20 Seismic Design

20.1 Introduction

Earthquakes result from the sudden movement of tectonic plates in the earth's crust. The movement takes place at fault lines, and the energy released is transmitted through the earth in the form of waves that cause ground motion many miles from the epicenter. Regions adjacent to active fault lines are the most prone to experience earthquakes. The map in Fig. 20.1 shows the *maximum considered ground motion* for the contiguous 48 states. The mapped values represent the expected peak acceleration of a single degree-of-freedom system with a 0.2 sec period and 5 percent of critical damping. Known as the *0.2 sec spectral response acceleration* S_s (subscript s for *short* period), it is used, along with the 1.0 sec spectral response acceleration S_1 (mapped in a similar manner), to establish the loading criteria for seismic design. Accelerations S_s and S_1 are based on historical records and local geology. For most of the country, they represent earthquake ground motion with a "likelihood of exceedance of 2 percent in 50 years," a value that is equivalent to a return period of about 2500 years (Ref. 20.1).

As experienced by structures, earthquakes consist of random horizontal and vertical movements of the earth's surface. As the ground moves, inertia tends to keep structures in place (Fig. 20.2), resulting in the imposition of displacements and forces that can have catastrophic results. The purpose of seismic design is to proportion structures so that they can withstand the displacements and the forces induced by the ground motion.

Historically in North America, seismic design has emphasized the effects of horizontal ground motion, because the horizontal components of an earthquake usually exceed the vertical component and because structures are usually much stiffer and stronger in response to vertical loads than they are in response to horizontal loads. Experience has shown that the horizontal components are the most destructive. For structural design, the intensity of an earthquake is usually described in terms of the ground acceleration as a fraction of the acceleration of gravity, i.e., 0.1, 0.2, or $0.3g$. Although peak acceleration is an important design parameter, the frequency characteristics and duration of an earthquake are also important; the closer the frequency of the earthquake motion is to the natural frequency of a structure and the longer the duration of the earthquake, the greater the potential for damage.

Based on elastic behavior, structures subjected to a major earthquake would be required to undergo large displacements. However, North American practice (Ref. 20.2) requires that structures be designed for only a fraction of the forces associated with those displacements. The relatively low design forces are justified by the observations that buildings designed for low forces have behaved satisfactorily and that structures dissipate significant energy as the materials yield and behave inelastically.

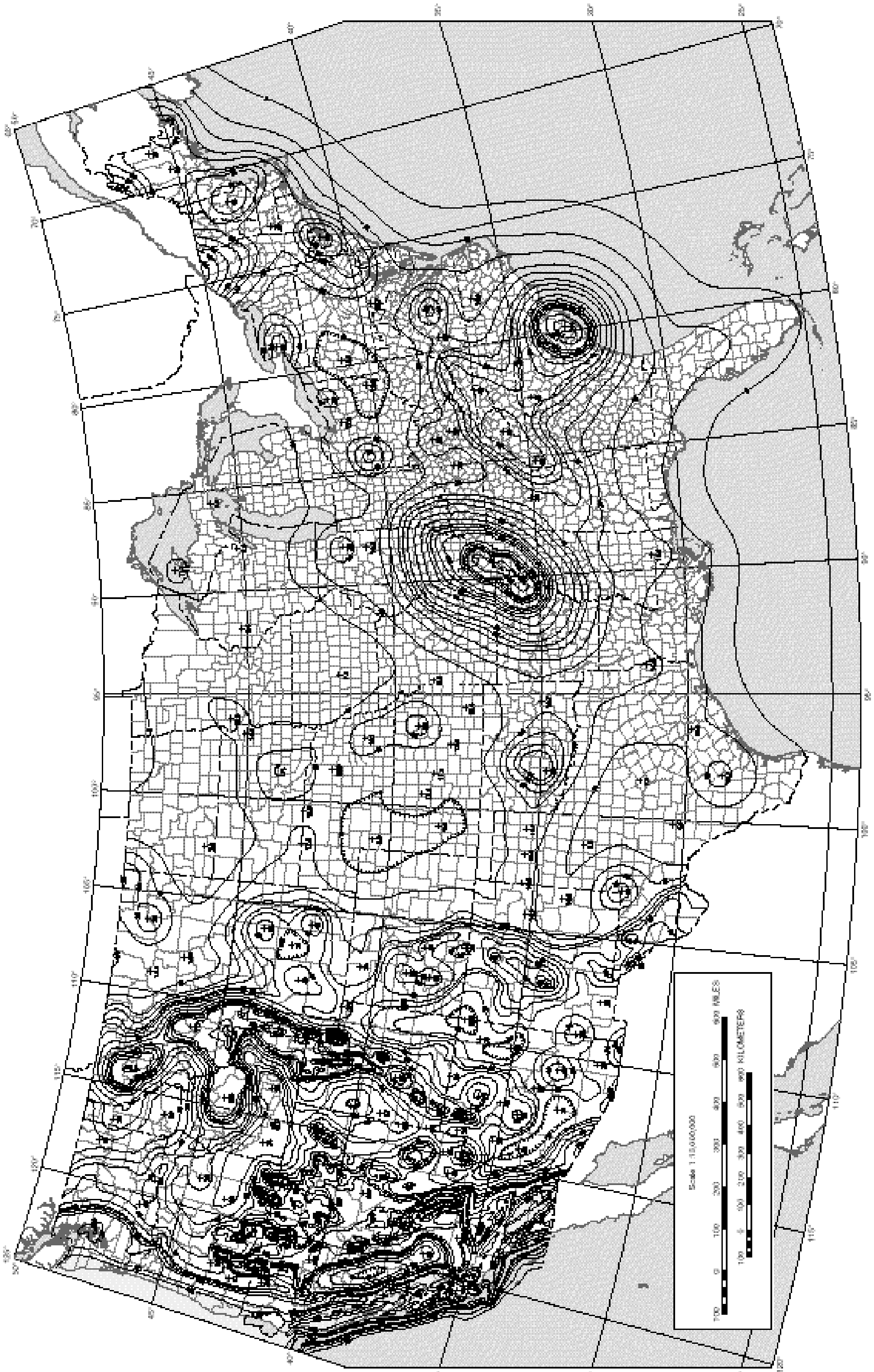

FIGURE 20.1
Map showing maximum considered earthquake ground motion, 0.2 sec spectral response acceleration (5 percent of critical damping), for the contiguous United States. (*United States Geological Survey.*)

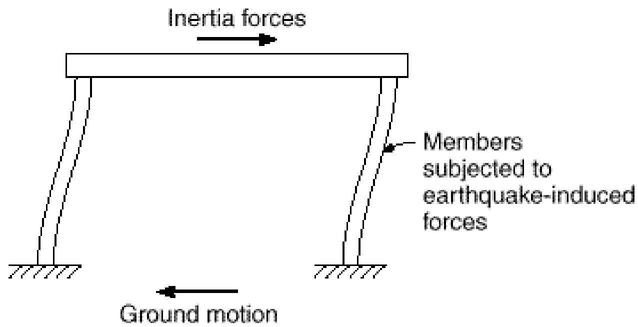

FIGURE 20.2
Structure subjected to ground motion.

This nonlinear behavior, however, usually translates into increased displacements, which may result in major nonstructural damage and require significant ductility. Displacements may also be of such a magnitude that the strength of the structure is affected by stability considerations, such as discussed for slender columns in Chapter 9.

Designers of structures that may be subjected to earthquakes, therefore, are faced with a choice: (a) providing adequate stiffness and strength to limit the response of structures to the elastic range or (b) providing lower-strength structures, with presumably lower initial costs, that have the ability to withstand large inelastic deformations while maintaining their load-carrying capability.

20.2 Structural Response

The safety of a structure subjected to seismic loading rests on the designer's understanding of the response of the structure to ground motion. For many years, the goal of earthquake design in North America has been to construct buildings that will withstand *moderate earthquakes without damage and severe earthquakes without collapse*. Building codes have undergone regular modification as major earthquakes have exposed weaknesses in existing design criteria.

