Chapter 6

CONCRETE STRUCTURE DESIGN REQUIREMENTS

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

Ref. 6-1 Building Code Requirements for Reinforced Concrete, American Concrete Institute, ACI 318-89 (Revised 1992), excluding Appendix A

Ref. 6-2 Building Code Requirements for Structural Plain Concrete, ACI 318.1-89 (Revised 1992)

6.1.1 MODIFICATIONS TO REF. 6-1:

6.1.1.1: Replace Sec. 9.2.3 with Sec. 2.2.6 of this document.

6.1.1.2: Insert the following definitions in Sec. 21.1:

"Connection - Interface where two precast concrete elements or a precast concrete element and a cast-in-place element are interconnected.

"Coupling Beam - A horizontal element in plane with and connecting two adjacent structural walls.

"Joint - For monolithic reinforced concrete structures and precast concrete structural systems satisfying 21.2.2.5 the joint is the geometric volume common to the intersecting members. Effective area of joint for frame for shear is defined in 21.5.3.

"Joint Region - Frames - The joint plus the volumes within lengths equal to twice the intersecting members' depths measured from the faces of the joint.

"Joint Region - Panels - The joint plus the volumes within lengths equal to twice the intersecting members' thickness measured from the faces of the joint.

"Non-Linear Action Location - Center of plastic hinge zone, line of shear slip or midpoint of extending element."
6.1.1.3: Replace Sec. 21.2.1.3 and 21.2.1.4 with the provisions of this chapter.

6.1.1.4: Insert the following new Sec. 21.2.1.6:

"A reinforced concrete structural system composed of precast elements shall be permitted for the seismic force resisting system if:

"1. It emulates the behavior of monolithic reinforced concrete construction and satisfies 21.2.2.5, or

"2. It relies on the unique properties of a structural system composed of interconnected precast elements and it is demonstrated by experimental evidence or analysis to safely sustain the seismic loading requirements of a comparable monolithic reinforced concrete structure satisfying Chapter 21."

6.1.1.5: Insert the following new Sec. 21.2.2.5, 21.2.2.6 and 21.2.2.7:

"21.2.2.5 Structural systems emulating the behavior of reinforced concrete construction and composed of precast concrete elements shall satisfy 21.2.2.6 or 21.2.2.7.

"21.2.2.6 Precast concrete structural systems shall utilize strong connections resulting in nonlinear response remote from those connections. Designs shall satisfy the requirements of 21.2.7 in addition to the requirements of 21.2 through 21.6, as applicable. For 21.4 the restrictions on cross-sectional dimensions and clear span to effective depth shall also apply to the dimensions and span between nonlinear response locations.

"21.2.2.7 If 21.2.2.6 is not satisfied systems shall utilize connections that result in joint regions having characteristics providing performance for the structure equal to or exceeding that for a comparable monolithic reinforced concrete structure. In addition, designs shall satisfy the requirements of 21.2.8 as well as the requirements of 21.2 through 21.6, as applicable."

6.1.1.6: Insert the following new Sec. 21.2.5.2 and 21.2.5.3:

"21.2.5.2 Prestressing tendons shall be permitted in flexural members of frames provided the average prestress, $f_{pc}$, calculated for an area equal to the member's shortest cross-sectional dimension multiplied by the perpendicular dimension, does not exceed the greater of 350 psi or $f'/12$ at locations of nonlinear action.

"21.2.5.3 For members in which prestressing tendons are used together with ASTM A706 or A615 (Grades 40 or 60) reinforcement to resist earthquake-induced forces, prestressing tendons shall not provide more that one quarter of the strength for both positive moments and negative moments at the joint face. Anchorages for tendons must be demonstrated to perform satisfactorily for seismic loadings. Anchorages assemblies shall withstand, without failure, a minimum of 50 cycles of loading ranging
between 40 and 85 percent of the minimum specified tensile strength of the tendon. Tendons shall extend through exterior joints and be anchored at the exterior face or beyond."

6.1.1.7: Insert the following new Sec. 21.2.7:

"21.2.7 Members resisting earthquake induced forces in precast concrete frames using strong connections shall satisfy the following conditions:

"21.2.7.1 Locations for non-linear action shall be selected so that there is a strong column/weak beam deformation mechanism under earthquake effects. For beam to continuous column connections, the locations shall not be closer to the column than three quarters of the depth of the beam. For beam to beam connections, the location shall be permitted to be anywhere within the flexural members of the frames but shall be no closer to the connection than three quarters of the depth of the beam. For column to continuous beam connections and column to column connections, locations shall be permitted to be anywhere within the beam length outside the joint. For column to footing connections where energy dissipation is required at the column base to complete the non-linear deformation mechanism, the locations for non-linear action shall be no closer to the footing than three quarters of the width of the column in the direction of the mechanism considered.

