the shallowest framing member beam.

20.8.3.3 — Longitudinal beam reinforcement outside the column core shall be confined by transverse reinforcement passing through the column that satisfies spacing requirements of 20.6.4.4, and requirements of 20.6.4.2 and 20.6.4.3, if such confinement is not provided by a beam framing into the joint.

20.8.3.4 — Where beam negative moment reinforcement is provided by headed deformed bars that terminate in the joint, the column shall extend above the top of the joint a distance at least the depth $h$ of the joint. Alternatively, the beam reinforcement shall be enclosed by additional vertical joint reinforcement providing equivalent confinement to the top face of the joint.

20.8.4 — Shear strength

20.8.4.1 — For normalweight concrete, $V_n$ of the joint shall not be taken as greater than the values specified below, in accordance with Table 20.8.4.1.

Table 20.8.4.1 — Nominal joint shear strength $V_n$

| Joint configuration | $V_n$ |
### Development length of bars in tension

#### 20.7.6.1
For bar sizes No. 3 through No. 11, the development length, \( \ell_{an} \), for a bar with a standard 90-degree hook in normal weight concrete shall not be less than the largest of \( 8d_b \) and 6 in., and the length required by Eq. (20.7.6.1)

\[
\ell_{an} = f_y d_b \left( \frac{65 \sqrt{f'_c}}{f'_c} \right) \tag{20.7.6.1}
\]

For lightweight concrete, \( \ell_{an} \), for a bar with a standard 90-degree hook shall not be less than the largest of \( 10d_b \) and 7-1/2 in., and 1.25 times the length required by Eq. (21.6.20.7.6.1).

#### 20.8.5.1
For bar sizes No. 3 through No. 11 terminating in a standard 90-degree hook, \( \ell_{dn} \), shall be determined from Eq. (20.8.5.1), but \( \ell_{dn} \), shall not be less than the larger of \( 8d_b \) and 6 in. for normal weight concrete nor less than the larger of \( 10d_b \) and 7-1/2 in. for lightweight concrete.

\[
\ell_{dn} = f_y d_b \left( \frac{65 \lambda \sqrt{f'_c}}{f'_c} \right) \tag{20.8.5.1}
\]

The value of \( \lambda \) shall be taken as 0.75 for lightweight and 1.0 for normal weight concrete.

### Effective cross-sectional area within a joint

#### 20.8.4.3
Effective cross-sectional area within a joint, \( A_j \), is the effective cross-sectional area within a joint shall be computed from joint depth times effective joint width. Joint depth shall be the overall depth of the column, \( h \). Effective joint width shall be the overall width of the column, except where a beam frames into a wider column, effective joint width shall not exceed the smaller of (a) and (b):

(a) Beam width plus joint depth

(b) Twice the smaller perpendicular distance from longitudinal axis of beam to column side.

#### 20.8.4.2
For lightweight concrete, the nominal shear strength of the joint shall not exceed three-quarters of the limits given in 20.8.4.1. **Note:** In LB10-4, one ballot comment recommended introducing lambda into the equations of 20.8.4 and deleting this section. This was discussed by Sub H at the Fall 2010 meeting and adopted. See table above.

### Notes and Definitions

1. See Table above.
2. For joints confined by beams on all four faces, \( A_j \) is defined in 20.8.4.3.
3. See 20.8.4.2.
4. \( \lambda \) shall be taken as 0.75 for lightweight concrete and 1.0 for normal weight concrete. **Note:** In LB10-4, one ballot comment recommended introducing lambda into the equations of 20.8.4 and deleting this section. This was discussed by Sub H at the Fall 2010 meeting and adopted. See table above.

### Table: Development Length of Bars in Tension

<table>
<thead>
<tr>
<th>Condition</th>
<th>Development Length</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bar sizes No. 3 through No. 11</td>
<td>( \ell_{an} = f_y d_b \left( \frac{65 \sqrt{f'_c}}{f'_c} \right) )</td>
</tr>
<tr>
<td>For lightweight concrete</td>
<td>( \ell_{dn} = f_y d_b \left( \frac{65 \lambda \sqrt{f'_c}}{f'_c} \right) )</td>
</tr>
</tbody>
</table>

Where:
- \( f_y \) is the yield strength of the reinforcing bar.
- \( d_b \) is the diameter of the reinforcing bar.
- \( f'_c \) is the specified strength of the concrete.
The 90-degree hook shall be located within the confined core of a column or of a boundary element with the hook bent into the joint. <<CH034>>

20.8.5.2 — For headed deformed bars satisfying 6.2.1.5, development in tension shall be in accordance with 21.4.4, except clear spacing between bars shall be permitted to be $3d_b$ or greater. <<CH034>>

20.8.5.3 — For bar sizes No. 3 through No. 11, $d_b$, the development length in tension for a straight bar, shall not be less than the larger greater of (a) and (b): <<CH034>>

(a) 2.5 times the length required by 20.8.5.1 if the depth of the concrete cast in one lift beneath the bar does not exceed 12 in.;

(b) 3.25 times the length required by 20.8.5.1 if the depth of the concrete cast in one lift beneath the bar exceeds 12 in.

20.8.5.4 — Straight bars terminated at a joint shall pass through the confined core of a column or of a boundary element. Any portion of $d_b$ not within the confined core shall be increased by a factor of 1.6. <<CH034>>

20.9 — Special moment frames constructed using precast concrete

20.9.1 — Scope

Requirements of 20.9 apply to special moment frames constructed using precast concrete forming part of the seismic-force-resisting system.

20.9.2 — Requirements

20.9.2.1 – Special moment frames with ductile connections constructed using precast concrete shall satisfy (a) and (b) and all requirements of 20.6 through 20.8 for special moment frames constructed with cast-in-place concrete:

(a) $V_n$ for connections computed according to 11.6.4 17.6.5 shall not be less than $2V_e$, where $V_e$ is calculated according to 20.6.5.1 or 20.7.6.1; <<Note that the upper bound limits are missing in the referenced code section.>>

(b) Mechanical splices of beam reinforcement shall be located not closer than $h/2$ from the joint face and shall meet the requirements of 20.2.7.

20.9.2.2 — Special moment frames with strong connections constructed using precast concrete shall satisfy all requirements of 20.6 through 20.8 for special moment frames constructed with cast-in-place concrete, as well as (a), (b), (c), and (d).

(a) Provisions of 20.6.2(a) shall apply to segments between locations where flexural yielding is intended to occur due to design displacements;

(b) Design strength of the strong connection, $f S_n$, shall be not less than $S_n$;

(c) Primary longitudinal reinforcement shall be made continuous across connections and shall be developed outside both the strong connection and the plastic hinge region; and
(d) For column-to-column connections, $\phi S_n$ shall not be less than 1.4$S_e$. At column-to-column connections, $\phi \frac{M}{N}$ shall not be less than 0.4$M_{pr}$ for the column within the story height, and $\phi V_n$ of the connection shall be not less than $V_e$ determined by 20.7.6.1.

20.9.2.3 — Special moment frames constructed using precast concrete and not satisfying the requirements of 20.9.2.1 or 20.9.2.2 shall satisfy the requirements of ACI 374.1 and the requirements of (a) and (b):

(a) Details and materials used in the test specimens shall be representative of those used in the structure; and

(b) The design procedure used to proportion the test specimens shall define the mechanism by which the frame resists gravity and earthquake effects, and shall establish acceptance values for sustaining that mechanism. Portions of the mechanism that deviate from Code requirements shall be contained in the test specimens and shall be tested to determine upper bounds for acceptance values.

20.10 — Special structural walls

20.10.1 — Scope

20.10.1.1 - Requirements of 20.10 apply to special structural walls and all components of special structural walls including coupling beams and wall piers forming part of the seismic-force-resisting system.

20.10.1.2 - Special structural walls constructed using precast concrete shall also comply with 20.11 in addition to 20.10.

20.10.2 — Reinforcement

20.10.2.1 — The distributed web reinforcement ratios, $\rho_i$ and $\rho_{iw}$, for structural walls shall not be less than 0.0025, except that if $V_u$ does not exceed $A_r \lambda \sqrt{F_p}$, $\rho_i$ and $\rho_{iw}$ shall be permitted to be reduced to the values required in 14.3 14.7.1.5. Reinforcement spacing each way in structural walls shall not exceed 18 in. Reinforcement contributing to $V_n$ shall be continuous and shall be distributed across the shear plane. <<The provision of 14.7.1.5 is found in Chapter 15, Walls. Check final disposition.>>

20.10.2.2 — At least two curtains of reinforcement shall be used in a wall if $V_u$ exceeds $V_u > 2A_r \lambda \sqrt{F_p}$ or $h_w / l_w > 2.0$, in which $h_w$ and $l_w$ refer to height and length of entire wall, respectively. <<CH030>>

20.10.2.3 — Reinforcement in structural walls shall be developed or spliced for $f_y$ in tension in accordance with Chapter 12, except 21.3, 21.4, and items (a) through (c):

(a) The effective depth of the member referenced in 12.10.3 shall be permitted to be taken as $0.8 \frac{d}{w}$ for walls. Longitudinal reinforcement shall extend beyond the point at which it is no longer required to resist flexure at least $0.8 \frac{d}{w}$, except at the top of a wall;

(b) The requirements of 12.11, 12.12, and 12.13 need not be satisfied.

(c) At locations where yielding of longitudinal reinforcement is likely to occur as a result of lateral displacements, development lengths of longitudinal reinforcement shall be 1.25 times the values calculated for $f_y$ in tension;
Mechanical splices of reinforcement shall conform to 20.2.7 and welded splices of reinforcement shall conform to 20.2.8.

20.10.3 — Design forces

$V_u$ shall be obtained from the lateral load analysis in accordance with the factored load combinations.

20.10.4 — Shear strength

20.10.4.1 — $V_n$ of structural walls shall not exceed

$$V_n = A_{cv} \left( \alpha_c \lambda \sqrt{f'_c} + \rho_t f_y \right)$$

(20.10.4.1)

where the coefficient $\alpha_c$ is 3.0 for $h_w / \ell_w \leq 1.5$, is 2.0 for $h_w / \ell_w \geq 2.0$, and varies linearly between 3.0 and 2.0 for $h_w / \ell_w$ between 1.5 and 2.0.

20.10.4.2 — In 20.10.4.1, the value of ratio $h_w / \ell_w$ used for determining $V_n$ for segments of a wall shall be the larger of the ratios for the entire wall and the segment of wall considered.

20.10.4.3 — Walls shall have distributed shear reinforcement providing resistance in two orthogonal directions in the plane of the wall. If $h_w / \ell_w$ does not exceed 2.0, reinforcement ratio $\rho_t$ shall not be less than reinforcement ratio $\rho_f$.

20.10.4.4 — For all vertical wall segments sharing a common lateral force, $V_n$ shall not be taken larger than $8A_{cv} \sqrt{f'_c}$, where $A_{cv}$ is the gross area of concrete bounded by web thickness and length of section. For any one of the individual vertical wall segments, $V_n$ shall not be taken larger than $10A_{cw} \sqrt{f'_c}$, where $A_{cw}$ is the area of concrete section of the individual vertical wall segment considered.

20.10.4.5 — For horizontal wall segments and coupling beams, $V_n$ shall not be taken larger than $10A_{cw} \sqrt{f'_c}$, where $A_{cw}$ is the area of concrete section of a horizontal wall segment or coupling beam.

20.10.5 — Design for flexure and axial loads

20.10.5.1 — Structural walls and portions of such walls subject to combined flexural and axial loads shall be designed in accordance with 9.4, 10.2 and 10.3, except that 10.2.7.8.4.1.4 and the nonlinear strain requirements of 10.2.2 shall not apply. Concrete and developed longitudinal reinforcement within effective flange widths, boundary elements, and the wall web shall be considered effective. The effects of openings shall be considered.

Note: The nonlinear strain requirements of 10.2.2 no longer appear in the referenced code sections.

20.10.5.2 — Unless a more detailed analysis is performed, effective flange widths of flanged sections shall extend from the face of the web a distance equal to the smaller of one-half the distance to an adjacent wall web and 25 percent of the total wall height.

20.10.6 — Boundary elements of special structural walls

20.10.6.1 — The need for special boundary elements at the edges of structural walls shall be evaluated in accordance with 20.10.6.2 or 20.10.6.3. The requirements of 20.10.6.4 and 20.10.6.5 shall also be satisfied.
20.10.6.2 — This section applies to walls or wall piers with $h_w / \ell_w \geq 2.0$ that are effectively continuous from the base of structure to top of wall and are designed to have a single critical section for flexure and axial loads. Walls not satisfying these requirements shall be designed by 20.10.6.3.

(a) Compression zones shall be reinforced with special boundary elements where

$$c \geq \frac{\ell_w}{600 (1.5 \delta_u / h_w)} \tag{20.10.6.2}$$

$c$ in Eq. (20.10.6.2) corresponds to the largest neutral axis depth calculated for the factored axial force and nominal moment strength consistent with the direction of the design displacement $\delta_u$. Ratio $\delta_u / h_w$ in Eq. (20.10.6.2) shall not be taken less than 0.007/0.005.

(b) Where special boundary elements are required by 20.10.6.2(a), the special boundary element transverse reinforcement shall extend vertically above and below from the critical section a distance not less than the larger of $\ell_w$ or $M_u / 4V_u$ except as permitted in 20.10.6.4(f).<ref><CH030></ref>

20.10.6.3 — Structural walls not designed to the provisions of 20.10.6.2 shall have special boundary elements at boundaries and edges around openings of structural walls where the maximum extreme fiber compressive stress, corresponding to load combinations including earthquake effects, $E$, exceeds $0.2f'_c$. The special boundary element shall be permitted to be discontinued where the calculated compressive stress is less than $0.15f'_c$. Stresses shall be calculated for the factored loads using a linearly elastic model and gross section properties. For walls with flanges, an effective flange width as defined in 20.10.5.2 shall be used.

20.10.6.4 — Where special boundary elements are required by 20.10.6.2 or 20.10.6.3, (a) through (e) shall be satisfied:

(a) The boundary element shall extend horizontally from the extreme compression fiber a distance not less than the larger of $c - 0.1\ell_w$ and $c / 2$, where $c$ is the largest neutral axis depth calculated for the factored axial force and nominal moment strength consistent with $\delta_u$.

(b) Width of flexural compression zone $b$ over the horizontal distance determined in 20.10.6.4(a), including flange if present, shall be greater than or equal to $h_w / 16$.

(c) For walls or wall piers with $h_w / \ell_w \geq 2.0$ that are effectively continuous from the base of structure to top of wall, designed to have a single critical section for flexure and axial loads, and with $c / \ell_w \geq 3 / 8$, width of the flexural compression zone $b$ over the length determined in 20.10.6.4(a) shall be greater than or equal to 12 inches.

(d) In flanged sections, the boundary element shall include the effective flange width in compression and shall extend at least 12 in. into the web;

(e) The boundary element transverse reinforcement shall satisfy the requirements of 20.7.5.2 (a) through (e) and 20.7.5.3 through 20.7.5.4, except Eq. (20.7.5.4b) need not be satisfied the value $h_x$ in 20.7.5.2 shall not exceed the lesser of 14 in. and two-thirds of the boundary element thickness, and the transverse reinforcement spacing limit of 20.7.5.3(a) shall be one-third of the least dimension of the boundary element; <ref><CH031></ref>

(f) Amount of transverse reinforcement shall be as required in Table 20.10.6.4(f).<ref><CH031></ref>
Table 20.10.6.4(f) – Transverse reinforcement for special boundary elements

<table>
<thead>
<tr>
<th>Transverse reinforcement</th>
<th>Applicable expressions</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\frac{A_{sh}}{h_{bc}}$ for rectilinear hoop</td>
<td>Greater of (a) and (b)</td>
</tr>
<tr>
<td>$\rho_s$ for spiral or circular hoop</td>
<td>Greater of (c) and (d)</td>
</tr>
</tbody>
</table>

(a) If the longitudinal reinforcement ratio at the wall boundary is greater than $400/f_y$, boundary transverse reinforcement shall satisfy 20.7.5.2 over the distance determined in accordance with and 20.10.6.4(a). The maximum longitudinal spacing of transverse reinforcement in at the wall boundary shall not exceed the lesser of 8 in. and $8d_s$ of the smallest primary flexural reinforcing bars, except the spacing shall not exceed the lesser of 6 in. and $6d_s$ within a distance equal to the greater of $\rho_{yw}$ and $M_u / 4V_y$ above and below critical sections where yielding of longitudinal reinforcement is likely to occur as a result of inelastic lateral displacements; <<CH030, with minor post-ballot editing>>

(b) Except when $V_u$ in the plane of the wall is less than $A_{cv}\sqrt{T_c}$, horizontal reinforcement terminating at the edges of structural walls without boundary elements shall...
have a standard hook engaging the edge reinforcement or the edge reinforcement shall be
enclosed in U-stirrups having the same size and spacing as, and spliced to, the horizontal
reinforcement.