Design for earthquakes differs from design for gravity and wind loads in the relatively greater sensitivity of earthquake-induced forces to the geometry of the structure. Without careful design, forces and displacements can be concentrated in portions of a structure that are not capable of providing adequate strength or ductility. Steps to strengthen a member for one type of loading may actually increase the forces in the member and change the mode of failure from ductile to brittle.

a. Structural Considerations

The closer the frequency of the ground motion is to one of the natural frequencies of a structure, the greater the likelihood of the structure experiencing resonance, resulting in an increase in both displacement and damage. Therefore, earthquake response depends strongly on the geometric properties of a structure, especially height. Tall buildings respond more strongly to long-period (low-frequency) ground motion, while short buildings respond more strongly to short-period (high-frequency) ground motion. Figure 20.3 shows the shapes for the principal modes of vibration of a three-story frame structure. The relative contribution of each mode to the lateral displacement of the structure depends on the frequency characteristics of the ground motion. The first mode (Fig. 20.3*a*) usually provides the greatest contribution to lateral dis-

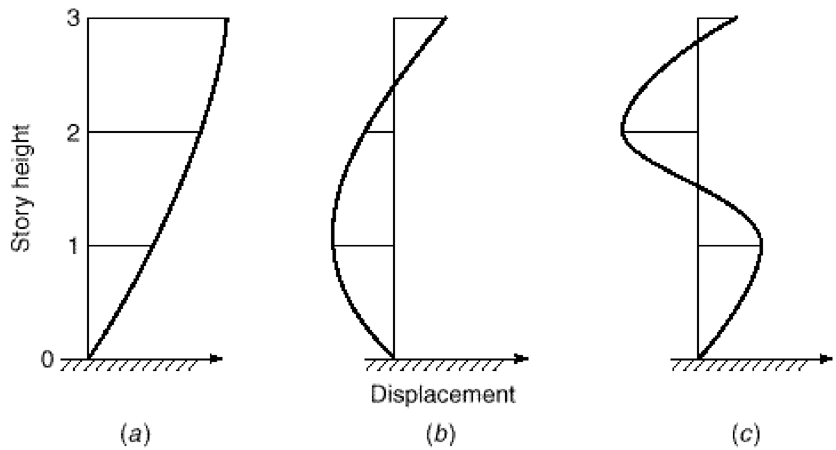

FIGURE 20.3
Modal shapes for a three-story building: (*a*) first mode; (*b*) second mode; (*c*) third mode. (*Adapted from Ref. 20.3.*)

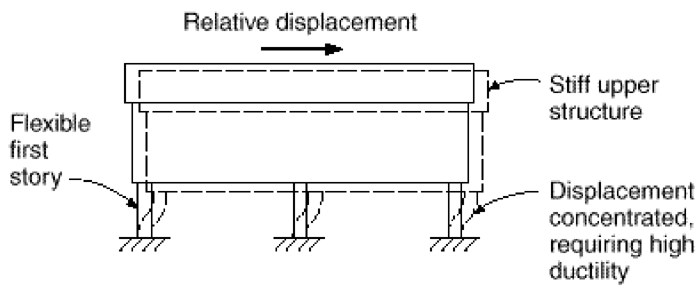

FIGURE 20.4
Soft first story supporting a stiff upper structure.

placement. The taller a structure, the more susceptible it is to the effects of higher modes of vibration, which are generally additive to the effects of the lower modes and tend to have the greatest influence on the upper stories. Under any circumstances, the longer the duration of an earthquake, the greater the potential for damage.

The configuration of a structure also has a major effect on its response to an earthquake. Structures with a discontinuity in stiffness or geometry can be subjected to undesirably high displacements or forces. For example, the discontinuance of shear walls, infill walls, or even cladding at a particular story level, such as shown in Fig. 20.4, will have the result of concentrating the displacement in the open, or "soft," story. The high displacement will, in turn, require a large amount of ductility if the structure is not to fail. Such a design is not recommended, and the stiffening members should be continued to the foundation. The problems associated with a soft story are illustrated in Fig. 20.5, which shows the Olive View Hospital following the 1971 San Fernando earthquake. The high ductility "demand" could not be satisfied by the column at the right, with low amounts of transverse reinforcement. Even the columns at center, with significant transverse reinforcement, performed poorly because the transverse reinforcement was not continued into the joint, resulting in the formation of hinges at the column ends. Figure 20.6 illustrates structures with vertical geometric and plan irregularities, which result in torsion induced by ground motion.

Within a structure, stiffer members tend to pick up a greater portion of the load. When a frame is combined with a shear wall, this can have the positive effect of reducing the displacements of the structure and decreasing both structural and nonstructural damage. However, when the effects of higher stiffness members, such as masonry infill walls, are not considered in the design, unexpected and often undesirable results can occur.

FIGURE 20.5
Damage to soft story columns in the Olive View Hospital as a result of the 1971 San Fernando earthquake. *(Photograph by James L. Stratta. Courtesy of the Federal Emergency Management Agency.)*

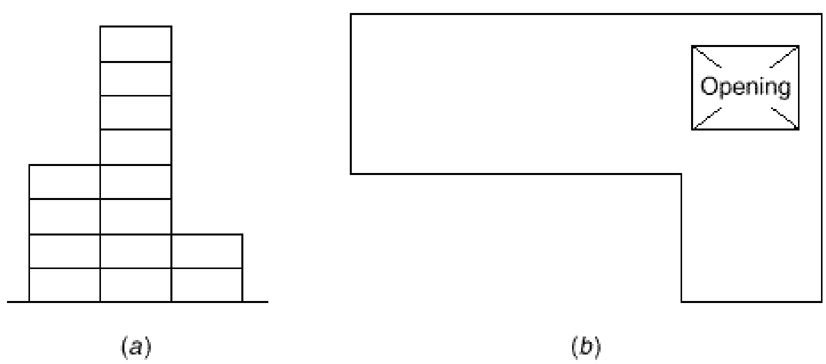

FIGURE 20.6
Structures with (*a*) vertical geometric and (*b*) plan irregularities. *(Adapted from Ref. 20.3.)*

Finally, any discussion of structural considerations would be incomplete without emphasizing the need to provide adequate separation between structures. Lateral displacements can result in structures coming in contact during an earthquake, resulting in major damage due to hammering, as shown in Fig. 20.7. Spacing requirements to ensure that adjacent structures do not come into contact as the result of earthquake-induced motion are specified in Ref. 20.2.

b. Member Considerations

Members designed for seismic loading must perform in a ductile fashion and dissipate energy in a manner that does not compromise the strength of the structure. Both the overall design and the structural details must be considered to meet this goal.

FIGURE 20.7
Damage caused by hammering for buildings with inadequate separation in 1985 Mexico City earthquake. (*Photograph by Jack Moehle.*)

The principal method of ensuring ductility in members subject to shear and bending is to provide confinement for the concrete. This is accomplished through the use of closed *hoops* or spiral reinforcement, which enclose the core of beams and columns. Specific criteria are discussed in Sections 20.4, 20.5, and 20.6. When confinement is provided, beams and columns can undergo nonlinear cyclic bending while maintaining their flexural strength and without deteriorating due to diagonal tension cracking. The formation of *ductile hinges* allows reinforced concrete frames to dissipate energy.

Successful seismic design of frames requires that the structures be proportioned so that hinges occur at locations that least compromise strength. For a frame undergoing lateral displacement, such as shown in Fig. 20.8*a*, the flexural capacity of the members at a joint (Fig. 20.8*b*) should be such that the columns are stronger than the beams. In this way, hinges will form in the beams rather than the columns, minimizing the portion of the structure affected by nonlinear behavior and maintaining the overall vertical load capacity. For these reasons, the "weak beam–strong column" approach is used to design reinforced concrete frames subject to seismic loading.