"21.2.7.2 Design strength of cross sections of connections shall be based on

\[
\phi S_{n,\text{CONNECTION}} \geq 11 \frac{S_{n,\text{FRAME}}}{10}
\]  \hspace{1cm} (21-A)

where \(S_{n,\text{CONNECTION}}\) is nominal strength (moment or shear) at connection cross-section, \(\phi\) is strength reduction factor for that force, and \(S_{n,\text{FRAME}}\) is the force, (moment or shear), determined from consideration of the probable resistances at the locations for non-linear action required by 21.2.7.1.

"21.2.7.3 In addition to 21.2.7.2 at column to column connections the design moment strength \(\phi M_{n}\) shall not be less than 0.4 times the maximum \(M_{pr}\) for the column within the story height."

6.1.1.8: Insert the following new Sec. 21.2.8

"21.2.8 Precast concrete frame and wall systems designed using 21.2.2.7 shall satisfy the following conditions:

"21.2.8.1 The deformed shape of the structure under specified lateral loads shall emulate that for the same structure constructed in monolithic reinforced concrete.
"21.2.8.2 The main longitudinal reinforcement of frame members, and the boundary reinforcement for walls, shall be made continuous across connections and able to develop a stress of at least 1.25f_y in tension and compression.

"21.2.8.3 Design of connection cross sections shall be based on the assumption that the capacity for connection moment, M_{pr}, is its probable strength in flexure or the capacity for connection shear, S_{pr}, is its probable strength in shear.

"21.2.8.4 When connection capacity is M_{pr} the co-existing applied shear, S, must be not greater than \( \phi S_n \text{CONN}\) times a non-linear action modification factor, 0.5 \( \delta_s \), where \( \delta_s \) is a factor for reversed cyclic loading, varying linearly between 1.0 for a shear that reverses to not greater than 50 percent of its maximum value and 4/5 for a shear that fully reverses.

"21.2.8.5 When connection capacity is S_{pr} the co-existing applied moment, M, must be not greater than \( \phi M_n \text{CONN}\).

"21.2.8.6 Where the connection is subject to an axial force in addition to shear and moment that axial force shall be considered in computing the probable and nominal strengths in 21.2.8.4 and 21.2.8.5.

"21.2.8.7 The probable moment strength M_{pr} of the connection shall be determined using a strength reduction factor of 1.0 and a reinforcing steel stress of at least 1.25 f_y.

"21.2.8.8 The probable shear strength S_{pr} for shear slip shall be taken as the strength calculated from Sec. 11.7 times \( \delta_s \).

"21.2.8.9 The probable and nominal shear strengths for the connection shall be less than the corresponding shear strengths immediately adjacent to the connection of the members meeting at the connection."

6.1.1.9: Change Sec. 21.3.3.4 to read as follows:

"Where hoops are not required, stirrups with 135-degree or greater hooks with 6-bar-diameter but not less than 3-inch (75 mm) extensions shall be spaced not more than d/2 throughout the length of the member."

6.1.1.10: Add the following new Sec. 21.4.5.3:

"At any section where the design strength, \( \phi P_m \), of the column is less than the sum of the shear \( V_e \) computed in accordance with Sec. 21.4.5.1 for all the beams framing into the column above the level under consideration, special transverse reinforcement shall be provided. For beams framing into opposite sides of the column, the moment components may be assumed to be of opposite sign. For determination of the nominal
strength, \( P_n \), of the column, these moments may be assumed to result from the deformation of the frame in any one principal axis."

6.1.1.11: Change the reference to Sec. 9.2 in Sec. 21.6.3 to the load combination specified in Sec. 2.2.6 of this document for earthquake forces.

6.1.1.12: Add a new Sec. 21.6.6 as follows and renumber existing Sec. 21.6.6 through 21.6.8 to 21.6.7 through 21.6.9:

"21.6.6.1 A cast-in-place reinforced concrete slab used as a diaphragm to resist earthquake forces shall not be less than 2-inches (50 mm) thick.

"21.6.6.2 A cast-in-place reinforced topping slab bonded to a precast floor or roof system and used as a diaphragm to resist earthquake forces shall comply with all of the following:

1. The topping slab shall not be less than 2-1/2 inches (63 mm) thick;
2. Its connections shall be proportioned and detailed to provide a complete load path for transfer of shear to chords, collectors, and resisting elements.
3. Bonding of the topping slab to the precast concrete elements shall be in accordance with Sec. 11.7.9.

6.1.1.13: Add a new Sec. 21.6.7 as follows and renumber Sec. 21.6.6 through 21.6.7 as modified by Sec. 6.1.1.12 to 21.6.8 through 21.6.10.