20.10.7 — Coupling beams

20.10.7.1 — Coupling beams with \((\frac{f_n}{h}) \geq 4\) shall satisfy the requirements of 20.6, with the wall
boundary interpreted as being a column. The provisions of 20.6.2(b) and 20.6.2(c) need not be satisfied
if it can be shown by analysis that the beam has adequate lateral stability. <<The added text will be
clarified in commentary.>>

20.10.7.2 — Coupling beams with \((\frac{f_n}{h}) < 2\) and with \(V_u\) exceeding \(4A_{cw}\) shall be
reinforced with two intersecting groups of diagonally placed bars symmetrical about the midspan,
unless it can be shown that loss of stiffness and strength of the coupling beams will not impair the
vertical load-carrying ability of the structure, the egress from the structure, or the integrity of
nonstructural components and their connections to the structure.

20.10.7.3 — Coupling beams not governed by 20.10.7.1 or 20.10.7.2 shall be permitted to be
reinforced either with two intersecting groups of diagonally placed bars symmetrical about the midspan
or according to 20.6.3 through 20.6.5, with the wall boundary interpreted as being a column.

20.10.7.4 — Coupling beams reinforced with two intersecting groups of diagonally placed bars
symmetrical about the midspan shall satisfy (a), (b), and either (c) or (d). Requirements of 11.7 17.3
shall not apply.

(a) \(V_n\) shall be determined by

\[
V_n = 2A_{yd}f_y\sin\alpha \leq 10\sqrt{f_y' A_{cw}} \tag{20.10.7.2}
\]

where \(\alpha\) is the angle between the diagonal bars and the longitudinal axis of the coupling
beam.

(b) Each group of diagonal bars shall consist of a minimum of four bars provided in two or
more layers. The diagonal bars shall be embedded into the wall not less than 1.25 times
the development length for \(f_y\) in tension.

(c) Each group of diagonal bars shall be enclosed by rectilinear transverse reinforcement having out-to-out
dimensions not smaller than \(\frac{b_w}{2}\) in the direction parallel to \(b_w\) and \(\frac{b_w}{5}\) along the other sides, where
\(b_w\) is the web width of the coupling beam. The transverse reinforcement shall satisfy be in accordance
with 20.7.5.2 (a) through (e) and 20.7.5.4, with \(A_{sh}\) not less than the greater of (i) and (ii)

\[
(i) \quad 0.09sb_w \frac{f_y'}{f_{yx}}
\]

\[
(ii) \quad 0.3sb_w \frac{A_g}{A_{ch}} - 1 \frac{f_y'}{f_{yx}}. \tag{CH031}
\]

For the purpose of computing calculating \(A_g\) for use in Eq. (14.6.3.1) and (20.7.5.4b), the
cement cover as required in 6.10.5 shall be assumed on all four sides of each group of
diagonal bars. The transverse reinforcement shall have spacing measured parallel to the
diagonal bars satisfying 20.7.5.3(c) and not exceeding six times the diameter \(6d_d\) of the
diagonal bars, and shall have spacing of crossties or legs of hoops measured
perpendicular to the diagonal bars not exceeding 14 in. For the purpose of computing \( A_{sf} \) for use in Eq. (10-5), (14.6.3.1) and (20.7.5.4b), the concrete cover as required in 7.7.6.10.5 shall be assumed on all four sides of each group of diagonal bars. The transverse reinforcement, or its alternatively configured transverse reinforcement satisfying the spacing and volume ratio requirements of the transverse reinforcement along the diagonals, shall continue through the intersection of the diagonal bars. Additional longitudinal and transverse reinforcement shall be distributed around the beam perimeter with total area in each direction not less than 0.002\( b_w \)s and spacing not exceeding 12 in.

(d) Transverse reinforcement shall be provided for the entire beam cross section satisfying in accordance with 20.7.5.2 (a) through (e), 20.7.5.4, and 20.7.5.7, with \( A_{sh} \) not less than the greater of (i) and (ii):

\[
\begin{align*}
(i) & \quad 0.09 s b_c \frac{f'_c}{f_{ys}} \quad \text{and} \\
(ii) & \quad 0.3 s b_c \frac{A_{sf}}{A_{sh}} - 1 \frac{f'_c}{f_{ys}}, \quad 20.7.5.4, \text{and} \ 20.7.5.7
\end{align*}
\]

Longitudinal spacing of transverse reinforcement shall not exceed the smaller of 6 in. and six times the diameter \( d_b \) of the diagonal bars, and with spacing of crossties or legs of hoops both vertically and horizontally in the plane of the beam cross section shall not exceeding 8 in. Each crosstie and each hoop leg shall engage a longitudinal bar of equal or larger diameter. It shall be permitted to configure hoops as specified in 20.6.4.3.

**20.10.8 — Wall piers**

**20.10.8.1** — Wall piers shall satisfy the special moment frame requirements for columns of 20.7.4, 20.7.5, and 20.7.6, with joint faces taken as the top and bottom of the clear height of the wall pier. Alternatively, wall piers with \( \frac{\ell_{w}}{b_w} > 2.5 \) shall satisfy (a) through (f):

(a) Design shear force shall be determined in accordance with 20.7.6.1 with joint faces taken as the top and bottom of the clear height of the wall pier. Where the legally adopted general building code **General Building Code** includes provisions to account for overstrength of the seismic-force-resisting system, the design shear force need not exceed \( \Omega_s \) times the factored shear determined by analysis of the structure for earthquake load effects.

(b) \( V_s \) and distributed shear reinforcement shall satisfy 20.10.4.

(c) Transverse reinforcement shall be in the form of hoops except it shall be permitted to use single-leg horizontal reinforcement parallel to \( \ell_w \) where only one curtain of distributed shear reinforcement is provided. Single-leg horizontal reinforcement shall have 180-degree bends at each end that engage wall pier boundary longitudinal reinforcement.

(d) Vertical spacing of transverse reinforcement shall not exceed 6 in.

(e) Transverse reinforcement shall extend at least 12 in. above and below the clear height of the wall pier.

(f) Special boundary elements shall be provided if required by 20.10.6.3.

**20.10.8.2** — For wall piers at the edge of a wall, horizontal reinforcement shall be provided in adjacent wall segments above and below the wall pier and be proportioned to transfer the design shear force from the wall pier into the adjacent wall segments.

**20.10.9 — Construction joints**
All construction joints in structural walls shall conform to 6.4 24.3.18, and contact surfaces shall be roughened as in 11.6.9 17.6.1.5 and 17.6.1.6. <<24.3.18 is found in Chapter 23, Construction and Formwork. Planned combination of Chapters 23 and 24 may affect final numbering.>>

20.10.10 — Discontinuous walls
Columns supporting discontinuous structural walls shall be reinforced in accordance with 20.7.5.6.

20.11 — Special structural walls constructed using precast concrete

20.11.1 — Scope
Requirements of 20.11 apply to special structural walls constructed using precast concrete forming part of the seismic-force-resisting system.

20.11.2 — Requirements
20.11.2.1 — Special structural walls constructed using precast concrete shall satisfy all requirements of 20.10 in addition to 20.5.2.
20.11.2.2 — Special structural walls constructed using precast concrete and unbonded post-tensioning tendons and not satisfying the requirements of 20.11.2.1 are permitted provided they satisfy the requirements of ACI ITG 5.1.

20.12 — Structural diaphragms and trusses

20.12.1 — Scope
20.12.1.1 - Requirements of 20.12 apply to floor and roof slabs acting as structural diaphragms and collectors forming part of the seismic-force-resisting system to transmit forces induced by earthquake ground motions in structures assigned to SDC D, E, or F. shall be designed in accordance with this section.
20.12.1.2 — Requirements of 20.12 apply to structural trusses forming part of the seismic-force-resisting system in SDC D, E, or F. This section also applies to collector elements and trusses forming part of the seismic-force-resisting system.

20.12.2 — Design forces
The earthquake design forces for structural diaphragms shall be obtained from the General Building Code legally adopted general building code using the applicable provisions and load combinations.

20.12.3 — Seismic load path
20.12.3.1 - All diaphragms and their connections shall be proportioned and detailed to provide for a complete transfer of forces to collector elements and to the vertical elements of the seismic-force-resisting system.
20.12.3.2 — Elements of a structural diaphragm system that are subjected primarily to axial forces and used to transfer diaphragm shear or flexural forces around openings or other discontinuities, shall comply with the requirements for collectors in 20.12.7.5 and 20.12.7.6.

20.12.4 — Cast-in-place composite-topping slab diaphragms
A cast-in-place composite-topping slab cast in place on a precast floor or roof shall be permitted to be used as a structural diaphragm, provided the cast-in-place topping slab is reinforced and the surface of the previously hardened concrete on which the topping slab is placed is clean, free of laitance, and intentionally roughened. <<wording changed to match next section>>
20.12.5 — Cast-in-place noncomposite topping slab diaphragms

A cast-in-place noncomposite topping on a precast floor or roof shall be permitted to serve as a structural diaphragm, provided the cast-in-place topping slab acting alone is proportioned and detailed to resist the design earthquake forces.

20.12.6 — Minimum thickness of diaphragms

Concrete slabs and composite topping slabs serving as structural diaphragms used to transmit earthquake forces shall not be less than 2 in. thick. Topping slabs placed over precast floor or roof elements, acting as structural diaphragms and not relying on composite action with the precast elements to resist the design earthquake forces, shall have thickness not be less than 2-1/2 in. thick.

20.12.7 — Reinforcement

20.12.7.1 — The minimum reinforcement ratio for structural diaphragms shall be in conformance with 7.12 10.5. Except for post-tensioned slabs, reinforcement spacing each way in floor or roof systems shall not exceed 18 in. Where welded wire reinforcement is used as the distributed reinforcement to resist shear in topping slabs placed over precast floor and roof elements, the wires parallel to the joints between span of the precast elements shall be spaced not less than 10 in. on centers. Reinforcement provided for shear strength shall be continuous and shall be distributed uniformly across the shear plane.

20.12.7.2 — Bonded tendons used as reinforcement to resist collector forces, or diaphragm shear, or flexural tension shall be proportioned such that the stress due to design earthquake forces does not exceed 60,000 psi. Precompression from unbonded tendons shall be permitted to resist diaphragm design forces if a seismic load path is provided.

20.12.7.3 — All reinforcement used to resist collector forces, diaphragm shear, or flexural tension shall be developed or spliced for $f_y$ in tension.

20.12.7.4 — Type 2 splices are required where mechanical splices are used to transfer forces between the diaphragm and the vertical elements of the seismic-force-resisting system.

20.12.7.5 — Collector elements with compressive stresses exceeding $0.2f'_{ce}$ at any section shall have transverse reinforcement satisfying 20.10.6.4(c) over the length of the element. The specified transverse reinforcement is permitted to be discontinued at a section where the calculated compressive stress is less than $0.15f'_{ce}$.

Where design forces have been amplified to account for the overstrength of the vertical elements of the seismic-force-resisting system, the limit of $0.2f'_{ce}$ shall be increased to $0.5f'_{ce}$, and the limit of $0.15f'_{ce}$ shall be increased to $0.4f'_{ce}$.

20.12.7.6 — Longitudinal reinforcement for collector elements at splices and anchorage zones shall have either:

(a) A minimum center-to-center spacing of three longitudinal bar diameters, but not less than 1-1/2 in., and a minimum concrete clear cover of two and one-half longitudinal bar diameters, but not less than 2 in.; or

(b) Transverse reinforcement as required by 11.4.6.3, providing $A_y$, not less than the greater of $0.75\sqrt{\frac{f'_{ce}}{f_{yt}}} \frac{b_w}{s}$ and $(50b_w s)/f_{yt}$, except as required in 20.12.7.5. <<text introduced to Chapter 21 to avoid referencing the beam chapter from the diaphragm section.>>

20.12.8 — Flexural strength
Diaphragms and portions of diaphragms shall be designed for flexure in accordance with 10.2 and 10.3.9.3 except that the nonlinear distribution of strain requirements of 10.2.2 for deep beams need not apply. The effects of openings shall be considered.

<<Note: The nonlinear strain requirements of 10.2.2 no longer appear in the referenced code sections.>>.

<<Note: Referenced sections 9.2, 9.3, and 9.4 need to be checked again after Chapter 9 settles down to be certain only relevant requirements are referenced.>>.

20.12.9 — Shear strength

20.12.9.1 — \( V_n \) of structural diaphragms shall not exceed

\[ V_n = A_{sv} \left( 2 \lambda \sqrt{f_c'} + \rho_f f_y \right) \]  

(20.12.9.1)

For cast-in-place topping slab diaphragms on precast floor or roof members, \( A_{cv} \) shall be computed using only the thickness of topping slab for noncomposite topping slab diaphragms and the combined thickness of cast-in-place and precast elements for composite topping slab diaphragms. For composite topping slab diaphragms, the value of \( f_c' \) used to determine \( V_n \) shall not exceed the smaller of \( f_c' \) for the precast members and \( f_c' \) for the topping slab.

20.12.9.2 — \( V_n \) of structural diaphragms shall not exceed \( 8A_{sv} \sqrt{f_c'} \).

20.12.9.3 — Above joints between precast elements in noncomposite and composite cast-in-place topping slab diaphragms, \( V_n \) shall not exceed

\[ V_n = A_{sv} f_y \mu \]  

(20.12.9.3)

where \( A_{sv} \) is the total area of shear friction reinforcement within the topping slab, including both distributed and boundary reinforcement, that is oriented perpendicular to joints in the precast system and coefficient of friction, \( \mu \), is 1.0. \( \lambda \) is given in 11.6.4.3.5.2.4.1. At least one-half of \( A_{sv} \) shall be uniformly distributed along the length of the potential shear plane. The area of distributed reinforcement in the topping slab shall satisfy 7.12.2.1 10.5.4.2 in each direction.

20.12.9.4 — Above joints between precast elements in noncomposite and composite cast-in-place topping slab diaphragms, \( V_n \) shall not exceed the limits in 11.6.6 17.6.5.3 where \( A_c \) is computed using only the thickness of the topping slab only.

20.12.10 — Construction joints

All construction joints in diaphragms shall conform to 6.4 24.3.18, and contact surfaces shall be roughened as in 14.6.9 17.6.1.5 and 17.6.1.6. <<24.3.18 will move to chapter 23 when chapters 23 and 24 are combined. Needs to be tracked.>>

20.12.11 — Structural trusses

20.12.11.1 — Structural truss elements with compressive stresses exceeding 0.2\( f_c' \) at any section shall have transverse reinforcement, as given in accordance with 20.7.5.2, 2.7.5.3, through 20.7.5.4 and 20.7.5.7, and Table 20.12.11.1, over the length of the element.
Table 20.12.11.1 – Transverse reinforcement for structural trusses

<table>
<thead>
<tr>
<th>Transverse reinforcement</th>
<th>Applicable expressions</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \frac{\rho_s}{\rho_d} ) for rectilinear hoop</td>
<td>Greater of (a) and (b)</td>
</tr>
<tr>
<td>( 0.3 \frac{A_g}{A_{ch}} - 1 \frac{f_c}{f_{yt}} ) ( a )</td>
<td></td>
</tr>
<tr>
<td>( 0.09 \frac{f_c}{f_{yt}} ) ( b )</td>
<td></td>
</tr>
<tr>
<td>( \rho_s ) for spiral or circular hoop</td>
<td>Greater of (c) and (d)</td>
</tr>
<tr>
<td>( 0.45 \frac{A_g}{A_{ch}} - 1 \frac{f_c}{f_{yt}} ) ( c )</td>
<td></td>
</tr>
<tr>
<td>( 0.12 \frac{f_c}{f_{yt}} ) ( d )</td>
<td></td>
</tr>
</tbody>
</table>

<<CH031>>

20.12.11.2 — All continuous reinforcement in structural truss elements shall be developed or spliced for \( f_y \) in tension.

20.13 — Foundations

20.13.1 — Scope

20.13.1.1 — Foundations resisting earthquake-induced forces or transferring earthquake-induced forces between structure and ground in structures assigned to SDC D, E, or F shall comply with 20.13 and other applicable Code provisions.

20.13.1.2 — The provisions in this section for piles, drilled piers, caissons, and slabs-on-ground shall supplement other applicable Code design and construction criteria. See 1.1.8. <<Old 1.1.6 was not located.>>

20.13.2 — Footings, foundation mats, and pile caps

20.13.2.1 — Longitudinal reinforcement of columns and structural walls resisting forces induced by earthquake effects shall extend into the footing, mat, or pile cap, and shall be fully developed for tension at the interface.