When hinges form in a beam, or in extreme cases within a column, the moments at the end of the member, which are governed by flexural strength, determine the shear

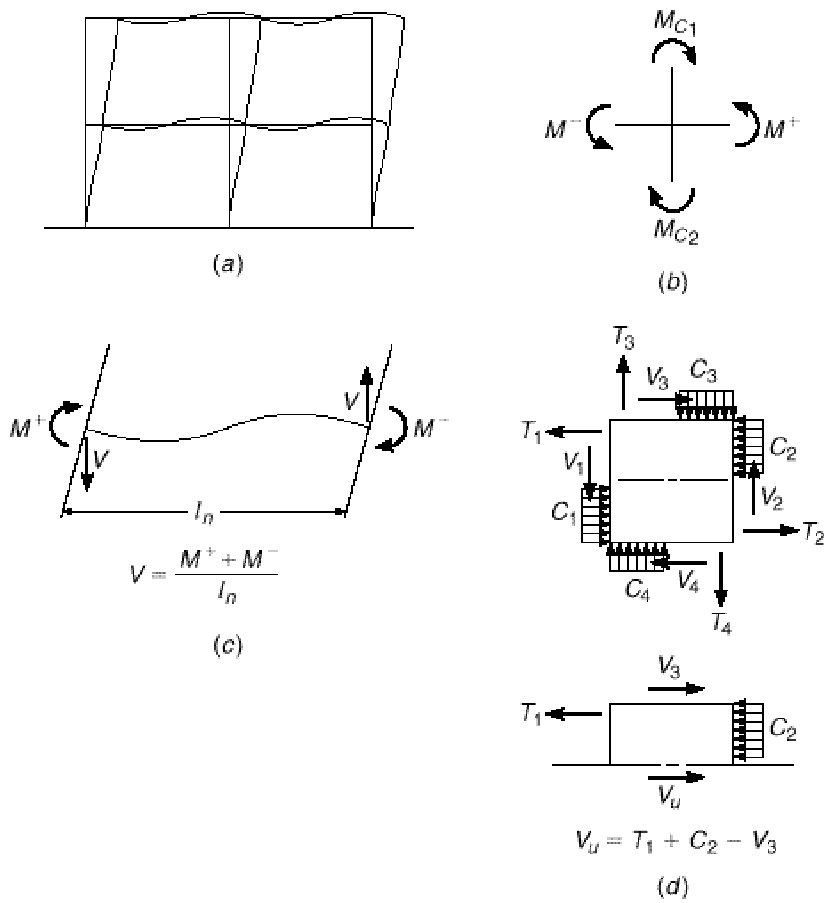

FIGURE 20.8
Frame subjected to lateral loading: (*a*) deflected shape; (*b*) moments acting on beam-column joint; (*c*) deflected shape and forces acting on a beam; (*d*) forces acting on faces of a joint due to lateral load.

that must be carried, as illustrated in Fig. 20.8*c*. The shear V corresponding to a flexural failure at both ends of a beam or column is

$$V = \frac{M^+ + M^-}{l_n} \tag{20.1}$$

where M^+ and M^- = flexural capacities at the ends of the member
l_n = clear span between supports

The member must be checked for adequacy under the shear V in addition to shear resulting from dead and live gravity loads. Transverse reinforcement is added, as required. For members with inadequate shear capacity, the response will be dominated by the formation of diagonal cracks, rather than ductile hinges, resulting in a substantial reduction in the energy dissipation capacity of the member.

If short members are used in a frame, the members may be unintentionally strong in flexure compared to their shear capacity. An example would be columns in a structure with deep spandrel beams or with "nonstructural" walls with openings that expose a portion of the columns to the full lateral load. As a result the exposed region, called a *captive column*, responds by undergoing a shear failure, as shown in Fig. 20.9.

The lateral displacement of a frame places beam-column joints under high shear stresses because of the change from positive to negative bending in the flexural members from one side of the joint to the other, as shown in Fig. 20.8*d*. The joint must be

FIGURE 20.9
Shear failure in a captive column without adequate transverse reinforcement. (*Photograph by Jack Moehle.*)

able to withstand the high shear stresses and allow for a change in bar stress from tension to compression between the faces of the joint. Such a transfer of shear and bond is often made difficult by congestion of reinforcement through the joint. Thus, designers must ensure that joints not only have adequate strength but are constructable. Two-way systems without beams are especially vulnerable because of low ductility at the slab-column intersection.

Additional discussion of seismic design can be found in Refs. 20.3 to 20.7.

20.3 Seismic Loading Criteria

In the United States, the design criteria for earthquake loading are based on design procedures developed by the Building Seismic Safety Council (Ref. 20.1) and incorporated in *Minimum Design Loads for Buildings and Other Structures* (SEI/ASCE 7) (Ref. 20.2). The values of the spectral response accelerations S_s and S_1 are obtained from detailed maps produced by the United States Geological Survey† (e.g., Fig. 20.1) and included in SEI/ASCE 7. The values of S_s and S_1 are used to determine the spectral response accelerations S_{DS} and S_{D1} that are used in design.

$$S_{DS} = \frac{2}{3} F_a S_s \tag{20.2}$$

$$S_{D1} = \frac{2}{3} F_v S_1 \tag{20.3}$$

† A full set of maps is available at the United States Geological Survey website.

where F_a and F_v are site coefficients that range from 0.8 to 0.25 and from 0.8 to 0.35, respectively, as a function of the geotechnical properties of the building site and the values of S_s and S_1, respectively. Higher values of F_a and F_v are possible for some sites. The coefficients F_a and F_v increase in magnitude as site conditions change from hard rock to thick, soft clays and (for softer foundations) as the values of S_s and S_1 decrease.

Structures are assigned to one of six *seismic design categories*, A through F, as a function of (a) structure occupancy and use and (b) the values of S_{DS} and S_{D1}. Requirements for seismic design and detailing are minimal for Seismic Design Categories A and B but become progressively more rigorous for Seismic Design Categories C through F.

As presented in Table 1.2, earthquake loading is included in two combinations of factored load.

$$U = 1.2D + 1.0E + 1.0L + 0.2S \tag{20.4}$$

$$U = 0.9D + 1.0E + 1.6H \tag{20.5}$$

where D = dead load
E = earthquake load
H = weight or pressure from soil
L = live load
S = snow load

The values of the earthquake load E used in Eqs. (20.4) and (20.5) are, respectively,

$$E = \cdot\, Q_E + 0.2S_{DS}D \tag{20.6a}$$

$$E = \cdot\, Q_E - 0.2S_{DS}D \tag{20.6b}$$

where Q_E = effect of horizontal seismic forces
$\cdot$ = reliability factor

$\cdot$ is taken as 1.0 for structures in Seismic Design Categories A through C and as $\cdot_x$ for structures in Seismic Design Categories D through F, where

$$\cdot_x = 2 - \frac{20}{r_{max_x} \cdot \overline{A_x}} \tag{20.7}$$

$\leq$ 1.5 for Seismic Design Category D

$\leq$ 1.1 for Seismic Design Categories E and F

where r_{max_x} is the ratio of the design story shear resisted by the single element carrying the most shear in a story to the total story shear for a given direction of loading. For braced frames, the value of r_{max_x} is equal to the lateral force component in the most heavily loaded braced element divided by the story shear; for moment frames, r_{max_x} is the sum of the shears in any two adjacent columns in the plane of the moment frame divided by the story shear (Ref. 20.2). Other criteria for r_{max_x} are specified for buildings with shear walls (Ref. 20.2).

Equations (20.4) and (20.6*a*) are used when dead load adds to the effects of horizontal ground motion, while Eqs. (20.5) and (20.6*b*) are used when dead load counteracts the effects of horizontal ground motion. Thus, the total load factor for dead load is greater than 1.2 in Eq. (20.4) and less than 0.9 in Eq. (20.5).

SEI/ASCE 7 specifies six procedures for determining the horizontal earthquake load Q_E. These procedures include three progressively more detailed methods that represent earthquake loading through the use of equivalent static lateral loads, *modal* *response spectrum analysis, linear time-history analysis,* and *nonlinear time-history analysis.* The method selected depends on the seismic design category. Buildings in Seismic Design Category A (S_{DS} less than $0.167g$ and S_{D1} less than $0.067g$, where g is the acceleration of gravity) may be designed by any of the methods. The required level of sophistication in determining Q_E increases, however, with increases in S_{DS} and S_{D1} and the nature of the structural occupancy or use. Most reinforced concrete structures in Seismic Design Categories B through F must be designed using *equivalent lateral force analysis* (the most detailed of the three equivalent static lateral load procedures), modal response analysis, or time-history analysis. These procedures are discussed next.

a. Equivalent Lateral Force Procedure

According to SEI/ASCE 7 (Ref. 20.2), equivalent lateral force analysis may be applied to all structures with S_{DS} less than $0.33g$ and S_{D1} less than $0.133g$, as well as structures subjected to much higher design spectral response accelerations, if the structures meet certain requirements. More sophisticated dynamic analysis procedures must be used otherwise.