"21.6.7 Coupling Beams:

"21.6.7.1: For coupling beams with \( l_n/d \geq 4 \), the design shall conform to the requirements of Sec. 21.2 and 21.3 for structures in Seismic Performance Category D or E or to Sec. 21.8 for structures in Seismic Performance Category C. It shall be permitted to waive the requirements of Sec. 21.3.1.3 and 21.3.1.4 if it can be shown by rational analysis that lateral stability is adequate or if alternative means of maintaining lateral stability is provided.

"21.6.7.2: Coupling beams with \( l_n/d < 4 \) shall be permitted to be reinforced with two intersecting groups of symmetrical diagonal bars. Coupling beams with \( l_n/d < 4 \) and with factored shear force \( V_u \) exceeding \( 4f_{c'}b_wd \) (the metric equivalent is \( 0.332\sqrt{f_{c'}}b_wd \) where \( f_{c'} \) is in MPa and \( b_w \) and \( D \) are in mm) shall be reinforced with two intersecting groups of symmetrical diagonal bars. Each group shall consist of a
minimum of four bars assembled in a core each side of which is a minimum of $b_w/2$. The design shear strength, $\phi V_n$, of these coupling beams shall be determined by:

$$\phi V_n = 2\phi f_y \sin \alpha A_{w\delta} \leq 10\phi \sqrt{f_c^d} b_w d$$ (21.5)

where:

$\alpha$ = the angle between the diagonal reinforcement and the longitudinal axis,

$A_{w\delta}$ = total area of reinforcement in each group of diagonal bars, and

$\phi$ = 0.85.

The metric equivalent of Eq. 21.5 is as follows:

$$0.83 \phi \sqrt{f_c^d} b_w d$$

where $f_c$ is in mm.

**EXCEPTION:** The design of coupling beams need not comply with the requirements for diagonal reinforcement if it can be shown that failure of the coupling beams will not impair the vertical load carrying capacity of the structure, the egress from the structure, or the integrity of nonstructural components and connections or produce other unacceptable effects. The analysis shall take into account the changes of stiffness of the structure due to the failure of coupling beams. Design strength of coupling beams assumed to be part of the seismic force resisting system shall not be reduced below the values otherwise required.

"21.6.7.3: Each group of diagonally placed bars shall be enclosed in transverse reinforcement conforming to Sec. 21.4.4.1 through 21.4.4.3. For the purpose of computing $A_g$ as per Eq. 10-5 and 21-3, the minimum cover as specified in Sec. 7.7 shall be assumed over each group of diagonally placed reinforcing bars.

"21.6.7.4: Reinforcement parallel and transverse to the longitudinal axis shall be provided and, as a minimum, shall conform to Sec. 10.5, 11.8.9, and 11.8.10.

"21.6.7.5: Contribution of the diagonal reinforcement to nominal flexural strength of the coupling beam area shall be considered.

6.1.1.14: Change the title of Sec. 21.8 to read: "Requirements for Intermediate Moment Frames."
6.1.2 MODIFICATIONS TO REF. 6-2:

6.1.2.1: Amend Sec. 1.2.3 to read:

"Plain concrete shall not be used for structural members where special design considerations are required for blast unless explicitly permitted by the legally adopted general building code."

6.2 BOLTS AND HEADED STUD ANCHORS IN CONCRETE: Bolts and headed stud anchors shall be solidly cast in concrete. The factored loads on embedded anchor bolts and headed stud anchors shall not exceed the design strengths determined by Sec. 6.2.2.

6.2.1 LOAD FACTOR MULTIPLIERS: In addition to the load factors in Sec. 2.2.6, a multiplier of 2 shall be used if special inspection is not provided or of 1.3 if it is provided. When anchors are embedded in the tension zone of a member, the load factors in Sec. 2.2.6 shall have a multiplier of 3 if special inspection is not provided or of 2 if it is provided.

6.2.2 STRENGTH OF ANCHORS: Strength of anchors cast in concrete shall be taken as the lesser of the strengths associated with concrete failure and the anchor steel failure. Where feasible, anchor connections, particularly those subject to seismic or other dynamic loads, shall be designed and detailed such that the connection failure is initiated by failure of the anchor steel rather than by failure of the surrounding concrete. Reinforcement also shall be permitted to be used for direct transfer of tension and shear loads. Such reinforcement shall be designed with proper consideration of its development and its orientation with respect to the postulated concrete failure planes.

The strength of headed bolts and headed studs cast in concrete shall be based on testing in accordance with Sec. 6.2.3 or calculated in accordance with Sec. 6.2.4. The bearing area of headed anchors shall be at least one and one-half times the shank area.