20.13.2.2 — Columns designed assuming fixed-end conditions at the foundation shall comply with 20.13.2.1 and, if hooks are required, longitudinal reinforcement resisting flexure shall have 90-degree hooks near the bottom of the foundation with the free end of the bars oriented toward the center of the column.

20.13.2.3 — Columns or boundary elements of special structural walls that have an edge within one-half the footing depth from an edge of the footing shall have transverse reinforcement in accordance with 20.7.5.2 through 20.7.5.4 provided below the top of the footing. This reinforcement shall extend into the footing, mat, or pile cap and be developed a length equal to the development length, calculated for \( f_y \) in tension, of the column or boundary element longitudinal reinforcement.

20.13.2.4 — Where earthquake effects create uplift forces in boundary elements of special structural walls or columns, flexural reinforcement shall be provided in the top of the footing, mat or pile cap to resist actions resulting from the design factored load combinations, and shall not be less than
required by 10.5 11.6.2 and 14.6 15.6.2 or 15.6.5. <<Track this when the foundations chapter is rewritten. 10.5 may become 13.6.2.1.>>

20.13.2.5 — Structural plain concrete in footings and basement walls shall be in accordance with Chapter 27 has 26.1.3 as the section number.>>

20.13.3 — Grade beams and slabs-on-ground

20.13.3.1 — Grade beams designed to act as horizontal ties between pile caps or footings shall have continuous longitudinal reinforcement that shall be developed within or beyond the supported column or anchored within the pile cap or footing at all discontinuities.

20.13.3.2 — Grade beams designed to act as horizontal ties between pile caps or footings shall be proportioned such that the smallest cross-sectional dimension shall be equal to or greater than the clear spacing between connected columns divided by 20, but need not be greater than 18 in. Closed ties shall be provided at a spacing not to exceed the lesser of one-half the smallest orthogonal cross-sectional dimension and 12 in.

20.13.3.3 — Grade beams and beams that are part of a mat foundation subjected to flexure from columns that are part of the seismic-force-resisting system shall conform to 20.6.

20.13.3.4 — Slabs-on-ground that resist seismic forces from walls or columns that are part of the seismic-force-resisting system shall be designed as structural diaphragms in accordance with 20.12. The design drawings contract documents shall clearly state indicate that the slab-on-ground is a structural diaphragm and part of the seismic-force-resisting system.

20.13.4 — Piles, piers, and caissons

20.13.4.1 — Provisions of 20.13.4 shall apply to concrete piles, piers, and caissons supporting structures designed for earthquake resistance.

20.13.4.2 — Piles, piers, or caissons resisting tension loads shall have continuous longitudinal reinforcement over the length resisting design tension forces. The longitudinal reinforcement shall be detailed to transfer tension forces within the pile cap to supported structural members.

20.13.4.3 — Where tension forces induced by earthquake effects are transferred between pile cap or mat foundation and precast pile by reinforcing bars grouted or post-installed in the top of the pile, the grouting system shall have been demonstrated by test to develop at least 1.25fy of the bar.

20.13.4.4 — Piles, piers, or caissons shall have transverse reinforcement in accordance with 20.7.5.2 through 20.7.5.4 at locations (a) and (b):

(a) At the top of the member for at least 5 times the member cross-sectional dimension, but not less than 6 ft below the bottom of the pile cap;

(b) For the portion of piles in soil that is not capable of providing lateral support, or in air and water, along the entire unsupported length plus the length required in 20.13.4.3(a).

20.13.4.5 — For precast concrete driven piles, the length of transverse reinforcement provided shall be sufficient to account for potential variations in the elevation of pile tips.

20.13.4.6 — Pile caps incorporating batter piles shall be designed to resist the full compressive strength of the batter piles acting as short columns. The slenderness effects of batter piles shall be considered for the portion of the piles in soil that is not capable of providing lateral support, or in air or water.
20.14 — Members not designated as part of the seismic-force-resisting system

<<This section was reorganized in response to LB11-02b.>>

20.14.1 — Scope

Requirements of 20.14 apply to members not designated as part of the seismic-force-resisting system in structures assigned to SDC D, E, and F.

20.14.2 — Design Actions

Members not designated as part of the seismic-force-resisting system shall be evaluated for gravity load combinations of \((1.2D + 1.0L + 0.2S)\) or \(0.9D\), whichever is critical, acting simultaneously with the design displacement \(\delta_u\). The load factor on the live load, \(L\), shall be permitted to be reduced to 0.5 except for garages, areas occupied as places of public assembly, and all areas where \(L\) is greater than 100 lb/ ft².

20.14.3 - Cast-in-Place Beams and Columns

Frame members Cast-in-place beams and columns assumed not to contribute to lateral resistance, except two-way slabs without beams and wall piers, shall be detailed according to 20.14.3.1 or 20.14.3.2 depending on the magnitude of moments and shears induced in those members when subjected to the design displacement \(\delta_u\). If effects of \(\delta_u\) are not explicitly checked, it shall be permitted to apply the requirements of 20.14.3.2.

20.14.3.1 — Where the induced moments and shears under design displacements, \(\delta_u\), combined with the factored gravity moments and shears do not exceed the design moment and shear strength of the frame member, the conditions of 20.14.3.1 (a) through (c) shall be satisfied. The gravity load combinations of \((1.2D + 1.0L + 0.2S)\) or \(0.9D\), whichever is critical, shall be used. The load factor on the live load, \(L\), shall be permitted to be reduced to 0.5 except for garages, areas occupied as places of public assembly, and all areas where \(L\) is greater than 100 lb/ ft².

(a) Members with factored gravity axial forces not exceeding \(\frac{A_g f_c'}{10}\). Beams shall satisfy 20.6.3.1. Stirrups shall be spaced not more than \(d/2\) throughout the length of the member. Spacing of transverse reinforcement shall not exceed \(d/2\). Where factored axial force exceeds \(\frac{A_g f_c'}{10}\), transverse reinforcement shall be hoops satisfying 20.7.5.2 at spacing \(s_o\) according to 20.14.3.1(b).

(b) Members with factored gravity axial forces exceeding \(\frac{A_g f_c'}{10}\). Columns shall satisfy 20.7.4.1, 20.7.5.2, and 20.7.6. The maximum longitudinal spacing of ties hoops shall be \(s_o\) for the full member column length. Spacing \(s_o\) shall not exceed the smaller of six diameters of the smallest longitudinal bar enclosed and 6 in.

(c) Members Columns with factored gravity axial forces exceeding \(0.35P_O\) shall satisfy 20.14.3.1(b) and 20.7.5.7. The amount of transverse reinforcement provided shall be one-half of that required by 20.7.5.4 but shall not be spaced greater than \(s_o\) for the full member column length.

20.14.3.2 — If the induced moments or shears under design displacements, \(\delta_u\), exceed \(\phi M_n\) or \(\phi V_n\) of the frame member, or if induced moments or shears are not calculated, the conditions of 20.14.3.2 (a) through (c) shall be satisfied.
(a) — Materials, mechanical splices, and welded splices shall satisfy the requirements for special moment frames in 20.2.5 through 20.2.8, 20.1.4.2, 20.1.4.3, 20.1.5.2, 20.1.5.4, and 20.1.5.5. Mechanical splices shall satisfy 20.1.6 and welded splices shall satisfy 20.1.7.1.

(b) — Members with factored gravity axial forces not exceeding $A_g f_c'/10$. Beams shall satisfy 20.14.3.1(a) and 20.6.5. Stirrups shall be spaced at not more than $d'/2$ throughout the length of the member.

(c) — Members with factored gravity axial forces exceeding $A_g f_c'/10$. Columns shall satisfy 20.7.4, 20.7.5, 20.7.6, and 20.8.3.1.

20.14.4 – Precast Beams and Columns

Precast concrete frame members assumed not to contribute to lateral resistance, including their connections, shall satisfy (a), (b), and (c), in addition to 20.14.2 through 20.14.4 20.14.3:

(a) Ties specified in 20.14.3.2 20.14.3.1(b) shall be provided over the entire column height, including the depth of the beams;

(b) Structural integrity reinforcement, as specified in 16.5.4.10, shall be provided; and

(c) Bearing length at support of a beam shall be at least 2 in. longer than determined from calculations using bearing strength values from 10.14.17.5.4.2.

20.14.5 – Slab-Column Connections

For slab-column connections of two-way slabs without beams, slab shear reinforcement satisfying the requirements of 11.11.3 and 11.11.5 12.7.6 or 12.7.7 and providing $V_{ug} V_{c}$ not less than $3.5 \sqrt{f_c'} b_r d - 3.5 \sqrt{f_c'}$, shall extend at least four times the slab thickness from the face of the support if the design story drift ratio exceeds the greater of (a) and (b):

(a) 0.005

(b) $0.035 - 0.05 \frac{V_{ug}}{\phi V_c}$

unless either (a) or (b) is satisfied.

(a) The design shear stress $\phi V_c$ is not less than the shear stress $V_c$ calculated according to 12.4.6 considering the requirements of 11.11.7 using the design shear $V_{ug}$ and the induced moment transferred between the slab and column under the design displacement;

(b) The design story drift ratio does not exceed the larger of 0.005 and $0.035 - 0.05 \left( \frac{V_{ug}}{V_c} - 1 \right)$

Design story drift ratio shall be taken as the larger of the design story drift ratios of the adjacent stories above and below the slab-column connection. $V_{ug} V_c$ is defined in 11.11.2 9.6.8. $V_{ug} V_{ug}$ is the factored shear force on the slab critical section for two-way action due to gravity loads, calculated for the load combination $1.2D + 1.0L + 0.2S$.

The load factor on the live load, $L$, shall be permitted to be reduced to 0.5 except for garages, areas occupied as places of public assembly, and all areas where $L$ is greater than 100 lb/ft².

20.14.5 20.14.6 – Wall Piers
Wall piers not designated as part of the seismic-force-resisting system shall satisfy the requirements of 20.10.8. Where the legally adopted general building code General Building Code includes provisions to account for overstrength of the seismic-force-resisting system, it shall be permitted to determine the design shear force as $\Omega \delta_u$ times the shear induced under design displacements, $\delta_u$. 
Commentary References


R20.1 — General requirements Scope

R20.1.1 — Scope

Chapter 20 does not apply to structures assigned to Seismic Design Category A. For structures assigned to Seismic Design Categories B and C, Chapter 20 applies to structural systems designated as part of the seismic-force-resisting system. For structures assigned to Seismic Design Categories D, E, and F, this includes diaphragms and foundations. Furthermore, in structures assigned to Seismic Design Categories D, E, and F, Chapter 20 also applies to structural systems not designated as part of the seismic-force-resisting system.

Chapter 20 contains provisions considered to be the minimum requirements for a cast-in-place or precast concrete structure capable of sustaining a series of oscillations into the inelastic range of response without critical deterioration in strength. The integrity of the structure in the inelastic range of response should be maintained because the design earthquake forces defined in documents such as ASCE/SEI 7,20.1 the IBC,20.2 the UBC,20.3 and the NEHRP20.4 provisions are considered less than those corresponding to linear response at the anticipated earthquake intensity.20.4-20.7

The design philosophy in Chapter 20 is for cast-in-place concrete structures to respond in the nonlinear range when subjected to design-level ground motions, with decreased stiffness and increased energy dissipation but without critical strength decay. Precast concrete structures designed in accordance with Chapter 20 are intended to emulate cast-in-place construction, except 20.5, 20.9.2.3, and 20.11.3 permit precast construction with alternative yielding.
mechanisms permits special precast moment frames designed in accordance with ACI 374.1. As a properly detailed cast-in-place or precast concrete structure responds to strong-ground motion, its effective stiffness decreases and its energy dissipation increases. These changes tend to reduce the response accelerations and lateral inertia forces relative to values that would occur were the structure to remain linearly elastic and lightly damped. Thus, the use of design forces representing earthquake effects such as those in ASCE/SEI 7 requires that the seismic-force-resisting system retain a substantial portion of its strength into the inelastic range under displacement reversals.

The provisions of Chapter 21-20 relate detailing requirements to type of structural framing and seismic design category (SDC). SDCs are adopted directly from ASCE/SEI 7, and relate to considerations of seismic hazard level, soil type, occupancy, and use. Before the 2008 Code, low, intermediate, and high seismic risk designations were used to delineate detailing requirements. For a qualitative comparison of SDCs and seismic risk designations, see Table R1.1.9.1-R4.4.6.1. The assignment of a structure to a SDC is regulated by the general building code legally adopted general building code of which this Code forms a part (see 1.1.9.4.4.6.1).

R20.2 — General

The intent of this Code is that all structures and members satisfy all applicable provisions of this Code. Structures assigned to Seismic Design Category A need not satisfy requirements of Chapter 20 but must satisfy all applicable requirements of this Code. Structures assigned to Seismic Design Categories B through F must satisfy requirements of Chapter 20 in addition to all other applicable requirements of this Code.

The design and detailing requirements should be compatible with the level of energy dissipation (or toughness) inelastic response assumed in the computation of the design earthquake forces. The terms “ordinary,” “intermediate,” and “special” are specifically used to facilitate this compatibility. For any given structural element or system, the terms “ordinary,” “intermediate,” and “special” refer to increasing requirements for detailing and proportioning, with expectations of increased deformation capacity. Structural elements or systems identified as “ordinary” may have limited inelastic deformation capability, and are most suitable in structures assigned to lower Seismic Design Categories. Structural elements or systems identified as “intermediate” have moderately enhanced inelastic deformation capability, and are more suitable in structures assigned to intermediate Seismic Design Categories. Structural elements or systems identified as “special” have enhanced inelastic deformation capability and can be used in structures assigned to highest Seismic Design Categories. The degree of required toughness and, therefore, the level of required detailing, increases for structures progressing from ordinary through intermediate to special categories. It is essential that structures assigned to higher SDCs possess a higher degree of toughness. It is permitted, however, to design for higher toughness in the lower SDCs and take advantage of the lower design force levels.

The provisions of Chapters 1 through 19 and 22 are considered to be adequate for structures assigned to SDC A (corresponding to lowest seismic hazard). For structures assigned to SDC
Category B, additional requirements apply.

Structures assigned to Seismic Design Category B are not expected to be subjected to strong ground motion, but instead are expected to experience only low levels of shaking ground motion at an average long time intervals. ACI 318 provides some requirements for beam-column ordinary moment frames to improve toughness deformation capacity at minimal cost.

Structures assigned to SDC Seismic Design Category C may be subjected to moderately strong ground shaking motion. The designated seismic-force-resisting system typically comprises some combination of ordinary cast-in-place structural walls, intermediate precast structural walls, and intermediate moment frames. The legally adopted general building code of which this Code forms a part also may contain provisions for use of other seismic-force-resisting systems in SDC Seismic Design Category C. Section 21.1.1.7–20.2.1.6 defines requirements for whatever system is selected.

Structures assigned to SDC Seismic Design Category D, E, or F may be subjected to strong ground shaking motion. It is the intent of Committee 318 that the seismic-force-resisting system of structural concrete buildings assigned to SDC Seismic Design Category D, E, or F be provided by special moment frames, special structural walls, or a combination of the two. In addition to 21.1.2 through 21.1.8–20.2.2 through 20.2.8, these structures also are required to satisfy requirements for continuous inspection (4.3.5–1.5.4), diaphragms and trusses (21.11–20.12), foundations (21.12–20.13), and gravity-load-resisting elements that are not designated as part of the seismic-force-resisting system (21.13–20.14). These provisions have been developed to provide the structure with adequate toughness deformation capacity for the high demands expected for these SDC Seismic Design Categories.

The legally adopted general building code of which this Code forms a part may also permit the use of intermediate moment frames as part of dual systems for some buildings assigned to SDC Seismic Design Category D, E, or F. It is not the intention of ACI Committee 318 to recommend the use of intermediate moment frames as part of moment-resisting frame or dual systems in SDC Seismic Design Category D, E, or F. The legally adopted general building code may also permit substantiated alternative or nonprescriptive designs or, with various supplementary provisions, the use of ordinary or intermediate systems for nonbuilding structures in the higher SDC Seismic Design Categories. These are not the typical applications around which that were considered in the writing of this chapter is written, but wherever the term “ordinary” or “intermediate” moment frame is used in reference to reinforced concrete, 21.2–20.3 or 21.3–20.4 apply.

Table R21.1–R20.2 summarizes the applicability of the provisions of Chapter 21–20 as they are typically applied where when using the minimum requirements in the various SDC Seismic Design Categories. Where special systems are used for structures in SDC Seismic Design Category B or C, it is not required to satisfy the requirements of 21.13–20.14, although it should be verified that members not designated as part of the seismic-force-resisting system will be stable under design displacements.