The equivalent lateral force procedure provides for the calculation of the total lateral force, defined as the design base shear V, which is then distributed over the height of the building. The design base shear V is calculated for a given direction of loading according to the equation

$$V = C_s W \tag{20.8}$$

where W is the total dead load plus applicable portions of other loads, and

$$C_s = \frac{S_{DS}}{R \cdot I} \tag{20.9}$$

which need not be greater than

$$C_s = \frac{S_{D1}}{T \cdot R \cdot I \cdot} \tag{20.10}$$

but may not be less than

$$C_s = 0.44 I S_{DS} \tag{20.11}$$

or for the highest seismic design categories (E and F),

$$C_s = \frac{0.5 S_1}{R \cdot I} \tag{20.12}$$

where R = response modification factor (depends on the structural system). Values of R for reinforced concrete structures range from 4 to 8, based on ability of the structural system to sustain earthquake loading and to dissipate energy

I = occupancy important factor = 1.0, 1.25, or 1.5, depending upon the occupancy and use of the structure

T = fundamental period of the structure

According to SEI/ASCE 7, the period T can be calculated based on an analysis that accounts for the structural properties and deformational characteristics of the elements within the structure. Approximate methods may also be used in which the fundamental period of the structure may be calculated as

$$T = C_t h_n^x \tag{20.13}$$

where h_n = height above the base to the highest level of structure, ft
C_t = 0.016 for reinforced concrete moment-resisting frames in which frames resist 100 percent of required seismic force and are not enclosed or adjoined by more rigid components that will prevent frame from deflecting when subjected to seismic forces, and 0.020 for all other reinforced concrete buildings
x = 0.90 for $C_t = 0.016$ and 0.75 for $C_t = 0.020$

Alternately, for structures not exceeding 12 stories in height, in which the lateral force-resisting system consists of a moment-resisting frame and the story height is at least 10 ft,

$$T = 0.1N \tag{20.14}$$

where N = number of stories.

For shear wall structures, SEI/ASCE 7 permits T to be approximated as

$$T = \frac{0.0019}{\sqrt{C_w}} h_n \tag{20.15}$$

where

$$C_w = \frac{100}{A_B} \sum_{i=1}^{n} \left(\frac{h_n}{h_i}\right)^2 \frac{A_i}{1 + 0.83\left(\frac{h_i}{D_i}\right)^2} \tag{20.16}$$

where A_B = base area of structure, ft^2
A_i = area of shear wall, ft^2
D_i = length of shear wall i, ft
n = number of shear walls in building that are effective in resisting lateral forces in direction under consideration

The total base shear V is distributed over the height of the structure in accordance with Eq. (20.17),

$$F_x = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} V \tag{20.17}$$

where F_x = lateral seismic force induced at level x
w_x, w_i = portion of W at level x and level i, respectively
h_x, h_i = height to level x and level i, respectively
k = exponent related to structural period, = 1 for $T \leq 0.5$ sec and = 2 for $T \geq 2.5$ sec. For $0.5 < T < 2.5$, k is determined by linear interpolation or set to a value of 2

The design shear at any story V_x equals the sum of the forces F_x at and above that story. For a 10-story building with a uniform mass distribution over the height and T = 1.0 sec, the lateral forces and story shears are distributed as shown in Fig. 20.10.

At each level, V_x is distributed in proportion to the stiffness of the elements in the vertical lateral force–resisting system. To account for unintentional building irregularities that may cause a horizontal torsional moment, a minimum 5 percent eccentricity must be applied if the vertical lateral force–resisting systems are connected by a floor system that is rigid in its own plane.

In addition to the criteria just described, SEI/ASCE 7 includes criteria to account for overturning effects and provides limits on story drift. P-Δ effects must be considered (as discussed in Chapter 9), and the effects of upward loads must be accounted for in the design of horizontal cantilever components and prestressed members.

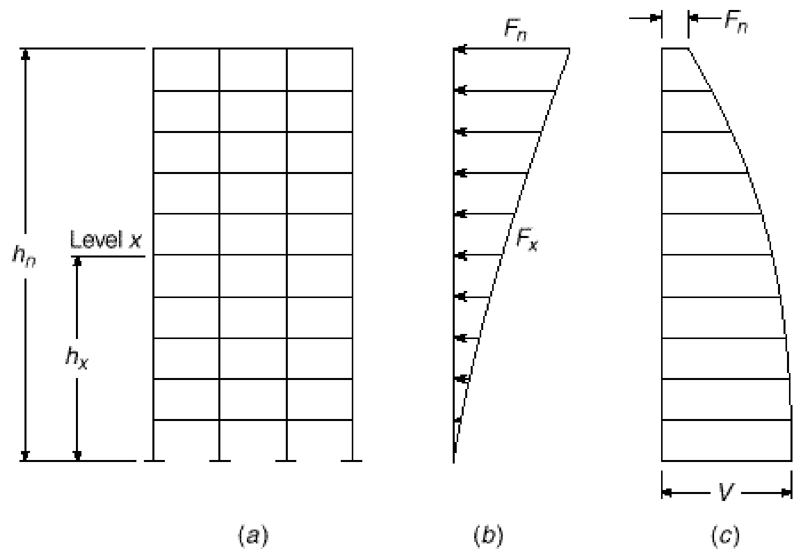

FIGURE 20.10
Forces based on SEI/ASCE 7 (Ref. 20.2) equivalent lateral force procedure: (a) structure; (b) distribution of lateral forces over height; (c) story shears.

b. Dynamic Lateral Force Procedures

SEI/ASCE 7 includes dynamic lateral force procedures that involve the use of (a) response spectra, which provide the earthquake-induced forces as a function of the natural periods of the structure, or (b) time-history analyses of the structural response based on a series of ground motion acceleration histories that are representative of ground motion expected at the site. Both procedures require the development of a mathematical model of the structure to represent the spatial distribution of mass and stiffness. Response spectra are used to calculate peak forces for a "sufficient number of nodes to obtain the combined modal mass participation of at least 90 percent of the actual mass in each of two orthogonal directions" (Ref. 20.2). Since these forces do not always act in the same direction, as shown in Fig. 20.3, the peak forces are averaged statistically, in most cases using the square root of the sum of the squares to obtain equivalent static lateral forces for use in design. In cases where the periods in the translational and torsional modes are closely spaced and result in significant cross correlation of the modes, the so-called *complete quadratic combination* method is used (Ref. 20.8). When time-history analyses, which may include a linear or nonlinear representation of the structure, are used, design forces are obtained directly from the analyses. Both modal response spectrum and time-history procedures provide more realistic representations of the seismically induced forces in a structure than do equivalent lateral force analyses. The details of these methods are presented in Refs. 20.1 and 20.2.

20.4 ACI Special Provisions for Seismic Design

Criteria for seismic design are contained in Special Provisions for Seismic Design, Chapter 21 of the ACI Code (Ref. 20.9). The principal goal of the Special Provisions is to ensure adequate toughness under inelastic displacement reversals brought on by earthquake loading. The provisions accomplish this goal by requiring the designer to provide for concrete confinement and inelastic rotation capacity. The provisions apply to frames, walls, coupling beams, diaphragms, and trusses in structures subjected to "high seismic risk," corresponding to Seismic Design Categories D, E, and F, and to frames, including two-way slab systems, subject to "moderate/intermediate seismic risk," corresponding to Seismic Design Category C. No special requirements are placed on structures subject to low or no seismic risk. Structural systems designed for high and moderate seismic risk are referred to as *special* and *intermediate*, respectively.

The ACI Special Provisions are based on many of the observations made earlier in this chapter. The effect of nonstructural elements on overall structural response must be considered, as must the response of the nonstructural elements themselves. Structural elements that are not specifically proportioned to carry earthquake loads must also be considered.

The load factors used for earthquake loads are given in Eqs. (20.4) and (20.5). The strength-reduction factors used for seismic design are the same as those used for nonseismic design (Table 1.3), with the additional requirements that $\phi = 0.60$ for shear, if the nominal shear capacity of a member is less than the shear based on the nominal flexural strength [see Eq. (20.1)], and $\phi = 0.85$ for shear in joints and diagonally reinforced coupling beams.

To ensure adequate ductility and toughness under inelastic rotation, ACI Code 21.2.4 sets a minimum concrete strength of 3000 psi. For lightweight aggregate concrete, an *upper limit* of 5000 psi is placed on concrete strength; this limit is based on a lack of experimental evidence for higher-strength lightweight concretes.