6.2.3 STRENGTH BASED ON TESTS: The strength of anchors shall be based on not less than 10 representative tests conforming to the proposed materials and anchor size and type, embedment length, and configuration as to attachment plates, loads applied, and concrete edge distances. The nominal strength shall be the mean value derived from the tests minus one standard deviation. The strength reduction factor applied to the nominal strength shall be 0.8 when anchor failure governs in the majority of tests and 0.65 when concrete failure controls.

6.2.4 STRENGTH BASED ON CALCULATIONS: Calculations for design strength shall be in accordance with Sec. 6.2.4.1 through 6.2.4.3.
6.2.4.1 Strength in Tension: The design tensile strength of the individual anchors or adequately connected groups of anchors shall be the minimum of $P_s$ or $\phi P_c$ where:

1. Design tensile strength governed by steel, $P_s$, in pounds (N), is:

$$P_s = 0.9 A_b F_u n$$  \hfill (6.2.4.1-1)

where:

$A_b = \text{the area, in}^2 \text{ (mm}^2\text{)} \text{ of the shank of the bolt or stud,}$

$n = \text{the number of anchors, and}$

$F_u = \text{the specified ultimate tensile strength, psi (MPa), of the anchor. A307 bolts or A108 studs are permitted to be assumed to have } F_u \text{ of 60,000 psi (414 Mpa).}$

2. Design tensile strength governed by concrete failure, $\phi P_c$ in pounds (N) is as follows:

a. For individual anchors or groups of anchors with individual anchors spaced at least twice their embedment length apart and spaced not less than one anchor embedment length from a free edge of the concrete:

$$\phi P_c = \phi \lambda \sqrt{f_c'(2.8A_s)n}$$  \hfill (6.2.4.1-2)

where:

$A_s = \text{area (in}^2\text{) of the assumed failure surface taken as a truncated cone sloping at 45 degrees from the head of the anchor to the concrete surface as shown in Figure 6.2.4.1a;}$

![Shear cone failure for a single headed anchor.](image)

**FIGURE 6.2.4.1a** Shear cone failure for a single headed anchor.
\( f'_c = \) concrete strength (psi)--6,000 psi (41 MPa) maximum;

\( \phi = \) strength reduction factor of 0.65 except that where special transverse reinforcing is provided to confine the concrete engaged by the anchor and is extended to pass through the failure surface into adjacent concrete, \( \phi \) is permitted to be taken as 0.85;

\( \lambda = \) lightweight concrete factor--1 for normal weight concrete, 0.85 for sand-lightweight concrete, and 0.75 for lightweight concrete.

The metric equivalent of Eq. 6.2.4.1-2 is:

\[
\phi P_c = \frac{\phi \lambda \sqrt{f'_c (2.8A_s)} n}{12}
\]

where \( A_s \) is in \( \text{mm}^2 \) and \( f'_c \) is MPa.

Where any anchors are closer to a free edge of the concrete than the anchor embedment length, the design tensile strength of those anchors shall be reduced proportionately to the edge distance divided by the embedment length. For multiple edge distances less than the embedment length, use multiple reductions.

b. For anchor groups where individual anchors are spaced closer together than two embedment lengths:

\[
\phi P_c = \phi \lambda \sqrt{f'_c (2.8A_p + 4A_t)} \quad \text{(6.2.4.1-3)}
\]

where:

\( A_p = \) area (in.\(^2\)) of an assumed failure surface taken as a truncated pyramid extending from the heads of the outside anchors in the group at 45 degrees to the concrete surface as shown in Figure 6.2.4.1b;

\( A_t = \) area (in.\(^2\)) of the flat bottom surface of the truncated pyramid of the assumed concrete failure surface shown in Figure 6.2.4.1b.
The metric equivalent of Eq. 6.2.4.1-3 is:

\[ \phi P_c = \frac{\phi \lambda \sqrt{f'_c(2.8A_p + 4A_t)}}{12} \]

where \( A_p \) and \( A_t \) are in mm\(^2\) and \( f'_c \) is in MPa.

If any anchors are closer to a free edge of the concrete than the anchor embedment length, the design tensile strength shall be reduced by using the reduced area \( A_p \) in the equation above.