The proportioning and detailing requirements in Chapter 21–20 are based predominantly on field and laboratory
experience with monolithic reinforced concrete building structures and precast concrete building structures designed
and detailed to behave like monolithic building structures. Extrapolation of these requirements to other types of cast-
in-place or precast concrete structures should be based on evidence provided by field experience, tests, or analysis.
The acceptance criteria for moment frames given in ACI 374.1 can be used in conjunction with Chapter 21 to
demonstrate that the strength, energy dissipation capacity, and toughness deformation capacity of a proposed frame
system equals or exceeds that provided by a comparable monolithic concrete system. ACI ITG-5.1 provides similar
information for precast wall systems.

The toughness requirements in 21.1.1.8 20.2.1.7 refer to the concern for the requirement to maintain structural
integrity of the entire seismic-force-resisting system at lateral displacements anticipated for ground motions
corresponding to the maximum considered earthquake motion, design earthquake. Depending on the energy-
dissipation characteristics of the structural system used, such displacements may be larger than for a monolithic
reinforced concrete structure satisfying the prescriptive provisions of other parts of this Code.

R20.2.2 — Analysis and proportioning of structural members

It is assumed that the distribution of required strength to the various components of a seismic-force-resisting system
will be guided by determined from the analysis of a linearly elastic model of the system acted upon by the factored
forces, as required by the general building code legally adopted general building code. If nonlinear response history
analyses are to be used, base motions should be selected after a detailed study of the site conditions and local
seismic history.

Because the design basis for earthquake-resistant design admits nonlinear response, it is necessary to investigate the
stability of the seismic-force-resisting system, as well as its interaction with other structural and nonstructural
members, under expected lateral displacements corresponding to maximum considered earthquake ground motion,
at displacements larger than those indicated by linear analysis. To handle this without having to resort to nonlinear
response analysis, one option is to multiply by a factor of at least two the displacements from linear analysis by
using the factored lateral forces, unless the legally adopted general building code specifies the factors to be used as in the IBC or the UBC. For lateral displacement calculations, assuming all the horizontal
structural members to be fully cracked is likely to lead to better estimates of the possible drift than using uncracked
stiffness for all members. The analysis assumptions described in 8.8.6.3.1.2 and 8.6.3.2.3 8.6.3.1.4 also may be
used to estimate lateral deflections of reinforced concrete building systems.

The main objective of Chapter 21 is the safety of the structure. The intent of 21.1.2.1 and 21.1.2.2-20.2.2.1 and
20.2.2.2 is to draw attention to the influence of nonstructural members on structural response and to hazards from
falling objects.

Section 21.1.2.3-20.2.2.3 serves as an alert that the base of structure as defined in analysis may not necessarily
correspond to the foundation or ground level. Details of columns and walls extending below the base of structure to
the foundation are required to be consistent with those above the base of structure.

In selecting member sizes for earthquake-resistant structures, it is important to consider constructability problems related to congestion of reinforcement. The design should be such that all reinforcement can be assembled and placed in the proper location and that concrete can be cast and consolidated properly. Use of the upper limits of permitted reinforcement ratios is likely to lead to insurmountable construction problems, especially at frame joints.

**R20.2.4 — Strength reduction factors**

Section 20.2.4.1(a) refers to brittle shear-controlled members such as low-rise walls, portions of walls between openings, or diaphragms for which it is impractical to reinforce to raise their nominal shear strength. The requirement of 20.2.4.1(b) is intended to increase the shear strength of the diaphragm and its connections in buildings for which the shear strength reduction factor for walls is 0.60, as those structures tend to have relatively high overstrength. The shear strength reduction factor for diaphragms to be 0.60 if the shear strength reduction factor for the walls is 0.60.

**R20.2.5 — Concrete in special moment frames and special structural walls**

Requirements of this section refer to concrete quality in frames and walls that resist earthquake-induced forces. The maximum specified compressive strength of lightweight concrete to be used in structural design calculations is limited to 5000 psi, primarily because of paucity of experimental and field data on the behavior of members made with lightweight concrete subjected to displacement reversals in the nonlinear range. If convincing evidence is developed for a specific application, the limit on maximum specified compressive strength of lightweight concrete may be increased to a level justified by the evidence.
R20.2.6 — Reinforcement in special moment frames and special structural walls <<Note: This commentary is left here for now pending updates in chapter 6.>>

Use of longitudinal reinforcement with strength substantially higher than that assumed in design will lead to higher shear and bond stresses at the time of development of yield moments. These conditions may lead to brittle failures in shear or bond and should be avoided even if such failures may occur at higher loads than those anticipated in design. Therefore, a ceiling an upper limit is placed on the actual yield strength of the steel [see 21.1.5.2(a) 6.3.4]. ASTM A706 for low-alloy steel reinforcing bars now includes both Grade 60 (442) and Grade 80 (550), however, only Grade 60 is generally permitted because of insufficient data to confirm applicability of existing code provisions for structures using the higher grade. Section 21.1.1.8 20.2.1.7 permits alternative material such as ASTM A706 Grade 80 if results of tests and analytical studies are presented in support of its use.

The requirement for a tensile strength larger than the yield strength of the reinforcement [21.1.5.2(b) 6.3.4] is based on the assumption that the capability of a structural member to develop inelastic rotation capacity is a function of the length of the yield region along the axis of the member. In interpreting experimental results, the length of the yield region has been related to the relative magnitudes of nominal and yield moments. According to this interpretation, the larger the ratio of nominal to yield moment, the longer the yield region. Chapter 21.6 requires that the ratio of actual tensile strength to actual yield strength is not less than be at least 1.25. Members with reinforcement not satisfying this condition can also develop inelastic rotation, but their behavior is sufficiently different to exclude them from direct consideration on the basis of rules derived from experience with members reinforced with strain-hardening steel.

The restrictions on the values of \( f_y \) and \( f_{yt} \) apply to all types of transverse reinforcement, including spirals, circular hoops, rectilinear hoops, and crossties. The restrictions on the values of \( f_y \) and \( f_{yt} \) in 11.4.2 6.3.4 for computing nominal shear strength are intended to limit the width of shear cracks. Research results 20.9.20.11 indicate that higher yield strengths can be used effectively as confinement reinforcement as specified in 21.6.4.4 20.7.5.4.

R20.2.7 — Mechanical splices in special moment frames and special structural walls <<Note: This is written assuming that the provisions for mechanical splices remain in this chapter.>>

In a structure undergoing inelastic deformations during an earthquake, the tensile stresses in reinforcement may approach the tensile strength of the reinforcement. The requirements for Type 2 mechanical splices are intended to avoid a splice failure when the reinforcement is subjected to expected stress levels in yielding regions. Type 1 mechanical splices are not required to satisfy the more stringent requirements for Type 2 mechanical splices, and may not be capable of resisting the stress levels expected in yielding regions. The locations of Type 1 mechanical splices are restricted because tensile stresses in reinforcement in yielding regions can exceed the strength requirements of 21.5.5 22.1.5.7. The restriction on Type 1 mechanical splices applies to all reinforcement resisting earthquake effects, including transverse reinforcement.
Recommended detailing practice would preclude the use of splices in regions of potential yielding in members resisting earthquake effects. If use of mechanical splices in regions of potential yielding cannot be avoided, there should be documentation on the actual strength characteristics of the bars to be spliced, on the force-deformation characteristics of the spliced bar, and on the ability of the Type 2 mechanical splice to be used to meet the specified performance requirements.

Although mechanical splices as defined by 20.2.7 need not be staggered, staggering is encouraged and may be necessary for constructability or provide enough space around the splice for installation or to meet the clear spacing requirements. \(<\text{CB035}>\)

**R20.2.8 — Welded splices in special moment frames and special structural walls**

**R20.2.8.1** — Welding of reinforcement should be according to in accordance with AWS D1.4 as required in Chapter 3-23.5.4. \(<\text{Latest version not posted at the time of this update}>\). The locations of welded splices are restricted because reinforcement tension stresses in yielding regions can exceed the strength requirements of 12.14.3.4 21.4.7. The restriction on welded splices applies to all reinforcement resisting earthquake effects, including transverse reinforcement.

**R20.2.8.2** — Welding of crossing reinforcing bars can lead to local embrittlement of the steel. If welding of crossing bars is used to facilitate fabrication or placement of reinforcement, it should be done only on bars added for such purposes. The prohibition of welding crossing reinforcing bars does not apply to bars that are welded with welding operations under continuous, competent control as in the manufacture of welded wire reinforcement.

**R20.3 — Ordinary moment frames**

These provisions were introduced in the 2008 Code and apply only to ordinary moment frames assigned to SDC Seismic Design Category B. The provisions for beam reinforcement are intended to improve continuity in the framing members as compared with the provisions of Chapters 1 through 18 and 21 through 23 and thereby improve lateral force resistance and structural integrity; these provisions do not apply to slab-column moment frames. The provisions for columns are intended to provide additional toughness capacity to resist shear for columns with proportions that would otherwise make them more susceptible to shear failure under earthquake loading.

**R20.4 — Intermediate moment frames**

The objective of the requirements in 21.3.3-20.4.2.3 and 20.4.3.1 is to reduce the risk of failure in shear in beams and columns during an earthquake. Two options are provided to determine the factored shear force.

**R20.4.2** — Section 20.4.2 contains requirements for providing beams with a threshold level of toughness.

According to 21.3.3.1(a) and 21.3.3.2(a) 20.4.2.3(a), the factored shear force is determined from a free-body diagram obtained by cutting through the member beam ends, with end moments assumed equal to the nominal
moment strengths acting in reverse curvature bending. Examples for a beam and a column are illustrated in (Fig. R21.3.3-R20.4.2). In all applications of 21.3.3.1(a) and 21.3.3.2(a) 20.4.2.3(a), shears are required to be calculated for moments due to reverse curvature bending, acting both clockwise and counterclockwise. Figure R21.3.3-R20.4.2 demonstrates only one of the two options that are to be considered for every member beam. The factored axial force, \( P_u \), should be chosen to develop the largest moment strength of the column.

To determine the maximum beam shear, it is assumed that its nominal moment strengths (\( \phi = 1.0 \) for moment) are developed simultaneously at both ends of its clear span. As indicated in Fig. R21.3.3-R20.4.2, the shear associated with this condition \( [(M_{nl} + M_{nr})/l_n] \) is added algebraically to the shear due to the factored gravity loads to obtain the design shear for the beam. For the example shown, both the dead load \( w_D \) and the live load \( w_L \) have been assumed to be uniformly distributed. Effects of \( E \) acting vertically are to be included if required by the general building code.

Option 21.3.3.1(b)-20.4.2.3(b) for beams bases \( V_u \) on the load combination including the earthquake effect, \( E \), which should be doubled. For example, the load combination defined by Eq. (9.5.3.1.e) would be

\[
U = 1.2D + 2.0E + 1.0L + 0.2S
\]

where \( E \) is the value specified by the governing code general building code. The factor of 1.0 applied to \( L \) is allowed to be reduced to 0.5 in accordance with 9.2.1(a)-7.3.3.

Transverse reinforcement at the ends of the beam is required to be hoops. In most cases, stirrups transverse reinforcement required by 21.3.3-20.4.2.3 for the design shear force will be more than those required by 20.4.2.4.

Beams may have be subjected to axial compressive force due to prestressing or applied loads. The additional requirements in 20.4.2.6 are intended to provide lateral support for beam longitudinal reinforcement.

**R20.4.3** — Section 20.4.3 contains requirements for providing columns with a threshold level of toughness. According to 21.3.3.1(a) and 21.3.3.2(a) 20.4.3.1(a), the factored shear force is determined from a free-body diagram obtained by cutting through the member column ends, with end moments assumed equal to the nominal moment strengths acting in reverse curvature bending. Examples for a beam and a column are illustrated in (Fig. R21.3.3-R20.4.2). In all applications of 21.3.3.1(a) and 21.3.3.2(a) 20.4.3.1(a), shears are required to be calculated for moments due to reverse curvature bending, acting both clockwise and counterclockwise. Figure R21.3.3-R20.4.2 demonstrates only one of the two options that are to be considered for every member column. The factored axial force, \( P_u \), should be chosen to develop the largest moment strength of the column within the range of design axial forces.

Option 21.3.3.1(b)-20.4.3.1(b) for columns is similar to that option 20.4.2.3(b) for beams except it bases \( V_u \) on load combinations including the earthquake effect, \( E \), with \( E \) increased by the overstrength factor, \( \Omega_o \), rather than the factor 2.0. In ASCE 7-10, \( \Omega_o = 3.0 \) for intermediate moment frames. The higher factor for columns relative to beams is because of greater concerns about shear failures in columns.
Transverse reinforcement at the ends of columns is required to be spirals or hoops. The amount of transverse reinforcement at the ends must satisfy both 20.4.3.1 and 20.4.3.2. Note that hoops require seismic hooks at both ends.

Discontinuous structural walls and other stiff members can impose large axial forces on supporting columns during earthquakes. The required transverse reinforcement in 21.3.5.6 20.4.3.6 is to improve column toughness under anticipated demands. The factored axial compressive force related to earthquake effect should include the factor $\Omega_o$ if required by the legally adopted general building code of which this Code forms a part.

**R20.4.5** — Section 20.4.5 applies to two-way slabs without beams, such as flat plates. Using load combinations of Eq. (9-5) 7.3.1e and (9-7) 7.3.1g may result in moments requiring top and bottom reinforcement at the supports.

The moment $M_{slab}$ refers, for a given design load combination with $E$ acting in one horizontal direction, to that portion of the factored slab moment that is balanced by the supporting members at a joint. It is not necessarily equal to the total design moment at the support for a load combination including earthquake effect. In accordance with 43.3.2.12.4.4.2, only a fraction of the moment $M_{slab}$ is assigned to the slab effective width. For edge and corner connections, flexural reinforcement perpendicular to the edge is not considered fully effective unless it is placed within the effective slab width.20.12,20.13 See Fig. **R21.3.6.1 R20.4.5.1**.

Application of the provisions of 21.3.6 20.4.5 is illustrated in Fig. **R21.3.6.2 and R21.3.6.3 R20.4.5.2 and R20.4.5.3**.

**R20.4.5.8** — The requirements apply to two-way slabs that are designated part of the seismic-force-resisting system. Slab-column connections in laboratory tests exhibited reduced lateral displacement ductility when the shear at the column connection exceeded the recommended limit. Slab-column connections also must satisfy shear and moment strength requirements of Chapters 11 and 13 under load combinations including earthquake effect.

**R20.5** — Intermediate precast structural walls

Connections between precast wall panels or between wall panels and the foundation are required to resist forces induced by earthquake motions and to provide for yielding in the vicinity of connections. When Type 2 mechanical splices are used to directly connect primary reinforcement, the probable strength of the splice should be at least 1.5 $t$ times the specified yield strength of the reinforcement.

**R20.6** — Flexural members **Beams** of special moment frames

**R20.6.1** — Scope

This section refers to beams of special moment frames resisting lateral loads induced by earthquake motions.
In previous Codes, any frame member subjected to a factored axial compressive force exceeding \( \frac{A_f f_c}{10} \) under any load combination was to be proportioned and detailed as described in 21.6-20.7. In the 2014 Code, all requirements for beams are contained in 20.6 regardless of the magnitude of axial compressive force.

This Code is written with the assumption that special moment frames comprise horizontal beams and vertical columns interconnected by beam-column joints. It is acceptable for beams and columns to be inclined provided the resulting system behaves as a frame, that is, lateral resistance is provided primarily by moment transfer between beams and columns rather than by strut or brace action. It is acceptable for beams of special moment frames to cantilever beyond columns, but such cantilevers are not part of the special moment frame that forms part of the seismic-force-resisting system. It is also acceptable for beams of a special moment frame to frame into a wall boundary if the boundary is reinforced as a special moment frame column in accordance with 20.7. A concrete braced frame, in which lateral resistance is provided primarily by axial forces in beams and columns, is not a recognized seismic-force-resisting system for buildings assigned to Seismic Design Category D, E, or F.

### R20.6.2 — Dimensional Limits

Experimental evidence indicates that, under reversals of displacement into the nonlinear range, behavior of continuous members having length-to-depth ratios of less than 4 is significantly different from the behavior of relatively slender members. Design rules derived from experience with relatively slender members do not apply directly to members with length-to-depth ratios less than 4, especially with respect to shear strength.

Geometric constraints indicated in 21.5.1.3 and 21.5.1.4-20.6.2.1(b) and (c) were derived from practice and research on reinforced concrete frames resisting earthquake-induced forces. The limits in 21.5.1.4-20.6.2.1(c) recognize that the maximum effective beam width that can effectively transfer forces into the beam-column joint depends principally on the column dimensions rather than on the depth of the beam, as suggested in the 2005 and earlier versions of the Code. An example of maximum effective beam width is shown in Fig. R21.5.1-R20.6.2.