Under ACI Code 21.2.5, reinforcing steel must meet ASTM A 706 (see Table 2.3). ASTM A 706 specifies a Grade 60 steel with a maximum yield strength of 78 ksi and a minimum tensile strength equal to 80 ksi. The actual tensile strength must be at least 1.25 times the actual yield strength. In addition to reinforcement manufactured under ASTM A 706, the Code allows the use of Grades 40 and 60 reinforcement meeting the requirements of ASTM A 615, provided that the actual yield strength does not exceed the specified yield by more than 18 ksi and that the actual tensile strength exceeds the actual yield strength by at least 25 percent. The upper limits on yield strength are used to limit the maximum moment capacity of the section because of the dependency of the earthquake-induced shear on the moment capacity [Eq. (20.1)]. The minimum ratio of tensile strength to yield strength helps provide adequate inelastic rotation capacity. Evidence reported in Ref. 20.11 indicates that an increase in the ratio of the ultimate moment to the yield moment results in an increase in the nonlinear deformation capacity of flexural members.

Confinement for concrete is provided by transverse reinforcement consisting of stirrups, hoops, and crossties. To ensure adequate anchorage, a *seismic hook* [with a bend not less than 135° and a 6 bar diameter (but not less than 3 in.) extension that engages the longitudinal reinforcement and projects into the interior of the stirrup or hoop] is used on stirrups, hoops, and crossties. *Hoops*, shown in Figs. 7.11*a*, *c–e* and 20.11, are closed ties that can be made up of several reinforcing elements, each having seismic hooks at both ends, or continuously wound ties with seismic hooks at both ends. A *crosstie* (see Fig. 20.11) is a continuous reinforcing bar with a seismic hook at one end and a hook with not less than a 90° bend and at least a 6 bar diameter extension at the other end. The hooks on crossties must engage peripheral longitudinal reinforcing bars.

In the following sections, ACI requirements for frames, walls, diaphragms, and trusses subject to seismic loading are discussed. Sections 20.5 and 20.6 describe the general design and detailing criteria for members in structures designed for regions of high seismic risk. Specific shear strength requirements are presented in Section 20.7. Section 20.8 describes requirements for frames subject to moderate seismic risk.

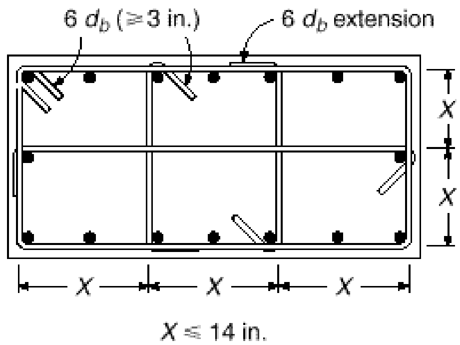

FIGURE 20.11
Example of transverse reinforcement in columns; consecutive crossties engaging the same longitudinal bars must have 90° hooks on opposite sides of columns. (*Adapted from Ref. 20.10.*)

20.5 ACI Provisions for Special Moment Frames

ACI Code Chapter 21 addresses four member types in frame structures, termed *special moment frames*, subject to high seismic risk: flexural members, members subjected to bending and axial load, joints, and members not proportioned to resist earthquake forces. Two-way slab systems without beams are prohibited as lateral load–resisting structures if subject to high seismic risk.

a. Flexural Members

Flexural members are defined by ACI Code 21.3.1 as structural members that resist earthquake-induced forces but have a factored axial compressive load that does not exceed $A_g f'_c/10$, where A_g is the gross area of the cross section. The members must have a clear span-to-effective depth ratio of at least 4, a width-to-depth ratio of at least 0.3, and a web width of not less than 10 in. nor more than the support width plus three-quarters of the flexural member depth on either side of the support. The minimum clear span-to-depth ratio helps ensure that flexural rather than shear strength dominates member behavior under inelastic load reversals. Minimum web dimensions help provide adequate confinement for the concrete, whereas the width relative to the support (typically a column) is limited to provide adequate moment transfer between beams and columns.

In accordance with ACI Code 21.3.2, both top and bottom minimum flexural steel is required. $A_{s,min}$ should not be less than given by Eq. (3.41) but need not be greater than four-thirds of that required by analysis, with a minimum of two reinforcing bars, top and bottom, throughout the member. In addition, the positive moment capacity at the face of columns must be at least one-half of the negative moment strength at the same location, and neither positive nor negative moment strength at any section in a member may be less than one-fourth of the maximum moment strength at either end of the member. These criteria are designed to provide for ductile behavior throughout the member, although the minimum of two reinforcing bars on the top and bottom is based principally on construction requirements. A maximum reinforcement ratio of 0.025 is set to limit problems with steel congestion and to ensure adequate member size for carrying shear that is governed by the flexural capacity of the member [Eq. (20.1)].

To obtain ductile performance, the location of lap splices is limited. They may not be used within joints, within twice the member depth from the face of a joint or at other locations where flexural yielding is expected as a result of lateral displacement of the frame. Lap splices must be enclosed by hoops or spirals with a maximum spacing of one-fourth of the effective depth or 4 in. Welded and mechanical connections may be used, provided that they are not used within a distance equal to twice the member depth from the face of a column or beam or sections where yielding of the reinforcement is likely to occur due to inelastic displacements under lateral load.

Transverse reinforcement is required throughout flexural members in frames resisting earthquake-induced forces. According to ACI Code 21.3.3, transverse reinforcement in the form of hoops must be used over a length equal to twice the member depth measured from the face of the supporting member toward midspan, at both ends of the flexural member, and over lengths equal to twice the member depth on both sides of a section where flexural yielding is likely to occur in connection with inelastic lateral displacements of the frame. The first hoop must be located not more than 2 in. from the face of the supporting member, and the maximum spacing of the hoops must not exceed one-fourth of the effective depth, 8 times the diameter of the smallest longitudinal bar, 24 times the diameter of the hoop bars, or 12 in.

To provide adequate support for longitudinal bars on the perimeter of a flexural member when the bars are placed in compression due to inelastic rotation, ACI Code 21.3.3 requires that hoops be arranged so that every corner and alternate longitudinal bar is provided lateral support by ties, in accordance with ACI Code 7.10.5.3. Arrangements meeting these criteria are illustrated in Fig. 8.2. Where hoops are not required, stirrups with seismic hooks at both ends must be provided throughout the member, with a maximum spacing of one-half of the effective depth. Hoops can be made up of a single reinforcing bar or two reinforcing bars consisting of a stirrup with seismic hooks at both ends and a crosstie. Examples of hoop reinforcement are presented in Figs. 7.11*a*, *c*–*e* and 20.11.

b. Members Subjected to Bending and Axial Load

To help ensure constructability and adequate confinement of the concrete, ACI Code 21.4.1 requires that members in frames designed to resist earthquake-induced forces, with a factored axial force exceeding $A_g f'_c/10$, have (a) a minimum cross-sectional dimension of at least 12 in. when measured on a straight line passing through the geometric centroid and (b) a ratio of the shortest cross-sectional dimension to the perpendicular dimension of at least 0.4.

To obtain a weak beam–strong column design, ACI Code 21.4.2 requires that the nominal flexural strengths of the columns framing into a joint exceed the nominal flexural strengths of the girders framing into the joint by at least 20 percent. This requirement is expressed as

$$\Sigma M_c \geq \frac{6}{5} \Sigma M_g \tag{20.18}$$

where ΣM_c = sum of moments at joint faces corresponding to nominal flexural strengths of columns framing into joint. Values of M_c are based on the factored axial load, consistent with the direction of the lateral forces, resulting in the lowest flexural strength.

ΣM_g = sum of moments at joint faces corresponding to nominal flexural strengths of girders framing into joint. In T-beam construction, where the slab is in tension under moment at the face of the joint, slab reinforcement within the effective flange width (see Section 3.8) is assumed to contribute to flexural strength if the slab reinforcement is developed at the critical section for flexure.

As shown in Fig. 20.8*b*, the flexural strengths are summed so that the column moments oppose the beam moments. Equation (20.18) must be satisfied for beam moments acting both clockwise and counterclockwise on the joint.

If Eq. (20.18) is not satisfied for beam moments acting in both directions, the columns must meet the minimum requirements for transverse reinforcement in ACI Code 21.4.4 (described below) over the full height of the member but may not be considered as adding strength or stiffness to the structure, if such additions assist in carrying earthquake-induced load. If, however, the stiffness of the columns increases the design base shear or the effects of torsion, they must be included in the analysis, but still may not be considered as contributing to structural capacity.