Anchor groups shall be checked for a critical failure surface passing completely through a concrete member along the 45 degree lines as shown in Figure 6.2.4.1c with \( A_t = 0 \) and \( A_p \) based on the area of the sloping failure surface passing completely through the concrete member. The lowest allowable load shall govern.
6.2.4.2 Strength in Shear: The design shear strength of anchors shall be the minimum of $V_s$ or $\phi V_c$, where the design shear strength governed by steel failure is $V_s$, in pounds (N), and the design shear strength governed by concrete failure is $\phi V_c$, in pounds (N). In situations where the embedment and/or concrete edge distances are limited, reinforcement to confine concrete to preclude its premature failure shall be permitted.

a. Where anchors are loaded toward an edge with edge distance $d_e$ from the back row of anchors as shown in Figure 6.2.4.2 equal to or greater than 15 anchor diameters and the distance from the front row of anchors to the edge equal to or greater than 6 anchor diameters:

$$V_s = (0.75 A_b F_u) n$$

(6.2.4.2-1)

$$\phi V_c = (\phi 800 A_b \lambda \sqrt{f_{c'}}) n$$

(6.2.4.2-2)

where:

$A_b =$ the area, in.² (mm²) of the shank of the bolt or stud;

$F_u =$ the specified ultimate tensile strength (psi) of the anchor. A307 bolts or A108 studs are permitted to be assumed to have $F_u$ of 60,000 psi (414 MPa);

$n =$ the number of anchors;

$\lambda =$ lightweight concrete factor--1 for normal weight concrete, 0.85 for sand-lightweight concrete, and 0.75 for lightweight concrete; and

$f_{c'} =$ concrete strength (psi)--6,000 psi (41 MPa) maximum.

The metric equivalent of Eq. 6.2.4.2-2 is:

$$\phi V_c = \frac{(\phi 800 A_b \lambda \sqrt{f_{c'}}) n}{12}$$

where $A_b$ is in mm² and $f_{c'}$ is in MPa.

b. Where anchors are loaded toward an edge with $d_e$ less than 15 anchor diameters or the front row closer to the edge than 6 anchor diameters:

$$V_s = (0.75 A_b F_u) n_b$$

(6.2.4.2-3)
\[ \phi V_e = \phi V'_e C_w C_t C_c \quad (6.2.4.2-4) \]

where:

- \( A_b \) = the area (in.\(^2\)) of the shank of the bolt or stud.
- \( F_u \) = the specified ultimate tensile strength (psi) of the anchor. A307 bolts or A108 studs are permitted to be assumed to have \( F_u \) of 60,000 psi (414 MPa).
- \( n_b \) = the number of anchors in the back row.
- \( \phi V'_e \) = the design shear strength of an anchor in the back row:
  \[ \phi V'_e = \phi 12.5d_e^{1.5}\lambda\sqrt{f'_c} \leq 800A_b\lambda\sqrt{f'_c} \quad (6.2.4.2-5) \]
  where \( d_e \) = the distance from the anchor axis to the free edge (in.).

- \( C_w \) = the adjustment factor for group width:
  \[ C_w = 1 + \left( \frac{b}{3.5d_e} \right) \leq n_b \quad (6.2.4.2-6) \]
  where \( b \) = the center-to-center distance of outermost anchors in the back row (see Figure 6.2.4.2) (in.) and \( d_e \) = the distance from the anchor axis to the free edge (in.).

- \( C_t \) = the adjustment factor for member thickness:
  \[ C_t = \frac{h}{1.3d_e} \leq 1.0 \quad (6.2.4.2-7) \]
  where \( h \) = the thickness of concrete (in.) and \( d_e \) is as above.

- \( C_c \) = the adjustment factor for member corner effects:
  \[ C_c = 0.4 + 0.7 \left( \frac{d_c}{d_e} \right) \leq 1.0 \quad (6.2.4.2-8) \]
  where \( d_c \) = the distance, measured perpendicular to the load, from the free edge of the concrete to the nearest anchor in in. (see Figure 6.2.4.2) and \( d_e \) is as above.
The metric equivalent of Eq. 6.2.4.2-5 is:

\[ \phi V'_c = \frac{\phi 12.5d_e^{15} \lambda \sqrt{f'_c}}{2.39} \leq \frac{\phi 800A_b \lambda \sqrt{f'_c}}{12} \]

where \( d_e \) is in mm, \( A_b \) is in mm² and \( f'_c \) is in MPa.

**FIGURE 6.2.4.2** Shear on a group of headed anchors.

6.2.4.3 Combined Tension and Shear: Where tension and shear act simultaneously, all of the following conditions must be met:

\[ \frac{1}{\phi} \left( \frac{V_u}{V_c} \right) \leq 1.0 \]  \hspace{1cm} (6.2.4.3-1a)

\[ \frac{1}{\phi} \left( \frac{P_u}{P_c} \right) \leq 1.0 \]  \hspace{1cm} (6.2.4.3-1b)

\[ \frac{1}{\phi} \left[ \left( \frac{P_u}{P_c} \right)^2 + \left( \frac{V_u}{V_c} \right)^2 \right] \leq 1.0 \]  \hspace{1cm} (6.2.4.3-2a)
\[
\left( \frac{P_u}{P_s} \right)^2 + \left( \frac{V_u}{V_s} \right)^2 \leq 1.0
\]

(6.2.4.3-2b)

where:

\[ P_u = \text{required tensile strength, in pounds (N), based on factored loads and} \]

\[ V_u = \text{required shear strength, in pounds (N), based on factored loads.} \]

6.3 CLASSIFICATION OF MOMENT FRAMES:

6.3.1 ORDINARY MOMENT FRAMES: Ordinary moment frames are frames conforming to the requirements of Ref. 6-1 exclusive of Chapter 21.