### R20.6.3 — Longitudinal reinforcement

Section 10.3.5 limits the net tensile strain, \( \varepsilon_t \), thereby indirectly limiting the tensile reinforcement ratio in a flexural member to a fraction of the amount that would produce balanced conditions. For a section subjected to bending only and loaded monotonically to yielding, this approach is feasible because the likelihood of compressive failure can be estimated reliably with the behavioral model assumed for determining the reinforcement ratio corresponding to balanced failure. The same behavioral model (because of incorrect assumptions such as linear strain distribution, well-defined yield point for the steel, limiting compressive strain in the concrete of 0.003, and compressive stresses in the shell concrete) does not describe the conditions in a flexural member subjected to reversals of displacements well into the inelastic range. Thus, there is little rationale for continuing to refer to balanced conditions in earthquake-resistant design of reinforced concrete structures.<<This seems out of date. We do not need to “excuse” this any more.>>
R20.6.3.1 — The limiting reinforcement ratio of 0.025 is based primarily on considerations of steel reinforcement congestion and, indirectly, on limiting shear stresses in beams of typical proportions. The requirement of at least two bars, top and bottom, refers again to construction rather than behavioral requirements.

R20.6.3.3 — Lap splices of reinforcement are prohibited at regions along lengths where flexural yielding is anticipated because such splices are not reliable under conditions of cyclic loading into the inelastic range. Transverse reinforcement for lap splices at any location is mandatory because of the likelihood potential of loss of shell spalling of concrete cover spalling and the need to confine the splice.

R20.6.3.5 — These provisions were developed, in part, based on observations of building performance in earthquakes. For calculating the average prestress, the smallest cross-sectional dimension in a beam normally is the web dimension, and is not intended to refer to the flange thickness. In a potential plastic hinge region, the limitation on strain and the requirement for unbonded tendons are intended to prevent fracture of tendons under inelastic earthquake deformation. Calculation of the strain in the prestressing steel is required considering the anticipated inelastic mechanism of the structure. For prestressing steel unbonded along the full beam span, strains generally will be well below the specified limit. For prestressing steel with short unbonded length through or adjacent to the joint, the additional strain due to earthquake deformation is calculated as the product of the depth to the neutral axis and the sum of plastic hinge rotations at the joint, divided by the unbonded length.

The restrictions on the flexural strength provided by the tendons are based on the results of analytical and experimental studies. Although satisfactory seismic performance can be obtained with greater amounts of prestressing steel, this restriction is needed to allow the use of the same response modification and deflection amplification factors as those specified in model codes for special moment frames without prestressing steel. Prestressed special moment frames will generally contain continuous prestressing steel that is anchored with adequate cover at or beyond the exterior face of each beam-column connection located at the ends of the moment frame.

Fatigue testing for 50 cycles of loading between 40 and 80 percent of the specified tensile strength of the prestressing steel has been an industry practice of a long standing industry practice. The 80 percent limit was increased to 85 percent to correspond to the 1 percent limit on the strain in prestressing steel. Testing over this range of stress is intended to conservatively simulate the effect of a severe earthquake. Additional details on testing procedures, but to different stress levels, are provided in Reference 20.19.

R20.6.4 — Transverse reinforcement

Transverse reinforcement is required primarily to confine the concrete and maintain lateral support for the reinforcing bars in regions where yielding is expected. Examples of hoops suitable for flexural members of frames beams are shown in Fig. R21.5.3.R20.6.4.

For many years In earlier Code editions, the upper limit on hoop spacing was the smallest of d/4, 8 longitudinal bar
diameters, 24 tie bar diameters, and 12 in. The upper limits were changed in the 2011 edition because of concerns about adequacy of longitudinal bar buckling restraint and confinement of in large beams.

In the case of members with varying strength along the span or members for which the permanent load represents a large proportion of the total design load, concentrations of inelastic rotation may occur within the span. If such a condition is anticipated, transverse reinforcement also should be provided in regions where yielding is expected. Because spalling of the concrete shell might occur during strong ground motion, especially at and near regions of flexural yielding, all web reinforcement should be provided in the form of closed hoops as defined in 20.5.3.5.

R20.6.5 — Shear strength requirements

R20.6.5.1 — Design forces

In determining the equivalent lateral forces representing earthquake effects for the type of frames considered, it is assumed that frame members will dissipate energy in the nonlinear range of response. Unless a frame member beam possesses a moment strength that is on the order of 3 or 4 of times the design moment forces, it should be assumed that it will yield in flexure in the event of a major earthquake. The design shear force should be selected so as to be a good approximation of the maximum shear that may develop in a member. Therefore, required shear strength for frame members is related to flexural strengths of the as-designed member rather than to factored shear forces indicated by lateral load analysis. The conditions described by 20.6.5.1 are illustrated in Fig. R20.5.4.

Because the actual yield strength of the longitudinal reinforcement may exceed the specified yield strength and because strain hardening of the reinforcement is likely to take place at a joint subjected to large rotations, required shear strengths are determined using a yield stress of at least 1.25fy in the longitudinal reinforcement.

R20.6.5.2 — Transverse reinforcement

Experimental studies of reinforced concrete members subjected to cyclic loading have demonstrated that more shear reinforcement is required to ensure a flexural failure if the member is subjected to alternating nonlinear displacements than if the member is loaded in only one direction: the necessary increase of shear reinforcement being higher in the case of no axial load. This observation is reflected in the Code (see 20.6.5.2) by eliminating the term representing the contribution of concrete to shear strength. The added conservatism on shear is deemed necessary in locations where potential flexural hinging may occur. However, this stratagem, chosen for its relative simplicity, should not be interpreted to mean that no concrete is required to resist shear. On the contrary, it may be argued that the concrete core resists all the shear with the shear (transverse) reinforcement confining and strengthening the concrete. The confined concrete core plays an important role in the behavior of the beam and should not be reduced to a minimum just because the design expression does not explicitly recognize it.
R20.7 — **Columns of special moment frames members subjected to bending and axial load**

R20.7.1 — Scope

This section is intended primarily for columns of special moment frames regardless of the magnitude of axial force load. Before 2014, the Code permitted columns with low levels of axial stress to be detailed as beams. Frame members, other than columns, that do not satisfy 20.5.1 are to be proportioned and detailed according to this section. These provisions apply to the frame member for all load combinations if the axial load exceeds 0.14\( f'_c \) in any load combination.

R20.7.2 — Dimensional limits

The geometric constraints in 21.6.1.1 and 21.6.1.2 follow from previous practice.\(^{20.22}\)

R20.7.3 — Minimum flexural strength of columns

The intent of 21.6.2.2-20.7.3.2 is to reduce the likelihood of yielding in columns that are considered as part of the seismic-force-resisting system. If columns are not stronger than beams framing into a joint, there is increased likelihood of inelastic action. In the worst case of weak columns, flexural yielding can occur at both ends of all columns in a given story, resulting in a column failure mechanism that can lead to collapse.

In 21.6.2.2-20.7.3.2, the nominal strengths of the girders beams and columns are calculated at the joint faces, and those strengths are compared directly using Eq. (21.1-20.7.3.2). The 1995 and earlier Codes required design strengths to be compared at the center of the joint, which typically produced similar results but with added computational calculation effort.

When determining the nominal flexural moment strength of a girder beam section in negative bending (top in tension), longitudinal reinforcement contained within an effective flange width of a top slab that acts monolithically with the girder beam increases the girder beam strength. Research\(^{20.23}\) on beam-column subassemblies under lateral loading indicates that using the effective flange widths defined in 8.10-8.3.3 gives reasonable estimates of girder beam negative bending moment strengths of interior connections at interstory displacements levels approaching 2 percent of story height. This effective width is conservative where the slab terminates in a weak spandrel.

If 21.6.2.2-20.7.3.2 cannot be satisfied at a joint, 21.6.2.3-20.7.3.3 requires that any positive contribution of the column or columns involved to the lateral strength and stiffness of the structure is to be ignored. Negative contributions of the column or columns should not be ignored. For example, ignoring the stiffness of the columns ought not to be used as a justification for reducing the design base shear. If inclusion of those columns in the analytical model of the building results in an increase in torsional effects, the increase should be considered as required by the general building code governing code. Furthermore, the column must be provided with transverse reinforcement to increase its toughness capacity to resistance to shear and axial forces.
R20.7.4 — Longitudinal reinforcement

The lower limit of the area of longitudinal reinforcement is to control time-dependent deformations and to have the yield moment exceed the cracking moment. The upper limit of the section area reflects concern for steel reinforcement congestion, load transfer from floor elements to column (especially in low-rise construction) and the development of high shear stresses.

Spalling of the shell concrete, which is likely to occur near the ends of the column in frames of typical configuration, makes lap splices in these locations vulnerable. If lap splices are to be used at all, they should be located near the midheight where stress reversal is likely to be limited to a smaller stress range than at locations near the joints. Transverse reinforcement is required along the lap-splice length because of the uncertainty in moment distributions along the height and the need for confinement of lap splices subjected to stress reversals. 20.24

R20.7.5 — Transverse reinforcement

Requirements of this section are concerned with confining the concrete and providing lateral support to the longitudinal reinforcement.

R20.7.5.1 — Section 21.6.4.1 20.7.5.1 This section stipulates a minimum length over which to provide closely-spaced transverse reinforcement at the member column ends, where flexural yielding normally occurs. Research results indicate that the length should be increased by 50 percent or more in locations, such as the base of a building, where axial loads and flexural demands may be especially high. 20.25

R20.7.5.2 — Sections 21.6.4.2 20.7.5.2 and 21.6.4.3 20.7.5.3 provide requirements for configuration of transverse reinforcement for columns and joints of special moment frames. Figure R21.6.4.2 20.7.5.2 shows an example of transverse reinforcement provided by one hoop and three crossties. Crossties with a 90-degree hook are not as effective as either crossties with 135-degree hooks or hoops in providing confinement. Tests show that if crosstie ends with 90-degree hooks are alternated, confinement will be sufficient. For lower values of \( P_u/A_{fg}' \) and lower concrete compressive strengths, crossties with 90-degree hooks are adequate if the ends are alternated along the length and around the perimeter of the column. For higher values of \( P_u/A_{fg}' \), for which compression-controlled behavior is expected, and for higher compressive strengths, for which behavior tends to be more brittle, the improved confinement provided by having corners of hoops or seismic hooks supporting all longitudinal bars is important to achieving intended performance. Where these conditions apply, crossties with seismic hooks at both ends are required. The 8-in. spacing of hoop and crosstie legs is also intended to improve performance under these critical conditions. For bundled bars, bends or hooks of hoops and crossties need to enclose the bundle, and longer
extensions on hooks should be considered. Column axial load, $P_u$, should reflect factored compressive demands from both earthquake and gravity loads. \(<\text{CH031}>>\)

In past editions of the Code, the requirements for transverse reinforcement in columns, walls, beam-column joints, and diagonally reinforced coupling beams referred to the same equations. In the 2014 edition of the Code, the equations and detailing requirements differ among the member types based on consideration of their loadings, deformations, and performance requirements. \(<\text{CH031}>>\)

R20.7.5.3 — The requirement that spacing not exceed one-quarter of the minimum member dimension is to obtain adequate concrete confinement. The requirement that spacing not exceed six bar diameters is intended to restrain longitudinal reinforcement buckling after spalling. The 4 in. spacing is for concrete confinement; 21.6.4.3 permits this limit to be relaxed to a maximum of 6 in. if the spacing of crossties or legs of overlapping hoops is less than 8 in. or less.

R20.7.5.4 — The effect of helical (spiral) reinforcement and adequately configured rectilinear hoop reinforcement on strength and ductility of columns is well established. While analytical procedures exist for calculation of strength and ductility capacity of columns under axial and moment reversals, the axial load and deformation demands during earthquake loading are not known with sufficient accuracy to justify calculation of required transverse reinforcement as a function of design earthquake demands. Instead, Eq. (10-5 14.6.3.1) and or (21-4 20.7.5.4b) are required, with the intent that spalling of shell concrete will not result in a loss of column axial load strength. Expressions (c) and (f) were developed from a review of column test data and are intended to result in columns capable of sustaining a drift ratio of 0.03 with limited strength degradation. Expressions (c) and (f) are triggered for axial load greater than 0.3A'$f'_c$, which corresponds approximately to the onset of compression-controlled behavior for symmetrically reinforced columns. The $k_n$ term decreases the required confinement for columns with closely spaced, laterally supported longitudinal reinforcement because such columns are more effectively confined than columns with more widely spaced longitudinal reinforcement. The $k_f$ term increases the required confinement for columns with $f'_c > 10,000$ psi.
because such columns can experience brittle failure if not well confined. Concrete strengths greater than 15,000 psi should be used with caution given the limited test data for such columns. The concrete strength used to determine the confinement reinforcement is required to be the same as that specified in the construction documents.

Equations (21-4 20.7.5.4b) and (21-5 20.7.5.4e) Expressions (a), (b), and (c) in Table 20.7.5.4 are to be satisfied in both cross-sectional directions of the rectangular core. For each direction, $b_c$ is the core dimension perpendicular to the tie legs that constitute $A_{sh}$, as shown in Fig. R21.6.4.2 R20.7.5.2.

Research results indicate that yield strengths higher than those specified in 11.4.2 6.3.4 high strength reinforcement can be used effectively as confinement reinforcement. Section 6.2.2.4 permits a value of $f_{y}$ as high as 100,000 psi to be used in Table 20.7.5.4. A value of $f_{y}$ of 100,000 psi is permitted in Eq. (21-3 20.7.5.4a), (21-4 20.7.5.4b), and (21-5 20.7.5.4c) where bars satisfying ASTM A1035 are used as confinement reinforcement.

R20.7.5.5 — The provisions of 21.6.4.5 20.7.5.5 are This provision is intended to provide reasonable protection and ductility to the midheight of columns outside the length $l_o$. Observations after earthquakes have shown significant damage to columns in this region, and the minimum ties hoops or spirals required should provide a more uniform toughness strength of the column along its length.

R20.7.5.6 — Columns supporting discontinued stiff members, such as walls or trusses, may develop considerable inelastic response. Therefore, it is required that these columns have the specified reinforcement throughout their length. This covers all columns beneath the level at which the stiff member has been discontinued, unless the factored forces corresponding to earthquake effect are low. See R21.11.7.5 R20.12.7.5 for discussion of the overstrength factor $\Omega_o$ applied in some codes.

R20.7.5.7 — The unreinforced shell may spall as the column deforms to resist earthquake effects. Separation of portions of the shell from the core caused by local spalling creates a falling hazard. The additional reinforcement is required to reduce the risk of portions of the shell falling away from the column.

R20.7.6 — Shear strength requirements

R20.7.6.1 — Design forces

The procedures of 21.5.4.1 20.6.5.1 also apply to members subjected to axial loads (for example, columns). Above the ground floor, the moment at a joint may be limited by the flexural strength of the beams framing into the joint. Where beams frame into opposite sides of a joint, the combined strength may be the sum of the negative moment strength of the beam on one side of the joint and the positive moment strength of the beam on the other side of the joint. Moment strengths are to be determined using a strength reduction factor of 1.0 and reinforcing steel.
reinforcement with an effective yield stress equal to at least $1.25f_y$. Distribution of the combined moment strength of the beams to the columns above and below the joint should be based on analysis. The value of $M_{pr}$ in Fig. R20.6.5 may be computed from the flexural member strengths at the beam-column joints.

R20.8 — Joints of special moment frames

R20.8.2 — General requirements

Development of inelastic rotations at the faces of joints of reinforced concrete frames is associated with strains in the flexural reinforcement well in excess of the yield strain. Consequently, joint shear force generated by the flexural reinforcement is calculated for a stress of $1.25f_y$ in the reinforcement (see 21.7.2.1-20.8.2.1). A detailed explanation of the reasons for the possible development of stresses in excess of the yield strength in beam tensile reinforcement is provided in Reference 20.8.

R20.8.2.2 — The design provisions for hooked bars are based mainly on research and experience for joints with standard 90-degree hooks. Therefore, standard 90-degree hooks generally are preferred to standard 180-degree hooks unless unusual considerations dictate use of 180-degree hooks. For bars in compression, the development length corresponds to the straight portion of a hooked or headed bar measured from the critical section to the onset of the bend for hooked bars and from the critical section to the head for headed bars. <<CH034>>

R20.8.2.3 — Research\textsuperscript{20.28-20.32} has shown that straight beam bars may slip within the beam-column joint during a series of large moment reversals. The bond stresses on these straight bars may be very large. To reduce slip substantially during the formation of adjacent beam hinging, it would be necessary to have a ratio of column dimension to bar diameter of approximately 1/32, which would result in very large joints. On reviewing the available tests, the limit of 1/20 of the column depth in the direction of loading for the maximum size of beam bars for normalweight concrete and a limit of 1/26 for lightweight concrete were chosen. Due to the lack of specific data for beam bars through lightweight concrete joints, the limit is based on an amplification factor of 1.3, which is approximately the reciprocal of the lightweight concrete modification factor of \textsuperscript{4} in Chapter 12.5.2.4, starting with the 1989 Code. The amplification factor was modified slightly in 2008 to 1/0.75 = 1.33, which did not affect this Code section. These limits provide reasonable control on the amount of potential slip of the beam bars in a beam-column joint, considering the number of anticipated inelastic excursions of the building frames during a major earthquake. A thorough treatment of this topic is given in Reference 20.33.