In accordance with ACI Code 21.4.3, the column reinforcement ratio based on the gross section ρ_g must meet the requirement: $0.01 \le \rho_g \le 0.06$. Welded splices and mechanical connections in columns must satisfy the same requirements specified for flexural members, whereas lapped splices must be designed for tension and are permitted only within the center half of columns.

ACI Code 21.4.4 specifies the use of minimum transverse reinforcement over length l_o from each joint face and on both sides of any section where flexural yielding is likely because of inelastic lateral displacement of the frame. The length l_o may not be less than (a) the depth of the member 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, or (c) 18 in.

Minimum transverse reinforcement is specified in terms of the ratio of the volume of the transverse reinforcement to the volume of the core confined by the reinforcement (measured out-to-out of the confining steel) ρ_s for spirals or circular hoop reinforcement as

$$\rho_s = 0.12 \frac{f_c'}{f_{yh}} \tag{20.19}$$

but not less than specified in Eq. (8.5), where f_{yh} is the specified yield strength of transverse reinforcement.

To provide similar confinement using rectangular hoop reinforcement, ACI Code 21.4.4 requires a minimum total cross-sectional area of transverse reinforcement A_{sh} along the length of the longitudinal reinforcement that may not be less than

$$A_{sh} = 0.3 \frac{sh_c f_c'}{f_{yh}} \left(\frac{A_g}{A_{ch}} - 1 \right) \tag{20.20}$$

or

$$A_{sh} = 0.09 \frac{sh_c f_c'}{f_{yh}} \tag{20.21}$$

where A_{ch} = cross-sectional area of column core, measured out-to-out of transverse reinforcement

s = spacing of transverse reinforcement

h_c = cross-sectional dimension of column core, measured center-to-center of confining reinforcement

Equations (20.20) and (8.5) need not be satisfied if the member core alone provides adequate strength to resist the earthquake effects. In accordance with ACI Code 21.4.4, the spacing of transverse reinforcement within l_o may not exceed one-quarter of the minimum member dimension, 6 times the diameter of the longitudinal bar, or

$$s_x = 4 + \frac{14 - h_x}{3} \tag{20.22a}$$

$$4 \text{ in.} \leq s_x \leq 6 \text{ in.} \tag{20.22b}$$

where h_x is the maximum horizontal spacing of hoop or crosstie legs on all faces of the column. The crossties or legs of overlapping hoops may not be spaced more than 14 in., as shown in Fig. 20.11.

For regions outside of l_o, when the minimum transverse reinforcement defined above is not provided, the spacing of spiral or hoop reinforcement may not exceed 6 times the diameter of the longitudinal column bars or 6 in.

To account for the major ductility demands that are placed on columns that support rigid members (see Figs. 20.4 and 20.5), the Code specifies that, for such columns, the minimum transverse reinforcement requirements must be satisfied throughout the *full column height* and that the transverse reinforcement must extend into the discontinued stiff member for at least the development length of the largest longitudinal reinforcement for walls and at least 12 in. into foundations.

EXAMPLE 20.1

Relative flexural strengths of members at a joint and minimum transverse column reinforcement. The exterior joint shown in Fig. 20.12 is part of a reinforced concrete frame designed to resist earthquake loads. A 6 in. slab, not shown, is reinforced with No. 5 (No. 16) bars spaced 10 in. center-to-center at the same level as the flexural steel in the beams. The member section dimensions and reinforcement are as shown. The frame story height is 12 ft. Material strengths are $f'_c = 4000$ psi and $f_y = 60{,}000$ psi. The maximum factored axial load on the upper column framing into the joint is 2210 kips, and the maximum factored axial load on the lower column is 2306 kips. Determine if the nominal flexural strengths of the columns exceed those of the beams by at least 20 percent, as required by Eq. (20.18), and determine the minimum transverse reinforcement required over the length l_o in the columns.

Solution. Checking the relative flexural strengths in the frame of the spandrel beams will be sufficient, since this is clearly the controlling case for the joint. In addition, because the beam reinforcement is the same on both sides of the joint, a single comparison will suffice for both clockwise and counterclockwise beam moments.

The negative nominal flexural strength of the beam at the joint is governed by the top steel, which consists of five No. 10 (No. 32) bars in the beams plus four No. 5 (No. 16) bars in the slab within the effective width of the top flange, $A_s = 6.35 + 1.24 = 7.59$ in^2. The yield force in the steel is

$$A_s f_y = 7.59 \times 60 = 455 \text{ kips}$$

The effective depth is $d = 36.0 - 1.5 - 0.5 - 1.27 \cdot 2 = 33.4$ in., and with stress block depth $a = 455 \cdot (0.85 \times 4 \times 27) = 4.96$ in., the nominal moment is

$$M_g = \frac{455}{12} \cdot 33.4 - \frac{4.96}{2} \cdot = 1172 \text{ ft-kips}$$

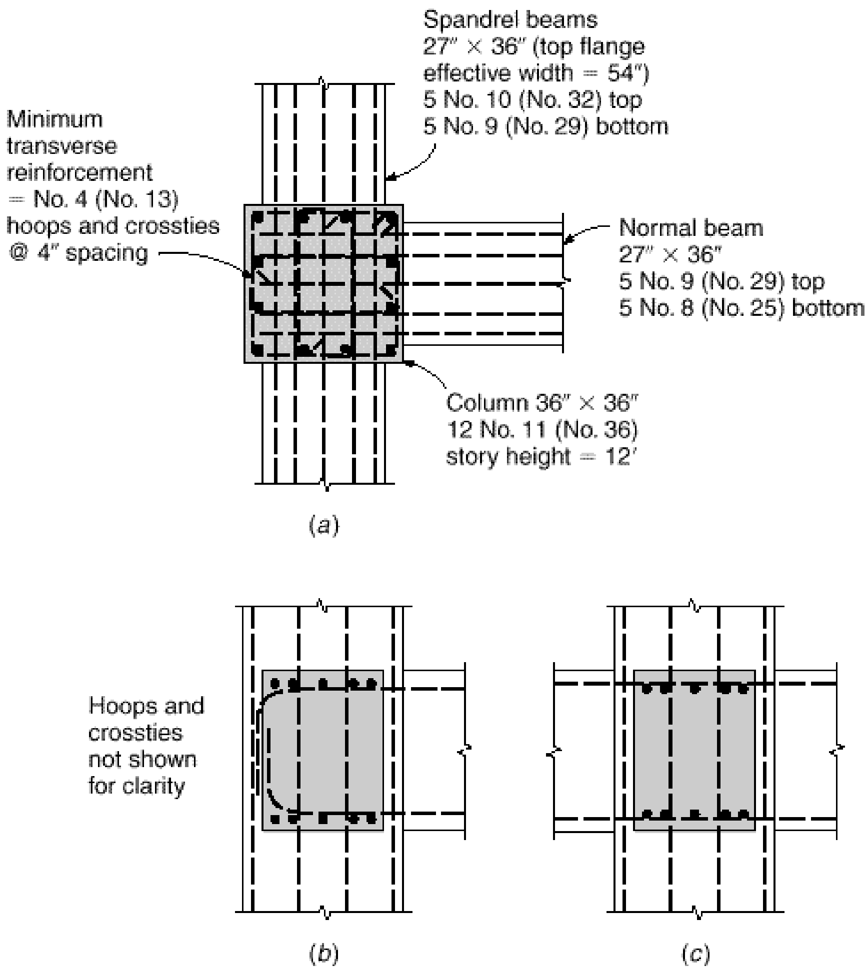

FIGURE 20.12
Exterior beam-column joint for Examples 20.1 and 20.2: (*a*) plan view; (*b*) cross section through spandrel beam; (*c*) cross section through normal beam. Note that confining reinforcement is not shown, except for column hoops and crossties in (*a*).