6.3.2 INTERMEDIATE MOMENT FRAMES: Intermediate moment frames are frames conforming to the requirements of Sec. 21.8 of Ref. 6-1 in addition to those requirements for ordinary moment frames.

6.3.3 SPECIAL MOMENT FRAMES: Special moment frames are frames conforming to the requirements of Sec. 21.2 through 21.5 of Ref. 6-1 in addition to those requirements for ordinary moment frames.

6.4 SEISMIC PERFORMANCE CATEGORY A: Buildings assigned to Category A may be of any construction permitted in Ref. 6-1 and 6-2 and these provisions.

6.5 SEISMIC PERFORMANCE CATEGORY B: Buildings assigned to Category B shall conform to all the requirements for Category A and to the additional requirements for Category B in other chapters of these provisions.

6.5.1 ORDINARY MOMENT FRAMES: In flexural members of ordinary moment frames forming part of the seismic-force-resisting system, at least two main flexural reinforcing bars shall be provided continuously top and bottom throughout the beams, through or developed within exterior columns or boundary elements.

Columns of ordinary moment frames having a clear height to maximum plan dimension ratio of five or less shall be designed for shear in accordance with Sec. 21.8.3 of Ref. 6-1.

6.5.2 MOMENT FRAMES: All moment frames that are part of the seismic force resisting system of a building assigned to Category B and founded on Soil Profile Type E or F shall be intermediate moment frames conforming to Sec. 6.3.2 or special moment frames conforming to Sec. 6.3.3.

6.6 SEISMIC PERFORMANCE CATEGORY C: Buildings assigned to Category C shall conform to all the requirements for Category B and to the additional requirements for Category C in other chapters of these provisions as well as to the requirements of this section.
6.6.1 MOMENT FRAMES: All moment frames that are part of the seismic force resisting system shall be intermediate moment frames conforming to Sec. 6.3.2 or special moment frames conforming to Sec. 6.3.3.

6.6.2 DISCONTINUOUS MEMBERS: Columns supporting reactions from discontinuous stiff members such as walls shall be provided with transverse reinforcement at the spacing $s_o$ as defined in Sec. 21.8.5.1 of Ref. 6-1 over their full height beneath the level at which the discontinuity occurs. This transverse reinforcement shall be extended above and below the column as required in Sec. 21.4.4.5 of Ref. 6-1.

6.6.3 PLAIN CONCRETE: Structural members of plain concrete in buildings assigned to Category C shall conform to all requirements for Category B and the additional provisions and limitations of this section.

6.6.3.1 Walls: Basement, foundation, or other walls below the base shall be reinforced as required by Sec. 7.1.6.5 of Ref. 6-2. Other walls shall be reinforced as required by Sec. 8.3.7.2.

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

**EXCEPTION:** In detached one- and two-family dwellings three stories or less in height, the projection of the footing beyond the face of the supported member shall be permitted to exceed the footing thickness.

Plain concrete footings supporting walls shall be provided with not less than two continuous longitudinal reinforcing bars. Bars shall not be smaller than No. 4 and shall have a total area of not less than 0.002 times the gross cross-sectional area of the footing. Continuity of reinforcement shall be provided at corners and intersections.

**EXCEPTION:** In detached one- and two-family dwellings three stories or less in height and constructed with stud bearing walls, plain concrete footings supporting walls shall be permitted without longitudinal reinforcement.

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

6.7 SEISMIC PERFORMANCE CATEGORIES D AND E: Buildings assigned to Category D or E shall conform to all of the requirements for Category C and to the additional requirements of this section.

6.7.1 MOMENT FRAMES: All moment frames that are part of the seismic force resisting system, regardless of height, shall be special moment frames conforming to Sec. 6.3.3.

6.7.2 SEISMIC FORCE RESISTING SYSTEM: All materials and components in the seismic force resisting system shall conform to Sec. 21.2 through 21.6 of Ref. 6-1.
6.7.3 FRAME MEMBERS NOT PROPORTIONED TO RESIST FORCES INDUCED BY EARTHQUAKE MOTIONS: All frame components assumed not to contribute to lateral force resistance shall conform to Sec. 2.2.2.4.3 of these provisions and to Sec. 21.7.1.1 or 21.7.1.2 and 21.7.2 of Ref. 6-1.