R20.8.3 — Transverse reinforcement

The Code requires transverse reinforcement in a joint regardless of the magnitude of the calculated shear force. In 21.7.3.2-20.8.3.2, the amount of confining reinforcement may be reduced and the spacing may be increased if horizontal members beams of adequate dimensions frame into all four sides of the joint. <<CH034>>

Section 21.7.3.3-20.8.3.3 refers to a joint where the width of the beam exceeds the corresponding column dimension.
In that case, beam reinforcement not confined by the column reinforcement should be provided lateral support either by a beam framing into the same joint or by transverse reinforcement.

An example of transverse reinforcement through the column provided to confine the beam reinforcement passing outside the column core is shown in Fig. R21.5.1—R20.6.2. Additional detailing guidance and design recommendations for both interior and exterior wide-beam connections with beam reinforcement passing outside the column core may be found in Reference 20.8.

R20.8.3.5—This section refers to a “knee” joint in which beam reinforcement terminates with headed deformed bars. Such joints require confinement of the headed beam bars along the top face of the joint. This confinement can be provided by either (a) a column that extends above the top of the joint or (b) vertical reinforcement hooked around the beam top reinforcing bars and extending downward into the joint in addition to the column longitudinal reinforcement. Detailing guidance and design recommendations for vertical joint reinforcement may be found in Reference 20.8. <<This should be the former reference 21.8>> <<CH034>>

R20.8.4—Shear strength

The requirements in Chapter 21 for proportioning joints are based on Reference 20.8 in that behavioral phenomena within the joint are interpreted in terms of a nominal shear strength of the joint. Because tests of joints20.28 and deep beams20.14 indicated that shear strength was not as sensitive to joint (shear) reinforcement as implied by the expression developed by Joint ACI-ASCE Committee 32620.34 for beams, Committee 318 set the strength of the joint as a function of only the compressive strength of the concrete (see 21.7.4—20.8.4) and requires a minimum amount of transverse reinforcement in the joint (see 21.7.3—20.8.3). The effective area of joint \( A_J \) is illustrated in Fig. R21.7.4—R20.8.4. In no case is \( A_J \) greater than the column cross-sectional area. A circular column should be considered as having a square section of equivalent area.

The three levels of shear strength required by 21.7.4.1—20.8.4.1 are based on the recommendation of ACI Committee 352.20.8. Test data reviewed by the committee20.25—20.26 indicate that the lower value given in 21.7.4.1—20.8.4.1 of the 1983 Code was unconservative when applied to corner joints.

Cyclic loading tests of joints with extensions of beams with lengths at least equal to their depths have indicated similar joint shear strengths to those of joints with continuous beams. These findings suggest that extensions of beams, when properly dimensioned and reinforced with longitudinal and transverse bars, provide effective confinement to the joint faces, thus delaying joint strength deterioration at large deformations.20.36

R20.8.5—Development length of bars in tension

R20.8.5.1—Minimum development embedment length in tension for deformed bars with standard hooks embedded in normal weight concrete is determined using Eq. (21—6—20.8.5.1), which is based on the requirements of 21.4.3. The development embedment length in tension of a deformed bar with a standard hook is defined as the distance, parallel to the bar, from the critical section (where the bar is to be developed) to a tangent drawn to the outside edge of the
hook. The tangent is to be drawn perpendicular to the axis of the bar (see Fig. R12.5).

Because Chapter 21 stipulates that the hook is to be embedded in confined concrete, the coefficients 0.7 (for concrete cover) and 0.8 (for ties) have been incorporated in the constant used in Eq. (21.6.20.8.5.1). The development length that would be derived directly from Eq. 12.5.21.4.3 is increased to reflect the effect of load reversals. Factors such as the actual stress in the reinforcement being more than the yield stress and the effective development length not necessarily starting at the face of the joint were implicitly considered in the development formulation of the expression for basic development length that has been used as the basis for Eq. (21.6.20.8.5.1).

For lightweight concrete, the length required by Eq. (21.6.20.8.5.1) is to be increased by 25 percent to compensate for variability of bond characteristics of reinforcing bars in various types of lightweight concrete.

The requirement for the hook to project into the joint is to improve development of a diagonal compression strut across the joint. The requirement applies to beam and column bars terminated at a joint with a standard hook.

The 3d_b spacing limit is based on limited studies of joints confined by transverse reinforcement consistent with special moment frame requirements in this chapter. To avoid congestion, it may be desirable to stagger the heads.

Minimum development length in tension for straight bars is a multiple of the length indicated by Eq. 21.7.5.2. Section 21.7.5.2(b) refers to top bars. Lack of reference to No. 14 and No. 18 bars in 20.8.5 is due to the paucity of information on anchorage of such bars subjected to load reversals simulating earthquake effects.

If the required straight embedment length of a reinforcing bar extends beyond the confined volume of concrete (as defined in 21.5.3, 21.6.4, 21.7.5, or 21.7.3), the required development length is increased on the premise that the limiting bond stress outside the confined region is less than that inside.

\[ l_{dm} = 1.6(l_d - l_{dc}) + l_{dc} \]

or

\[ l_{dm} = 1.6l_d - 0.6l_{dc} \]

where

- \( l_{dm} \) = required development length if bar is not entirely embedded in confined concrete;
- \( l_d \) = required development length in tension for straight bar as defined in 20.8.5.3; embedded in confined concrete;
- \( l_{dc} \) = length of bar embedded in confined concrete.
Lack of reference to No. 14 and No. 18 bars in 20.8.5 is due to the paucity of information on anchorage of such bars subjected to load reversals simulating earthquake effects. <<CH034>>

R20.9 — Special moment frames constructed using precast concrete

The detailing provisions in 21.8.2 and 21.8.3, 20.9.2.1 and 20.9.2.2 are intended to produce frames that respond to design displacements essentially like monolithic special moment frames.

Precast frame systems composed of concrete elements with ductile connections are expected to experience flexural yielding in connection regions. Reinforcement in ductile connections can be made continuous by using Type 2 mechanical splices or any other technique that provides development in tension or compression of at least 125 percent of the specified yield strength $f_y$ of bars and the specified tensile strength of bars. Requirements for mechanical splices are in addition to those in 21.1.6 and are intended to avoid strain concentrations over a short length of reinforcement adjacent to a splice device. Additional requirements for shear strength are provided in 21.8.2 to prevent sliding on connection faces. Precast frames composed of elements with ductile connections may be designed to promote yielding at locations not adjacent to the joints. Therefore, design shear, $V_e$, as computed according to 21.5.4.1 or 21.6.5.1, may not be conservative.

Precast concrete frame systems composed of elements joined using strong connections are intended to experience flexural yielding outside the connections. Strong connections include the length of the coupler mechanical splice hardware as shown in Fig. R21.8.3. Capacity-design techniques are used in 21.8.3(b) to ensure the strong connection remains elastic following formation of plastic hinges. Additional column requirements are provided to avoid hinging and strength deterioration of column-to-column connections.

Strain concentrations have been observed to cause brittle fracture of reinforcing bars at the face of mechanical splices in laboratory tests of precast beam-column connections. Locations of strong connections should be selected carefully or other measures should be taken, such as debonding of reinforcing bars in highly stressed regions, to avoid strain concentrations that can result in premature fracture of reinforcement.

R20.9.2.3 — Precast frame systems not satisfying the prescriptive requirements of Chapter 21 have been demonstrated in experimental studies to provide satisfactory seismic performance characteristics. ACI 374.1 defines a protocol for establishing a design procedure, validated by analysis and laboratory tests, for such frames. The design procedure should identify the load path or mechanism by which the frame resists gravity and earthquake effects. The tests should be configured to test investigate critical behaviors, and the measured quantities should establish upper-bound acceptance values for components of the load path, which may be in terms of limiting stresses, forces, strains, or other quantities. The design procedure used for the structure should not deviate from that used to design the test specimens, and acceptance values should not exceed values that were demonstrated by the tests to be acceptable. Materials and components used in the structure should be similar to those used in the tests. Deviations may be acceptable if the licensed design professional can demonstrate that those deviations do not...
adversely affect the behavior of the framing system.

ACI ITG-1.2.0.44 defines design requirements for one type of special precast concrete moment frame for use in accordance with 21.8.4-20.9.2.3.

R20.10 — Special structural walls and coupling beams

R20.10.1 — Scope

This section contains requirements for the dimensions and details of special structural walls and all components including coupling beams and wall piers. Wall piers are defined in 2.2-2.3. Design provisions for vertical wall segments depend on the aspect ratio of the wall segment in the plane of the wall \((h_w/l_w)\), and the aspect ratio of the horizontal cross section \((l_w/b_w)\), and generally follow the descriptions in Table R21.9.1-R20.10.1. The limiting aspect ratios for wall piers are based on engineering judgment. It is intended that flexural yielding of the vertical reinforcement in the pier should limit shear demand on the pier.

R20.10.2 — Reinforcement

Minimum reinforcement requirements in 21.9.2.1-20.10.2.1 follow from preceding Codes. The requirement for distributed shear reinforcement is related to the intent to control the width of inclined cracks. The requirement for two layers of reinforcement in walls carrying resisting substantial design shears in 21.9.2.2-20.10.2.2 is based on the observation that, under ordinary construction conditions, the probability of maintaining a single layer of reinforcement near the middle of the wall section is quite low. Furthermore, presence of reinforcement close to the surface tends to inhibit fragmentation of the concrete in the event of severe cracking during an earthquake. The requirement for two layers of vertical reinforcement in more slender walls is to improve lateral stability of the compression zone under cyclic loads following yielding of vertical reinforcement in tension. <<CH030>>

R20.10.2.3 — Requirements are based on provisions in Chapter 42-21. Because actual forces in longitudinal reinforcement of structural walls may exceed calculated forces, reinforcement should be developed or spliced to reach the yield strength of the bar in tension. Requirements of 12.11, 12.12, and 12.13 address issues related to beams and do not apply to walls. At locations where yielding of longitudinal reinforcement is expected, a 1.25 multiplier is applied to account for the likelihood that the actual yield strength exceeds the specified yield strength of the bar, as well as the influence of strain hardening and cyclic load reversals. Where transverse reinforcement is used, development lengths for straight and hooked bars may be reduced as permitted in 42.2-21.4.2 and 42.5-21.4.3, respectively, because closely spaced transverse reinforcement improves the performance of splices and hooks subjected to repeated inelastic demands. 20.45

R20.10.3 — Design forces

Design shears for structural walls are obtained from lateral load analysis with the appropriate load factors. However,
the possibility of yielding in components of such structures should be considered, as in the portion of a wall between
two window openings, in which case the actual shear may be in excess of the shear indicated by lateral load analysis
based on factored design forces.

R20.10.4 — Shear strength

Equation (21.7-20.10.4.1) recognizes the higher shear strength of walls with high shear-to-moment ratios. The nominal shear strength is given in terms of the net area of the section resisting shear. For a rectangular section without openings, the term $A_{cv}$ refers to the gross area of the cross section rather than to the
product of the width and the effective depth. The definition of $A_{cv}$ in Eq. (21.7-20.10.4.1) facilitates design calculations
for walls with uniformly distributed reinforcement and walls with openings.

A vertical wall segment refers to a part of a wall bounded horizontally by openings or by an opening and an edge. When
designing For an isolated wall or a vertical wall segment, $\rho_t$ refers to horizontal reinforcement and $\rho_l$ refers to
vertical reinforcement.

The ratio $h_w/l_w$ may refer to overall dimensions of a wall, or of a segment of the wall bounded by two openings, or an
opening and an edge. The intent of 21.9.4.2-20.10.4.2 is to make certain that any segment of a wall is not assigned a
unit strength larger than that for the entire wall. However, a wall segment with a ratio of $h_w/l_w$ higher than that of the
entire wall should be proportioned for the unit strength associated with the ratio $h_w/l_w$ based on the dimensions for that
segment.

To restrain the inclined cracks effectively, reinforcement included in $\rho_t$ and $\rho_l$ should be appropriately distributed along
the length and height of the wall (see 21.9.4.3-20.10.4.3). Chord reinforcement provided near wall edges in
concentrated amounts for resisting bending moment is not to be included in determining $\rho_t$ and $\rho_l$. Within practical limits,
shear reinforcement distribution should be uniform and at a small spacing.

If the factored shear force at a given level in a structure is resisted by several walls or several vertical wall segments
of a perforated wall, the average unit shear strength assumed for the total available cross-sectional area is limited to
$8\sqrt{f_c^{'}}$ with the additional requirement that the unit shear strength assigned to any single vertical wall segment does
not exceed $10\sqrt{f_c^{'}}$. The upper limit of strength to be assigned to any one member is imposed to limit the degree of
redistribution of shear force.

“Horizontal wall segments” in 21.9.4.5-20.10.4.5 refers to wall sections between two vertically aligned openings
(see Fig. 20.10.4.5). It is, in effect, a vertical wall segment rotated through 90 degrees. A horizontal wall
segment is also referred to as a coupling beam when the openings are aligned vertically over the building height.
When designing a horizontal wall segment or coupling beam, $\rho_t$ refers to vertical reinforcement and $\rho_l$ refers to
horizontal reinforcement.
R20.10.5 — Design for flexure and axial loads

R20.10.5.1 — Flexural strength of a wall or wall segment is determined according to procedures commonly used for columns. Strength should be determined considering the applied axial and lateral forces. Reinforcement concentrated in boundary elements and distributed in flanges and webs should be included in the strength calculations based on a strain compatibility analysis. The foundation supporting the wall should be designed to develop resist the wall boundary and web forces. For walls with openings, the influence of the opening or openings on flexural and shear strengths is to be considered and a load path around the opening or openings should be verified. Capacity-design concepts and strut-and-tie models may be useful for this purpose.\(^{20.47}\)

R20.10.5.2 — Where wall sections intersect to form L-, T-, C-, or other cross-sectional shapes, the influence of the flange on the behavior of the wall should be considered by selecting appropriate flange widths. Tests\(^{20.48}\) show that effective flange width increases with increasing drift level and the effectiveness of a flange in compression differs from that for a flange in tension. The value used for the effective compression flange width has little impact effect on the strength and deformation capacity of the wall; therefore, to simplify design, a single value of effective flange width based on an estimate of the effective tension flange width is used in both tension and compression.

R20.10.6 — Boundary elements of special structural walls

R20.10.6.1 — Two design approaches for evaluating detailing requirements at wall boundaries are included in Section 21.9.6.2. Section 21.9.6.2-20.10.6.2 allows the use of displacement-based design of walls, in which the structural details are determined directly on the basis of the expected lateral displacements of the wall. The provisions of 21.9.6.3-20.10.6.3 are similar to those of the 1995 Code, and have been retained because they are conservative for assessing required transverse reinforcement at wall boundaries for many walls. Requirements of 21.9.6.4-20.10.6.4 and 21.9.6.5-20.10.6.5 apply to structural walls designed by either 21.9.6.2-20.10.6.2 or 21.9.6.3-20.10.6.3.

R20.10.6.2 — Section 21.9.6.2 20.10.6.2 This section is based on the assumption that inelastic response of the wall is dominated by flexural action at a critical, yielding section. The wall should be proportioned so that the critical section occurs where intended. Equation (21-8-20.10.6.2) follows from a displacement-based approach.\(^{20.49,20.50}\) The approach assumes that special boundary elements are required to confine the concrete where the strain at the extreme compression fiber of the wall exceeds a critical value when the wall is displaced to 1.5 times the design displacement. The horizontal dimension of the special boundary element is intended to extend at least over the length where the compression strain exceeds the critical value. The height of the special boundary element is based on upper bound estimates of plastic hinge length and extends beyond the zone over which concrete spalling is likely to occur. The multiplier of 1.5 on design displacement was added to Equation 20.10.6.2 in the 2014 version of this Code to produce detailing requirements more consistent with the building code performance intent of a low probability of collapse in Maximum Considered Earthquake level.
shaking. The lower limit of 0.005 on the quantity \( \delta_u/h_w \) requires special boundary elements if wall boundary longitudinal reinforcement tensile strain does not reach approximately twice the limit used to define tension-controlled beam sections according to 9.4. The lower limit of 0.007 0.005 on the quantity \( \delta_u/h_w \) requires moderate wall deformation capacity for stiff buildings. \(<\text{CH30}>\)

The neutral axis depth \( c \) in Eq. (20.10.6.2) is the depth calculated according to 10.2 9.2, except the nonlinear strain requirements of 10.2.2 9.2.1.2 need not apply, corresponding to development of nominal flexural strength of the wall when displaced in the same direction as \( \delta_e \). The axial load is the factored axial load that is consistent with the design load combination that produces the design displacement \( \delta_e \).