The positive nominal flexural strength of the beam at the joint is determined by the bottom steel, five No. 9 (No. 29) bars, $A_s = 5.00$ in^2. The yield force in the steel is

$$A_s f_y = 5.00 \times 60 = 300 \text{ kips}$$

The effective depth is $d = 36.0 - 1.5 - 0.5 - 1.128/2 = 33.4$ in., and with stress block depth $a = 300/(0.85 \times 4 \times 54) = 1.63$ in., the nominal moment is

$$M_g = \frac{300}{12}\left(33.4 - \frac{1.63}{2}\right) = 815 \text{ ft-kips}$$

The minimum nominal flexural strengths of the columns in this example depend on the maximum factored axial loads, which are 2210 and 2306 kips for the upper and lower columns, respectively. For the 36 × 36 in. columns, this gives

$$\frac{P_u}{f_c' A_g} = \frac{2210}{4 \times 1296} = 0.426 \qquad \text{upper column}$$

$$\frac{P_u}{f_c' A_g} = \frac{2306}{4 \times 1296} = 0.445 \qquad \text{lower column}$$

With total reinforcement of 12 No. 11 (No. 36) bars, $A_{st} = 18.72$ in^2 and the reinforcement ratio $\rho_g = 18.72/1296 = 0.0144$. Using cover to the center of the bars of 3 in.,

· = (36 − 6)· 36 = 0.83, Graphs A.7 and A.8 in Appendix A are appropriate for determining the flexural capacity.

For the upper column,

$$R_n = \frac{M_c}{f_c' A_g h} = 0.167$$

$$M_c = 0.167 \times 4 \times 1296 \times \frac{36}{12} = 2597 \text{ ft-kips}$$

For the lower column,

$$R_n = \frac{M_c}{f_c' A_g h} = 0.164$$

$$M_c = 0.164 \times 4 \times 1296 \times \frac{36}{12} = 2550 \text{ ft-kips}$$

Checking the relative flexural capacities,

$$\cdot\ M_c = 2597 + 2550 = 5147 \text{ ft-kips}$$

$$\cdot\ M_g = 1172 + 815 = 1987 \text{ ft-kips}$$

By inspection, · $M_c \geq \frac{6}{5}$ · M_g.

Minimum transverse reinforcement is required over a length l_o on either side of the joint. According to ACI Code 21.4.4, l_o is the greater of (a) the depth $h = 36$ in., (b) one-sixth of the clear span = (12 × 12 − 36)· 6 = 18 in., or (c) 18 in. Since every corner and alternate longitudinal bar must have lateral support and because the spacing of crossties and legs of hoops is limited to a maximum of 14 in. within the plane of the transverse reinforcement, the scheme shown in Fig. 20.12*a* will be used, giving a maximum spacing of slightly less than 12.5 in. The maximum spacing of transverse reinforcement s is limited to the smaller of one-quarter of the minimum member dimension = 36· 4 = 9 in., 6 times the diameter of the longitudinal bar, 6 × 1.41 = 8.46 in., or

$$s_x = 4 + \frac{14 - 12.5}{3} = 4.5 \text{ in.}$$

with 4 in. ≤ s_x ≤ 6 in. A 4 in. spacing will be used.

Using No. 4 (No. 13) bars, the cross-sectional dimension of the column core, center-to-center of the confining steel, is $h_c = 32.5$ in., and the cross-sectional area of column core, out-to-out of the confining steel, is $A_{ch} = 33 \times 33 = 1089$ in^2.

For $f_{yh} = 60$ ksi, the total area of transverse reinforcement with the 4 in. spacing is the larger of Eqs. (20.20) and (20.21).

$$A_{sh} = 0.3\, \frac{4 \times 32.5 \times 4}{60} \cdot \frac{1296}{1089} - 1 \cdot = 0.49 \text{ in}^2$$

$$A_{sh} = 0.09\, \frac{4 \times 32.5 \times 4}{60} = 0.78 \text{ in}^2$$

The requirement for 0.78 in^2 is satisfied by four No. 4 (No. 13) bar legs.

c. Joints and Development of Reinforcement

The design of beam-column joints is discussed in Section 11.2. The forces acting on a joint subjected to lateral loads are illustrated in Fig. 11.4. The factored shear acting on a joint is

$$
\begin{aligned}
V_u &= T_1 + C_2 - V_{col} \\
&= T_1 + T_2 - V_{col}
\end{aligned}
\tag{20.23}
$$

where T_1 = tensile force in negative moment beam steel on one side of a joint
T_2 = tensile force in positive moment beam steel on one side of a joint
C_2 = compressive force counteracting T_2
V_{col} = shear in the column at top and bottom faces of the joint corresponding to the net moment in the joint and points of inflection at midheight of columns (see Fig. 11.5)

For seismic design, the forces T_1 and T_2 (= C_2) must be based on a stress in the flexural tension reinforcement of $1.25f_y$. In accordance with ACI Code 21.5.3, the nominal shear capacity of a joint depends on the degree of confinement provided by members framing into the joint.

For joints confined on all four faces	$20 \cdot \sqrt{f'_c}A_j$
For joints confined on three faces or two opposite faces	$15 \cdot \sqrt{f'_c}A_j$
For others	$12 \cdot \sqrt{f'_c}A_j$

where A_j is the effective cross-sectional area of the joint in a plane parallel to the plane of reinforcement generating shear in the joint. The joint depth is the overall depth of the column. For beams framing into a support of larger width, the effective width of the joint is the smaller of (a) beam width plus joint depth or (b) twice the smaller perpendicular distance from the longitudinal axis of the beam to the column side. The effective area of a joint is illustrated in Fig. 20.13. The nominal shear strength for lightweight aggregate concrete is limited to three-quarters of the values given above.

To provide adequate confinement within a joint, the transverse reinforcement used in columns must be continued through the joint, in accordance with ACI Code 21.5.2. This reinforcement may be reduced by one-half within the depth of the shallowest framing member and the spacing of spirals or hoops may be increased to 6 in., if beams or girders frame into all four sides of the joint and the flexural members cover at least three-fourths of the column width.

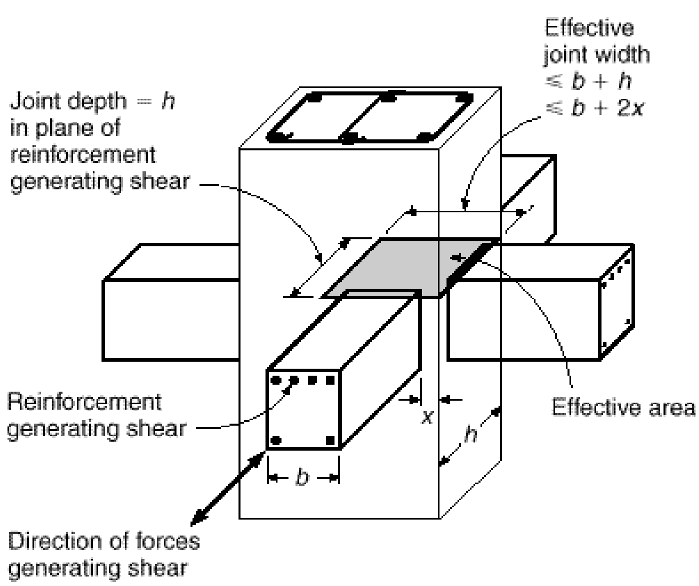

FIGURE 20.13
Effective area of joint A_j, which must be considered separately for forces in each direction of framing. Note that the joint illustrated does not meet conditions necessary to be considered as confined because the forming members do not cover at least $\frac{3}{4}$ of each joint face. *(Adapted from Ref. 20.10.)*

For joints where the beam is wider than the column, transverse reinforcement, as required for columns (ACI Code 21.4.4), must be provided to confine the flexural steel in the beam, unless confinement is provided by a transverse flexural member.

To provide adequate development of beam reinforcement passing through a joint, ACI Code 21.5.1 requires that the column dimension parallel to the beam reinforcement must be at least 20 times the diameter of the largest longitudinal bar for normal-weight concrete and 26 times the bar diameter for lightweight concrete. For beam longitudinal reinforcement that is terminated within a column, both hooked and straight reinforcement must be extended to the far face of the column core. The reinforcement must be anchored in compression as described in Section 5.7 (ACI Code Chapter 12) and anchored in tension in accordance with ACI Code 21.5.4, which requires that the development length of bars with 90° hooks l_{dh} must be not less than $8d_b$, 6 in., or

$$l_{dh} = \frac{f_y d_b}{65\sqrt{f'_c}} \tag{20.24}$$

For lightweight aggregate concrete, these values are, respectively, $10d_b$, 7.5 in., and 1.25 times the value in Eq. (20.24). The 90° hook must be located within the confined core of the column.

For straight bars anchored within a column core, the development length l_d of bottom bars must be at least 2.5 times the value required for hooks; l_d for top bars must be at least 3.5 times the length required for hooks.