6.7.4 PLAIN CONCRETE: Structural members of plain concrete are not permitted in buildings assigned to Category D or E.

EXCEPTIONS:

1. In detached one- and two-family dwellings three stories or less in height and constructed with stud bearing walls, plain concrete footings without longitudinal reinforcement supporting walls and isolated plain concrete footings supporting columns or pedestals are permitted.

2. In all other buildings, plain concrete footings supporting walls are permitted provided they are reinforced longitudinally as specified in Sec. 6.6.3.2.

3. In detached one- and two-family dwellings three stories or less in height and constructed with stud bearing walls, plain concrete foundation or basement walls are permitted provided the wall is not less than 7-1/2 in. (190 mm) thick and retains no more than 4 ft (1219 mm) of unbalanced fill.
Appendix to Chapter 6

REINFORCED CONCRETE STRUCTURAL SYSTEMS
COMPOSED FROM INTERCONNECTED PRECAST ELEMENTS

PREFACE: The provisions for reinforced concrete structural systems composed of precast elements in the body of the 1994 Provisions are for precast systems emulating monolithic reinforced concrete construction. However, one of the principal characteristics of precast systems is that they often are assembled using dry joints where connections are made by bolting, welding, post-tensioning, or other similar means. Research conducted to date documents concepts for design using dry joints and the behavior of subassemblages composed from interconnected precast elements both at and beyond peak strength levels for nonlinear reversed cyclic loadings (Applied Technology Council, 1981; Cheok and Lew, 1992; Clough, 1986; Elliott et al., 1987; Hawkins and Englekirk, 1987; Jayashanker and French, 1988; Mast, 1992; Nakaki and Englekirk, 1991; Neille, 1977; New Zealand Society, 1991; Pekau and Hum, 1991; Powell et al., 1993; Priestley, 1991; Priestley and Tao, 1992; Stanton et al., 1986; Stanton et al., 1991). This appendix is included for information and as a compilation of the current understanding of the performance under seismic loads of structural systems composed from interconnected precast elements. It is considered premature to base code provisions on this resource appendix; however, user review, trial designs, and comment on this appendix are encouraged. Please direct such feedback to the BSSC.

6A.1 GENERAL:

6A.1.1 SCOPE: Design and construction of lateral force resisting structural systems composed using interconnected precast concrete elements shall comply with the requirements of this appendix. The quality and testing of concrete and steel materials and the design and construction of the precast concrete components and systems that resist seismic forces shall conform to the requirements of the reference document listed in this section except as modified by the provisions of Chapter 6 and this appendix.

6A.1.2 REFERENCE DOCUMENT:

Ref. 6A-1 Building Code Requirements for Reinforced Concrete, American Concrete Institute, ACI 318-89 (Revised 1992), excluding Appendix A.

* See the Commentary for this appendix for full reference information.
6A.2 GENERAL PRINCIPLES: A reinforced concrete structural system composed from interconnected precast concrete elements shall be permitted for the lateral force resisting system:

1. If the force-deformation relationships for the connection regions have been validated through physical experiments or the use of analytical models based on the results of physical experiments that closely simulate the building's connection regions and

2. If the response of the building is analyzed using the force-deformation relationships for the connection and joint regions in combination with the force-deformation relationships for the precast concrete elements connected by those regions.

6A.3 LATERAL FORCE RESISTING STRUCTURAL FRAMING SYSTEMS:

6A.3.1: The basic structural and seismic force resisting systems and seismic performance category and building height limitations shall be those specified in Table 2.2.2. The response modification coefficients, \( R \), and the deflection amplification factors, \( C_d \), of Table 2.2.2 shall be taken as maximum values for interconnected construction.

6A.3.2: The response modification coefficients, \( R \), and the deflection amplification factors, \( C_d \), for interconnected construction shall be consistent with the detailing practice for the connections.

6A.3.3: Where force-deformation relationships for the connections have been determined from analytical modeling and have not been validated through physical experiments, \( R \) and \( C_d \) factors for interconnected construction shall be restricted as shown in Table 6A.3.3.