The height of the special boundary element is based on estimates of plastic hinge length and extends beyond the zone over which yielding of tension reinforcement and spalling of concrete are likely to occur. \(<\text{CH30}>\)

**R20.10.6.3** — By this procedure, the wall is considered to be acted on by gravity loads and the maximum shear and moment induced by earthquake in a given direction. Under this loading, the compressed boundary at the critical section resists the tributary gravity load plus the compressive resultant associated with the bending moment.

Recognizing that this loading condition may be repeated many times during the strong motion, the concrete is to be confined where the calculated compressive stresses exceed a nominal critical value equal to \( 0.2f'_c \). The stress is to be calculated for the factored forces on the section assuming linear response of the gross concrete section. The compressive stress of \( 0.2f'_c \) is used as an index value and does not necessarily describe the actual state of stress that may develop at the critical section under the influence of the actual inertia forces for the anticipated earthquake intensity.

**R20.10.6.4** — The horizontal dimension of the special boundary element is intended to extend at least over the length where the concrete compression strain exceeds the critical value. For flanged wall sections, including box shapes, L shapes, and C shapes, the calculation to determine the need for special boundary elements should include a direction of lateral load consistent with the orthogonal combinations defined in ASCE 7. The value of \( c/2 \) in 21.9.6.4(a) 20.10.6.4(a) is to provide a minimum length of the special boundary element. Good detailing practice is to arrange the longitudinal reinforcement and the confinement reinforcement such that all primary longitudinal reinforcement at the wall boundary is supported by transverse reinforcement. \(<\text{CH30}>\)

A slenderness limit is introduced into the 2014 edition of this Code based on lateral instability failures of slender wall boundaries observed in recent earthquakes and tests. \( 20.XX, 20.YY \) For walls with large cover, where spalling of cover concrete would lead to a significantly reduced section, increased boundary element thickness should be considered. \(<\text{CH30}>\)
A value of $c/w > 3/8$ is used to define a wall critical section that is not tension-controlled according to 9.4. A minimum wall thickness of 12 in. is imposed to reduce the likelihood of lateral instability of the compression zone after spalling of cover concrete. 

Where flanges are heavily stressed in compression, the web-to-flange interface is likely to be heavily stressed and may sustain local crushing failure unless special boundary element reinforcement extends into the web. Equation (21.4-20.7.5.4b) does not apply to walls.

Required transverse reinforcement at wall boundaries is based on column provisions. Equation (20.7.5.4b) was applied to wall special boundary elements prior to the 1999 edition of this Code. It is reinstated in the 2014 edition of this Code due to concerns that Equation (20.7.5.4c) does not provide adequate transverse reinforcement for thin walls where concrete cover accounts for a significant portion of the wall thickness. For wall special boundary elements having rectangular cross section, $A_g$ and $A_{ch}$ in Eq. 20.7.5.4b are defined as $A_g = \ell_w b$ and $A_{ch} = b c_1 \ell_w b c_2$, where dimensions are shown in Figure R20.10.6.4.1. This considers that concrete spalling is likely to occur only on the exposed faces of the confined boundary element. The limits on $h_x$ are intended to provide more uniform spacing of hoops and crossties for thin walls. Tests show that adequate performance can be achieved using vertical spacing larger than permitted by 21.6.4.3(a) 20.7.5.3(a). Requirements for vertical extensions of boundary elements are summarized in Figure R.20.10.6.4.2.

The horizontal reinforcement in a structural wall with low shear to moment ratio resists shear through truss action, with the horizontal bars acting like the stirrups in a beam. Thus, the horizontal bars provided for shear reinforcement must be developed within the confined core of the boundary element and extended as close to the end of the wall as cover requirements and proximity of other reinforcement permit. The requirement that the horizontal web reinforcement be anchored within the confined core of the boundary element and extended to within 6 in. from the end of the wall applies to all horizontal bars whether straight, hooked, or headed, as illustrated in Fig. R21.9.6.4 R20.10.6.4.1.

Cyclic load reversals may lead to buckling of boundary longitudinal reinforcement even in cases where the demands on the boundary of the wall do not require special boundary elements. For walls with moderate amounts of boundary longitudinal reinforcement, ties are required to inhibit buckling. The longitudinal reinforcement ratio is intended to include only the reinforcement at the wall boundary as indicated in Fig. R21.9.6.5. A larger spacing of ties relative to 21.9.6.4(c) 20.10.6.4(c) is allowed due to the lower deformation demands on the walls. Requirements of 20.10.6.5 apply over the entire wall height and are summarized in Figure R21.9.6.5.
The addition of hooks or U-stirrups at the ends of horizontal wall reinforcement provides anchorage so that the reinforcement will be effective in resisting shear forces. It will also tend to inhibit the buckling of the vertical edge reinforcement. In walls with low in-plane shear, the development of horizontal reinforcement is not necessary.

**R20.10.6.4.2** for cases where special boundary elements are required.*<sup>20.77</sup> <<CH30>>

Diagonal bars should be placed approximately symmetrically in the beam cross section, in two or more layers. The diagonally placed bars are intended to provide the entire shear and corresponding moment strength of the beam; designs deriving their moment strength from combinations of diagonal and longitudinal bars are not covered by these provisions.

Two confinement options are described. According to *21.9.7.4(e) 20.10.7.4(e)*, each diagonal element consists of a cage of longitudinal and transverse reinforcement as shown in Fig. *R21.9.7(a) R20.10.7(a)*. Each cage contains at least four diagonal bars and confines a concrete core. The requirement on side dimensions of the cage and its core is to provide adequate toughness and stability to the cross section when the bars are loaded beyond yielding. The minimum dimensions and required reinforcement clearances may control the wall width. Revisions were made in the 2008 Code to relax spacing of transverse reinforcement confining the diagonal bars, to clarify that confinement is required at the intersection of the diagonals, and to simplify design of the longitudinal and transverse reinforcement around the beam perimeter; beams with these new details are expected to perform acceptably. *The expressions for transverse reinforcement $A_{sh}$ are based on ensuring compression capacity of an equivalent column section is maintained after spalling of cover concrete.* <<CH031>>

Section *21.9.7.4(d) 20.10.7.4(d)* describes a second option for confinement of the diagonals introduced in the 2008 Code (Fig. *R21.9.7(b) R20.10.7(b)*). This second option is to confine the entire beam cross section instead of confining the individual diagonals. This option can considerably simplify field placement of hoops, which can otherwise be especially challenging where diagonal bars intersect each other or enter the wall boundary.

When for coupling beams are not used as part of the lateral-force-resisting system, the requirements for diagonal
reinforcement may be waived.

Test results demonstrate that beams reinforced as described in Section 21.9.7.20.10.7 have adequate ductility at sheaf forces exceeding \( 10 \sqrt{f'_{c}} b_{w}d \). Consequently, the use of a limit of \( 10 \sqrt{f'_{c}} A_{s} \) provides an acceptable upper limit.

**R20.10.8 — Wall piers**

Door and window placements in structural walls sometimes lead to narrow vertical wall segments that are considered to be wall piers. The dimensions defining wall piers are given in 2.2.2.3. Shear failures of wall piers have been observed in previous earthquakes. The intent of 21.9.8.20.10.8 is to provide sufficient shear strength to wall piers so that inelastic response, if it occurs, will be primarily in flexure, a flexural yielding mechanism will develop. The provisions apply to wall piers designated as part of the seismic-force-resisting system. Provisions for wall piers not designated as part of the seismic-force-resisting system are given in 21.13.20.14. The effect of all vertical wall segments on the response of the structural system, whether designated as part of the seismic-force-resisting system or not, should be considered as required by 21.1.2.20.2.2.

Wall piers having \( (\ell_{w}/b_{w}) \leq 2.5 \) behave essentially as columns. Section 21.9.8.20.10.8.1 requires that such members satisfy reinforcement and shear strength requirements of 21.6.3.20.7.4 through 21.6.5.20.7.6. Alternative provisions are provided for wall piers having \( (\ell_{w}/b_{w}) > 2.5 \).

The design shear force determined according to 21.6.5.20.7.6.1 may be unrealistically large in some cases. As an alternative, 21.9.8.1(a)20.10.8.1(a) permits the design shear force to be determined using factored load combinations in which the earthquake effect has been amplified to account for system overstrength. Documents such as the NEHRP provisions, ASCE/SEI 7, and the International Building Code represent the amplified earthquake effect using the factor \( \Omega_{e} \).

Section 21.9.8.220.10.8.2 addresses wall piers at the edge of a wall. Under in-plane shear, inclined cracks can propagate into segments of the wall directly above and below the wall pier. Unless there is sufficient reinforcement in the adjacent wall segments, shear failure within the adjacent wall segments can occur. The length of embedment of the provided reinforcement into the adjacent wall segments should be determined considering both development length requirements and shear strength of the wall segments. See Fig. 21.9.8.R20.10.8.

**R20.11 — Special structural walls constructed using precast concrete**

Experimental and analytical studies have demonstrated that some types of precast structural walls post-tensioned with unbonded tendons, and not satisfying the prescriptive requirements of Chapter 21–20, provide satisfactory seismic performance characteristics. ACI ITG-5.1 defines a protocol for establishing a design procedure, validated by analysis and laboratory tests, for such walls, with or without coupling beams.
ACI ITG-5.2\(^{20,57}\) defines design requirements for one type of special structural wall constructed using precast concrete and unbonded post-tensioning tendons, and validated for use in accordance with 24.10.3–20.11.2.2.

**R20.12 — Structural diaphragms and trusses**

**R20.12.1 — Scope**

Diaphragms as used in building construction are structural elements (such as a floor or roof) that provide some or all of the following functions:

(a) Support for building elements (such as walls, partitions, and cladding) resisting horizontal forces but not acting as part of the seismic-force-resisting system;

(b) Transfer of lateral forces from the point of application to the vertical elements of the seismic-force-resisting system;

(c) Connection of various components of the vertical seismic-force-resisting system with appropriate strength, stiffness, and toughness so the building responds as intended in the design.\(^{20,58}\)

**R20.12.2 — Design forces**

In the general building code earthquake design forces for floor and roof diaphragms typically are not computed directly during the lateral-force analysis that provides story forces and story shears. Instead, diaphragm design forces at each level are computed by a formula that amplifies the story forces recognizing dynamic effects and includes minimum and maximum limits. These forces are used with the governing load combinations to design diaphragms for shear and moment.

For collector elements, general building codes in use the general building code in the U.S. specify load combinations that amplify earthquake forces by a factor \(\Omega\). The forces amplified by \(\Omega\) are also used for the local diaphragm shear forces resulting from the transfer of collector forces, and for local diaphragm flexural moments resulting from any eccentricity of collector forces. The specific requirements for earthquake design forces for diaphragms and collectors depend on which edition of the general building code is used. The requirements may also vary according to the SDC Seismic Design Category.

For most concrete buildings subjected to inelastic earthquake demands, it is desirable to limit inelastic behavior of floor and roof diaphragms under the imposed earthquake forces and deformations. It is preferable for inelastic behavior to occur only in the intended locations of the vertical earthquake-force-resisting system that are detailed for ductile response, such as in the beam plastic hinges of special moment frames, or in flexural plastic hinges at the base of structural walls or in coupling beams. For buildings without long diaphragm spans between lateral-force-resisting elements, elastic diaphragm behavior is typically not difficult to achieve. For buildings where diaphragms could reach their flexural or shear strength before yielding occurs in the vertical seismic force-resisting system, designers should consider providing increased diaphragm strength.
R20.12.3 — Seismic load force path

R20.12.3.2 — Section 21.11.3.2-20.12.3.2 This provision applies to strut-like elements that often are present occur around openings, diaphragm edges, or other discontinuities in diaphragms. Figure R21.11.3.2-R20.12.3.2 shows an example. Such elements can be subjected to earthquake axial forces in combination with bending and shear from earthquake or gravity loads.

R20.12.4 — Cast-in-place composite-topping slab diaphragms

A bonded topping slab is required so that the floor or roof system can provide restraint against slab buckling. Reinforcement is required to ensure the continuity of the shear transfer across precast joints. The connection requirements are introduced to promote a complete system with necessary shear transfers.

R20.12.5 — Cast-in-place noncomposite topping slab diaphragms

Composite action between the topping slab and the precast floor elements is not required, provided that the topping slab is designed to resist the design seismic earthquake forces.

R20.12.6 — Minimum thickness of diaphragms

The minimum thickness of concrete diaphragms reflects current practice in joist and waffle systems and composite topping slabs on precast floor and roof systems. Thicker slabs are required when if the topping slab is not designed to does not act compositely with the precast system to resist the design seismic earthquake forces.

R20.12.7 — Reinforcement

R20.12.7.1 — Minimum reinforcement ratios for diaphragms correspond to the required amount of temperature and shrinkage reinforcement (7.12-10.4). The maximum spacing for web reinforcement is intended to control the width of inclined cracks. Minimum average prestress requirements (7.12.3 10.4.4.1) are considered to be adequate to limit the crack widths in post-tensioned floor systems; therefore, the maximum spacing requirements do not apply to these systems.

The minimum spacing requirement for welded wire reinforcement in topping slabs on precast floor systems (see 21.11.7.1) is to avoid fracture of the distributed reinforcement during an earthquake. Cracks in the topping slab open immediately above the boundary between the flanges of adjacent precast members, and the wires crossing those cracks are restrained by the transverse wires. Therefore, all the deformation associated with cracking should be accommodated in a distance not greater than the spacing of the transverse wires. A minimum spacing of 10 in. for the transverse wires is required in 21.11.7.1-20.12.7.1 to reduce the likelihood of fracture of the wires crossing the critical cracks during a design earthquake. The minimum spacing requirements do not apply to diaphragms reinforced with individual bars, because strains are distributed over a longer length.
**R20.12.7.3** — Bar development and lap splices are designed according to requirements of Chapter 21 for reinforcement in tension. Reductions in development or splice length for calculated stresses less than $f_y$ are not permitted, as indicated in 22.4.3.10.2.

**R20.12.7.5** — In documents such as the NEHRP provisions, ASCE/SEI 7, the International Building Code, and the Uniform Building Code, collector elements of diaphragms are designed for forces amplified by a factor, $\Omega_o$, to account for the overstrength in the vertical elements of the seismic-force-resisting systems. The amplification factor $\Omega_o$ ranges between 2 and 3 for most concrete structures, depending on the document selected and on the type of seismic force-resisting system. In some documents, the factor can be calculated based on the maximum forces that can be developed by the elements of the vertical seismic-force-resisting system.

Compressive stress calculated for the factored forces on a linearly elastic model based on gross section of the structural diaphragm is used as an index value to determine whether confining reinforcement is required. A calculated compressive stress of $0.2f'_c$ in a member, or $0.5f'_c$ for forces amplified by $\Omega_o$, is assumed to indicate that integrity of the entire structure depends on the capability of that member to resist substantial compressive force under severe cyclic loading. Therefore, transverse reinforcement is required in such members at such locations to provide confinement for the concrete and the reinforcement.

**R20.12.7.6** — This section 21.11.7.6 is intended to reduce the possibility of bar buckling and provide adequate bar development conditions in the vicinity of splices and anchorage zones.

**R20.12.8** — **Flexural strength**

Flexural strength for diaphragms is calculated using the same assumptions as for walls, columns, or beams. The design of diaphragms for flexure and other actions uses the applicable load combinations of 9.2.3.1 to consider earthquake forces acting concurrently with gravity or other loads.

The influence of slab openings on flexural and shear strength is to be considered, including evaluating the potential critical sections created by the openings. Strut-and-tie models are potentially useful for designing diaphragms with openings.

Earlier design practice assumed design moments for structural diaphragms were resisted entirely by chord forces acting at opposite edges of the diaphragm. This idealization was implicit in earlier versions of the Code, but has been replaced by an approach in which all longitudinal reinforcement, within the limits of 21.11.7, is assumed to contribute to the flexural strength of the diaphragm. This change reduces the required area of longitudinal reinforcement concentrated near the edge of the diaphragm, but should not be interpreted as a requirement to eliminate all boundary reinforcement.
The shear strength requirements for diaphragms are similar to those for slender structural walls and are based on the shear provisions for beams. The term $A_{cv}$ refers to the gross area of the diaphragm, but may not exceed the thickness times the width of the diaphragm. This corresponds to the gross area of the effective deep beam that forms the diaphragm. Distributed slab reinforcement, $\rho_s$, used to calculate shear strength of a diaphragm in Eq. (21-10) is positioned perpendicular to the diaphragm flexural reinforcement. Section 21.11.9.2-20.12.9.2 limits the maximum shear strength of the diaphragm.