<table>
<thead>
<tr>
<th>Restricted Response Modification Coefficient, ( R_j )</th>
<th>Restricted Deflection Amplification Factor, ( C_{dj} )</th>
<th>Seismic Performance Category (^a)</th>
<th>Connection Performance Category (^b)</th>
</tr>
</thead>
<tbody>
<tr>
<td>( R_j \leq R/2 )</td>
<td>( C_{dj} \leq C_d/2 )</td>
<td>P</td>
<td>P</td>
</tr>
<tr>
<td>( R/2 &lt; R_j \leq R - 1 )</td>
<td>( C_d/2 \leq C_{dj} \leq C_d - 1 )</td>
<td>P</td>
<td>P</td>
</tr>
</tbody>
</table>

NOTE: \( R = R \) value for monolithic concrete construction in Table 2.2.2 and \( C_d = C_d \) value for monolithic concrete construction in Table 2.2.2. \( R_j \) and \( C_{dj} \) shall be varied in step with \( R \) and \( C_d \) between limits shown. \( P \) = permitted and \( NP \) = not permitted.

\(^a\) See Table 1.4.4.

\(^b\) See Sec. 6A.4.3.
6A.3.4: Designs shall provide:

1. A continuous load path to the foundation for all components for seismic forces;

2. Force-deformation relationships for the connection and joint regions that result in a lateral deflection profile for the structure that has deflections increasing continuously with increasing height above the structure's base when a horizontal force is applied in any direction at the top of the structure; and

3. Integrity of the entire load path at deformations \( C_d \) times the elastic deformation.

6A.4 CONNECTION PERFORMANCE REQUIREMENTS:

6A.4.1: Connections that are part of the lateral force resisting system and are intended to be nonlinear action locations shall have hinging, sliding, or extending characteristics provided by at least one of the following means:

1. Member hinging in flexure due to reinforcement yield in tension and/or compression or in-plane dry joint opening rotation constrained by yielding in tension or compression of reinforcement crossing that joint.

2. Dry joint movement caused by yield in tension, flexure, or shear of steel plates, bars, or shapes crossing that joint.

3. In-plane dry joint slips caused by shears acting on constrained deformation devices such as friction bolted steel assemblies.

4. Other actions for which physical experiments have established the deformation response of the connection and the region surrounding the connection or joint.

6A.4.2: The seismic performance of a given connection depends on the characteristics of all three of the following:

1. Connector -- The device that crosses the interface between the interconnected precast elements or the cast-in-place element.

2. Anchorage -- The means by which the force in the connector is transferred into the precast or cast-in-place element, and

3. Connection Region -- The volume of element over which the force from the anchorage flows out to match the uniform stress state for the element.

6A.4.3: Based on the results of physical experiments or analytical modeling, nonlinear action, connections and their surrounding connection or joint regions shall be classified into Connection Seismic Performance Categories A, B, and C as follows:
1. For Connection Performance Category A, there shall be no special requirements.

2. For Connection Performance Category B, connections and their surrounding regions shall exhibit stable inelastic reversed cyclic deformation characteristics for the demands placed on them at the $R$ and $C_d$ values selected for the building's seismic force resisting system.

3. For Connection Performance Category C, connections and their surrounding regions shall have stiffness, strength, energy absorption, and energy dissipation capacities that ensure a performance for the building equivalent to that required for the $R$ and $C_d$ values selected for the building's seismic force resisting system.

6A.4.4: For lateral force resisting systems of Seismic Performance Category B, the nonlinear action connections shall be of Connection Performance Category B or C and the anchorage for any such connector transferring tensile or shear force shall be connected directly by welding or similar means or by adequate lap length to the principal reinforcement of the precast element or the cast-in-place element.

6A.4.5: For lateral force resisting systems of Seismic Performance Category C, D or E, the nonlinear action connections shall be of Connection Performance Category C with the anchorage specified in Sec. 6A.4.4 and with the stressed area at the connection interface for nominal strength calculations at least 30 percent of the cross-sectional area of the element measured at a distance equal to the section's largest dimension from that interface. The stressed area for principal reinforcement stressed in tension or shear shall be the same as that defined in Sec. 10.6.4 of Ref. 6A-1.

**6A.5 CONNECTION DESIGN REQUIREMENTS:**

6A.5.1: Connections that are nonlinear action locations shall satisfy the following design requirements:

1. The probable strength, $S_{pr}$, of the connector shall be determined using a $\phi$ value of unity and a steel stress of at least $1.25f_y$.

2. The connector shall be anchored either side of the interface for capacities at least 1.6 times the $S_{pr}$ value for that connector.

6A.5.2: Connectors that are part of the lateral load resisting path and intended to remain elastic while Connection Performance Category B or C connectors undergo nonlinear actions shall have a strength, $S_p$, at least 1.5 times the load calculated as acting on them when the nonlinear action of the building's structural system is fully developed.

6A.5.3: Connectors that are nonlinear action locations shall be proportioned so that they provide significant resistance only for the direction in which their capacity is intended to be utilized.
6A.5.4: Particular attention shall be given to grouting and welding requirements that shall permit quality control inspection and testing and make allowance for varying tolerances, material properties, and site conditions.