In addition to satisfying the provisions in 21.11.9.1 and 21.11.9.2, cast-in-place topping slab diaphragms must also satisfy 21.11.9.4, 20.12.9.1, and 20.12.9.2. Cast-in-place topping slabs on a precast floor or roof system tend to have shrinkage cracks that are aligned with the joints between adjacent precast members. Therefore, the additional shear strength requirements for topping slab diaphragms in 21.11.9.3-20.12.9.3 are based on a shear friction model, and the assumed crack plane corresponds to joints in the precast system along the direction of the applied shear, as shown in Fig. R11.6.4-R17.6. The coefficient of friction, $\mu$, in the shear friction model is taken equal to 1.0 for normalweight concrete due to the presence of these shrinkage cracks.

Both distributed and boundary reinforcement in the topping slab may be considered as shear friction reinforcement, $A_{rf}$. Boundary reinforcement within the diaphragm was called chord reinforcement in ACI 318 before 2008. Although the boundary reinforcement also resists flexural forces due to moment and axial force in the diaphragm, the reduction in the shear friction resistance in the tension zone is offset by the increase in shear friction resistance in the compression zone. Therefore, the area of boundary reinforcement used to resist shear friction need not be added to the area of boundary reinforcement used to resist moment and axial force flexural forces. The distributed topping slab reinforcement must contribute at least one-half of the nominal shear strength. It is assumed that connections between the precast elements do not contribute to the shear strength of the topping slab diaphragm.

Section 21.11.9.4-20.12.9.4 limits the maximum shear that may be transmitted by shear friction within a topping slab diaphragm.

The expressions for transverse reinforcement $A_{sh}$ are based on ensuring compression capacity of an equivalent column section is maintained after spalling of cover concrete. <<CH031>>

Requirements for foundations supporting buildings assigned to SDC Seismic Design Category D, E, or F were
added to the 1999 Code. They represent a consensus of a minimum level of good practice in designing and
detailing concrete foundations including piles, drilled piers, and caissons. It is desirable that inelastic response in
strong ground shaking motion occurs above the foundations, as repairs to foundations can be extremely difficult and
expensive.

R20.13.2 — Footings, foundation mats, and pile caps

R20.13.2.2 — Tests have demonstrated that flexural members terminating in a footing, slab, or beam (a T-
joint) should have their hooks turned inward toward the axis of the member for the joint to be able to resist the
flexure in the member forming the stem of the T.

R20.13.2.3 — Columns or boundary members supported close to the edge of the foundation, as often occurs near
property lines, should be detailed to prevent an edge failure of the footing, pile cap, or mat.

R20.13.2.4 — The purpose of this section is to emphasize that top reinforcement should be
provided as well as other required reinforcement.

R20.13.2.5 — Committee 318 recommends that foundation or basement walls be reinforced in buildings assigned
to SDC Seismic Design Category D, E, or F.

R20.13.3 — Grade beams and slabs-on-ground

For seismic earthquake conditions, slabs-on-ground (soil-supported slabs) are often part of the lateral-force-resisting
system and should be designed in accordance with this Code as well as other appropriate standards or guidelines.
See 1.1.7-1.4.4.

R20.13.3.2 — Grade beams between pile caps or footings can be separate beams beneath the slab-on-ground or
can be a thickened portion of the slab-on-ground. The cross-sectional limitation and minimum tie requirements
provide reasonable proportions.

R20.13.3.3 — Grade beams resisting seismic flexural stresses from column moments should have reinforcement
details similar to the beams of the frame above the foundation.

R20.13.3.4 — Slabs-on-ground often act as a diaphragm to hold the building together at the ground level and
minimize the effects of out-of-phase ground motion that may occur over the footprint of the building. In these cases, the
slab-on-ground should be adequately reinforced and detailed. The contract construction documents should clearly
state that these slabs-on-ground are structural members so as to prohibit sawcutting of the slab.

R20.13.4 — Piles, piers, and caissons

Adequate performance of piles and caissons for seismic loadings earthquake effects requires that these provisions be
met in addition to other applicable standards or guidelines. See R1.1.6. R1.4.5.

R20.13.4.1 — A load path is necessary at pile caps to transfer tension forces from the reinforcing bars in the column or boundary member through the pile cap to the reinforcement of the pile or caisson.

R20.13.4.2 — Grouted dowels in a blockout in the top of a precast concrete pile need to be developed, and testing is a practical means of demonstrating strength. Alternatively, reinforcing bars can be cast in the upper portion of the pile, exposed by chipping of concrete and mechanically spliced or welded to an extension.

R20.13.4.3 — During earthquakes, piles can be subjected to extremely high flexural demands at points of discontinuity, especially just below the pile cap and near the base of a soft or loose soil deposit. The Code requirement for confinement reinforcement at the top of the pile is based on numerous failures observed at this location in earthquakes. Transverse reinforcement is required in this region to provide ductile performance. Possible inelastic action in the pile at abrupt changes in soil deposits should also be considered, such as changes from soft to firm or loose to dense soil layers. Where precast piles are to be used, the potential for the pile tip to be driven to an elevation different than that specified in the contract construction documents needs to be considered when detailing the pile. If the pile reaches refusal at a shallower depth, a longer length of pile will need to be cut off. If this possibility is not foreseen, the length of transverse reinforcement required by 21.12.4.4, R20.13.4.3 may not be available after the excess pile length is cut off.

R20.13.4.6 — Extensive structural damage has often been observed at the junction of batter piles and the buildings. The pile cap and surrounding structure should be designed for the potentially large forces that can be developed in batter piles.

R20.14 — Members not designated as part of the seismic-force-resisting system

This Section 20.14 This section applies only to structures assigned to SDC Seismic Design Category D, E, or F. For those Seismic Design Categories, model building codes, such as the 2006 IBC, require that all structural members not designated as a part of the seismic-force-resisting system to are required to be designed to support gravity loads while subjected to the design displacement. For concrete structures, the provisions of 21.13-20.14 satisfy this requirement for columns, beams, slabs, and wall piers of the gravity system. Section 20.14.2.1 defines the load and displacement combinations that must be considered.

The design displacement is defined in 2.2.2.3. Models used to determine design displacement of buildings should be chosen to produce results that conservatively bound the values expected during the design earthquake and should include, as appropriate, effects of concrete cracking, foundation flexibility, and deformation of floor and roof diaphragms.

The provisions of 21.13-20.14 are based on the principle enabling intended to enable ductile flexural yielding of columns, beams, slabs, and wall piers under the design displacement, by providing sufficient confinement and
shear strength in elements that yield.

R20.14.3 — By the provisions of 20.14.3, 20.14.4, and 20.14.6, cast-in-place columns and beams, and wall piers, respectively, are assumed to yield if the combined effects of factored gravity loads and design displacements exceed the strengths specified in those provisions, or if the effects of design displacements are not calculated. Requirements for transverse reinforcement and shear strength vary with the axial load on the member and whether the member yields under the design displacement.

R20.14.4 — Damage to some buildings with precast concrete gravity systems during the 1994 Northridge earthquake was attributed to several factors addressed in 21.13.5. Columns should contain ties over their entire height, frame members not proportioned to resist earthquake forces should be tied together, and longer bearing lengths should be used to maintain integrity of the gravity system during shaking ground motion. The 2 in. increase in bearing length is based on an assumed 4 percent story drift ratio and 50 in. beam depth, and is considered to be conservative for the ground motions expected for structures assigned to SDC Seismic Design Category D, E, or F. In addition to the provisions of 21.13.5, 20.14.4, precast frame members assumed not to contribute to lateral resistance should also satisfy 21.13.2 through 21.13.4 the requirements for cast-in-place construction addressed in 20.14.3, as applicable.

R20.14.5.1 — Provisions for shear reinforcement at slab-column connections were added in 2005 are intended to reduce the likelihood of slab punching shear failure. The shear reinforcement is required unless either 21.13.6(a) or (b) is satisfied, if the design story drift ratio exceeds the value specified.

Section 21.13.6(a) 20.14.5.1(a) requires calculation of shear stress due to the factored shear force and induced moment according to 11.11.7.2. The induced moment is the moment that is calculated to occur at the slab-column connection when subjected to the design displacement. Section 13.5.1.2 and the accompanying Commentary provide guidance on selection of the stiffness of the slab-column connection for the purpose of this calculation.

Section 21.13.6(b) 20.14.5.1(b) does not require the calculation of induced moments, and is based on research that identifies the likelihood of punching shear failure considering the story drift ratio and shear stress due to gravity loads without moment about the slab critical section. Figure R21.13.6 illustrates the requirement. The requirement can be satisfied by adding slab shear reinforcement, increasing slab thickness, changing the design to reduce the design story drift ratio, or a combination of these.

If column capitals, drop panels, shear caps, or other changes in slab thickness are used, the requirements of 21.13.6 20.14.5 are evaluated at all potential critical sections, as required by 11.11.1.2.

R20.14.6 — Section 21.9.8 20.10.8 requires that the design shear force be determined according to 20.6.1, which in some cases may result in unrealistically large forces. As an alternative, the design shear force can be determined as the product of an overstrength factor and the shear induced when the wall pier is displaced by $\delta_u$. 

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The overstrength factor $\Omega_o$ included in documents such as the NEHRP provisions, ASCE/SEI 7, and the International Building Code can be used for this purpose.

### TABLE R20.2 — SECTIONS OF CHAPTER 21 TO BE SATISFIED IN TYPICAL APPLICATIONS

<table>
<thead>
<tr>
<th>Component resisting earthquake effect, unless otherwise noted</th>
<th>Seismic Design Category</th>
</tr>
</thead>
<tbody>
<tr>
<td>Analysis and design requirements</td>
<td>A (None) 21.1.2, 20.2.2 B (21.1.4, 20.2.1.3) 21.1.2, 20.2.2 C (21.1.5, 20.2.1.4) 21.1.2, 20.2.2 D, E, F (21.1.6, 20.2.1.5) 21.1.2, 20.2.2</td>
</tr>
<tr>
<td>Materials</td>
<td>None None None</td>
</tr>
<tr>
<td>Frame members</td>
<td>None 21.4, 20.5 21.4, 20.5 21.4, 20.5 21.4, 20.5</td>
</tr>
<tr>
<td>Structural walls and coupling beams</td>
<td>None None None</td>
</tr>
<tr>
<td>Structural diaphragms and trusses</td>
<td>None None None</td>
</tr>
<tr>
<td>Foundations</td>
<td>None None None</td>
</tr>
<tr>
<td>Frame members not designated as part of the seismic-force-resisting system proportioned to resist forces induced by earthquake motions</td>
<td>None None None</td>
</tr>
<tr>
<td>Anchors</td>
<td>None 21.1.8, 20.2.3 21.1.8, 20.2.3</td>
</tr>
</tbody>
</table>

*In addition to requirements of Chapters 1 through 19, 21 to 23, and ACI 318.1, except as modified by Chapter 21 or Section 22.10. Also applies in SDC Seismic Design Category D, E, and F.

† As permitted by the legally adopted general building code of which this Code forms a part.
### Table R20.10.1 — Governing Design Provisions for Vertical Wall Segments

<table>
<thead>
<tr>
<th>Clear height of vertical wall segment / length of vertical wall segment, ((h_w/l_w))</th>
<th>Length of vertical wall segment / Wall thickness, ((l_w/b_w))</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>((l_w/b_w)) (\leq 2.5)</td>
<td>(2.5 &lt; (l_w/b_w) \leq 6.0)</td>
<td>((l_w/b_w) &gt; 6.0)</td>
</tr>
<tr>
<td>(h_w/l_w &lt; 2.0)</td>
<td>Wall</td>
<td>Wall</td>
</tr>
<tr>
<td>(h_w/l_w \geq 2.0)</td>
<td>Wall pier required to satisfy specified column design requirements, see 21.9.8.1-20.10.8.1</td>
<td>Wall pier required to satisfy specified column design requirements or alternative requirements, see 21.9.8.1-20.10.8.1</td>
</tr>
</tbody>
</table>

* \(h_w\) is the clear height, \(l_w\) is the horizontal length, and \(b_w\) is the width of the web of the wall segment.
Fig. R20.4.2—Design shears for intermediate moment frames.
Fig. R20.4.5.1—Effective width for reinforcement placement in edge and corner connections.
Revised to include CH030, CH031, CH034, and CH036

**Fig. R20.4.5.2**—Location of reinforcement in slabs. <<Modify 21.3.6.1 to 20.4.5.1, 21.3.6.2 to 20.4.5.2, 21.3.6.3 to 20.4.5.3.>>

**Fig. R20.4.5.3**—Arrangement of reinforcement in slabs. <<Modify 21.3.6.4 to 20.4.5.4, 21.3.6.6 to 20.4.5.6, 21.3.6.7 to 20.4.5.7.>>
Fig. R20.6.2—Maximum effective width of wide beam and required transverse reinforcement.
Fig. R20.6.4—Examples of overlapping hoops and illustration of limit on maximum horizontal spacing of supported longitudinal bars. <<Update reference to 21.1 in the figure.>>
Notes on Fig. R21.5.4:

1. Direction of shear force $V_o$ depends on relative magnitudes of gravity loads and shear generated by end moments.

2. End moments $M_{pr}$ based on steel tensile stress of 1.25 $f_y$, where $f_y$ is specified yield strength. (Both end moments should be considered in both directions, clockwise and counter-clockwise).

3. End moment $M_{pr}$ for columns need not be greater than moments generated by the $M_{pr}$ of the beams framing into the beam-column joints. $V_o$ should not be less than that required by analysis of the structure.

Fig. R20.6.5—Design shears for beams and columns. <<In the note at the top of the figure, change 21.5.4 to 20.6.5.

In Note 2, change steel to reinforcement. >>
Consecutive crossties engaging the same longitudinal bar have their 90-degree hooks on opposite sides of column.

Fig. R20.7.5.2—Example of transverse reinforcement in columns. <<Update 21-2 to 20.7.5.3>>

The dimension $x_i$ from centerline to centerline of tie legs is not to exceed 14 inches. The term $h_x$ used in equation 21-2 is taken as the largest value of $x_i$.

Fig. R20.8.4—Effective joint area. <<Update 21.7.3.2 to 20.8.3.2 and 21.7.4.1 to 20.8.4.1>>
Fig. R20.9.2.2—Strong connection examples.
Fig. R20.10.4.5—Wall with openings.

Option with standard boundary element reinforcement, $A_{sh}$

Option with straight developed reinforcement

<<Numbering to be resolved with staff.>>

<<Revise Figure R20.10.6.4 as shown.>>
Fig. R20.10.6.4.1—Development of wall horizontal reinforcement in confined boundary element.

(a) Wall with \( h_w/l_w \geq 2.0 \) and a single critical section controlled by flexure and axial load designed using 20.10.6.2, 20.10.6.4, and 20.10.6.5

- \( \rho < 400/f_y \)
- \( \rho \geq 400/f_y \)

\[ \max \left( \frac{l_d}{4V_{ed, critical section}} \right) \]
\( \geq h_u/16 \) if \( c/l_u \geq 3/8 \), then \( b \geq 12 \) in.

(b) Option with standard hooks or headed reinforcement

- Ties not required
- Ties per 20.10.6.5
- Special boundary element \( \geq 12 \) in.
- \( \geq l_d \) for \( 1.25f_y \) (or hook as req’d.)

(boundary element not near edge of footing)

(boundary element near edge of footing or other support)
(b) Wall and wall pier designed using 20.10.6.3, 20.10.6.4, and 20.10.6.5

Notes: Requirement for special boundary element is triggered if maximum extreme fiber compressive stress $\sigma \geq 0.2 f'_c$. Once triggered, the special boundary element extends until $\sigma < 0.15 f'_c$. Since $h_w/l_w \leq 2.0$, 20.10.6.4(c) does not apply.

Fig. R.20.10.6.4.2 Summary of requirements for special walls

Fig. R.20.10.6.5—Longitudinal reinforcement ratios for typical wall boundary conditions.
(a) Confine of individual diagonals.

Note: For clarity in the elevation view, only part of the total required reinforcement is shown on each side of the line of symmetry.

(b) Full confinement of diagonally reinforced concrete beam section. Mod the note on Section BB to read: Successive crossties arranged such that 180-degree hooks alternate end for end around the cross section and along the length of the beam.

Fig. R20.10.7—Coupling beams with diagonally oriented reinforcement. Wall boundary reinforcement shown on one side only for clarity.
Fig. R20.10.8—Required horizontal reinforcement in wall segments above and below wall piers at the edge of a wall.

Fig. R20.12.3.2—Example of diaphragm subject to the requirements of 20.12.3.2 and showing an element having confinement as required by 20.12.7.5.
Fig. R20.14.5.1—Illustration of the criterion of 20.14.5.1(b). Modify label on horizontal axis to read $v_u/\phi V_c$.>