According to ACI Code 21.5.4, straight bars that are terminated at a joint must pass through the confined core of a column or a boundary element (discussed in Section 20.6). Because of the lower degree of confinement provided outside of the confined region, the Code requires that any portion of the straight embedment length that is not within the core must be increased by a factor of 1.6. Thus, the required development length l_{dm} of a bar that is not entirely embedded in confined concrete is

$$l_{dm} = 1.6(l_d - l_{dc}) + l_{dc} \tag{20.25a}$$

$$l_{dm} = 1.6l_d - 0.6l_{dc} \tag{20.25b}$$

where l_d = required development length for a straight bar embedded in confined concrete

l_{dc} = length embedded in confined concrete

EXAMPLE 20.2 **Design of exterior joint.** Design the joint shown in Fig. 20.12.

Solution. As discussed in Chapter 11, a joint must be detailed so that the beam and column bars do not interfere with each other and so that placement and consolidation of the concrete is practical. Bar placement is shown in Fig. 20.12.

Development of the spandrel beam flexural steel within the joint is checked based on the requirement that the column dimension must be at least 20 times the bar diameter of the largest bars. This requirement is met for the No. 10 (No. 32) bars used as top reinforcement.

$$20 \times 1.27 = 25.4 \text{ in.} < 36 \text{ in.}$$

The flexural steel in the normal beam must be anchored within the core of the column based on Eq. (20.24), but not less than $8d_b$ or 6 in. For the No. 9 (No. 29) top bars, Eq. (20.24) controls

$$l_{dh} = \frac{60{,}000 \times 1.128}{65\sqrt{4000}} = 16.5 \text{ in.}$$

The same holds true for the No. 8 (No. 25) bottom bars, which must also be anchored in tension (ACI Code 12.11.2) because lateral loading will subject the beam to both positive and negative bending moments at the exterior joint.

$$l_{dh} = \frac{60{,}000 \times 1.0}{65\sqrt{4000}} = 14.6 \text{ in.}$$

Since $3.5l_{dh}$ is not available for the top bars and $2.5l_{dh}$ is not available for the bottom bars, all flexural steel from the normal beam must be anchored using hooks, not straight reinforcement, extended to the far face of the column core, as shown in Fig. 20.12*b*.

To check the shear strength of the joint, the shear forces acting on the joint must be calculated based on a stress of $1.25f_y$ in the flexural reinforcement. By inspection, shear in the plane of the spandrel beam will control.

The tensile force in the negative steel is

$$T_1 = 1.25 \times 6.35 \times 60 = 476 \text{ kips}$$

For an effective depth of 34.4 in. (Example 20.1) and a depth of stress block $a = 476/(0.85 \times 4 \times 27) = 5.19$ in., the moment due to negative bending is

$$M^- = \frac{476}{12}\left(33.4 - \frac{5.19}{2}\right) = 1222 \text{ ft-kips}$$

For positive bending on the other side of the column,

$$T_2 = 1.25 \times 5.00 \times 60 = 375 \text{ kips}$$

$$a = \frac{375}{0.85 \times 4 \times 54} = 2.04 \text{ in.}$$

$$M^+ = \frac{375}{12}\left(33.4 - \frac{2.04}{2}\right) = 1012 \text{ ft-kips}$$

The column shear corresponding to the sum of the moments M^+ and M^- and based on the free body of the column between assumed midheight inflection points, as shown in Fig. 20.14*a*, is $V_{col} = (1222 + 1012)/12 = 186$ kips. The shear forces acting on the joint are shown in Fig. 20.14*b*, and the factored joint shear is

$$V_u = T_1 + T_2 - V_{col} = 476 + 375 - 186 = 665 \text{ kips}$$

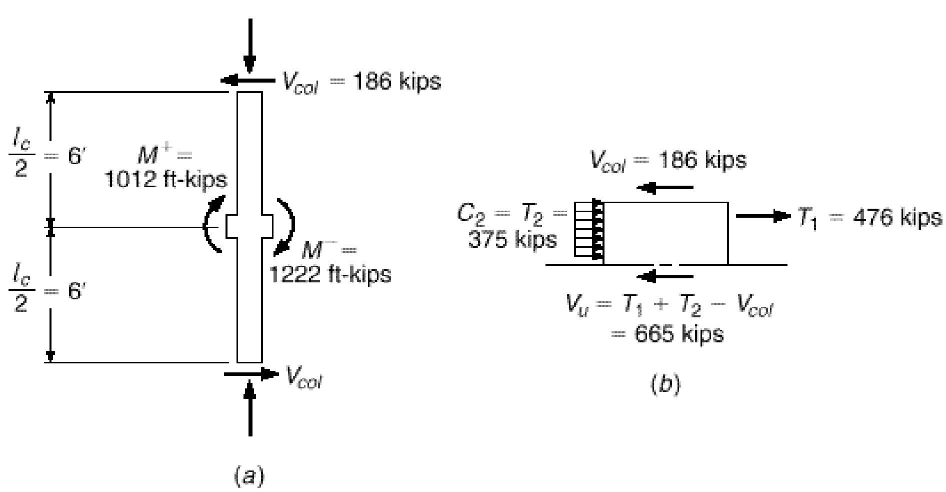

FIGURE 20.14
Free-body diagrams in plane of spandrel beam for Example 20.2: (*a*) column and joint region; (*b*) forces acting on joint due to lateral load.

For a joint confined on three faces with an effective cross-sectional area $A_j = 36 \times 36 = 1296$ in^2, the nominal and design capacities of the joint are

$$V_n = 15\sqrt{f'_c}\,A_j = \frac{15\sqrt{4000} \times 1296}{1000} = 1229 \text{ kips}$$

$$\phi V_n = 0.85 \times 1229 = 1045 \text{ kips}$$

Since $\phi V_n > V_u$, the joint is satisfactory for shear.

Because the joint is not confined on all four sides, the transverse reinforcement in the column must be continued, unchanged, through the joint.

d. Members Not Proportioned to Resist Earthquake Forces

Frame members in structures designed for seismic loading that are not proportioned to carry earthquake forces must still be able to support the factored gravity loads [see Eqs. (20.4) and (20.5)] for which they are designed as the structures undergo lateral displacement. To provide adequate strength and ductility, ACI Code 21.11.1 requires that these members be designed based on moments corresponding to the design displacement, which ACI Commentary R21.11 suggests should be based on models that will provide a conservatively large estimate of displacement. In this case, ACI Code 20.11.2 permits the load factor for live load L to be reduced to 0.5, except for garages, places of public assembly, and areas where $L > 100$ psf.

When the induced moments and shears, combined with the factored gravity moments and shears (see Table 1.2), do not exceed the design capacity of a frame member, ACI Code 21.11.2 requires that members with factored gravity axial forces below $A_g f'_c/10$ contain minimum longitudinal top and bottom reinforcement as provided in Eq. (3.41), a reinforcement ratio not greater than 0.025, and at least two continuous bars top and bottom. In addition, stirrups are required with a maximum spacing of $d/2$ throughout.

For members with factored gravity axial forces exceeding $A_g f'_c/10$, the longitudinal reinforcement must meet the requirements for columns proportioned for earthquake loads, and the transverse reinforcement must consist of hoops and crossties, as used in columns designed for seismic loading [as required by ACI Code 21.4.4.1(c) and 21.4.4.3]. The maximum longitudinal spacing of the transverse reinforcement s_o may not be more than 6 times the diameter of the smallest longitudinal bar or 6 in. throughout the column height. In addition, the transverse reinforcement must carry shear induced by inelastic rotation at the ends of the member, as required by ACI Code 21.4.5 (discussed in Section 20.7). Members with factored gravity axial forces exceeding 35 percent of the axial capacity without eccentricity $0.35P_o$ must be designed with transverse reinforcement equal to at least one-half of that specified in ACI Code 21.4.4.1 [see Eqs. (20.20), (20.21), and (20.23)].

If the induced moments or shears under the design lateral displacements exceed the design moment or shear strengths, or if such a calculation is not made, ACI Code 21.11.3 requires that the members meet the material criteria for concrete and steel in ACI Code 21.2.4 and 21.2.5 (see Section 20.4), along with criteria for mechanical and welded splices (ACI Code 21.2.6 and 21.2.7.1, respectively). For frame members with factored gravity axial loads below $A_g f'_c/10$, the minimum reinforcement criteria specified in ACI Code 21.7.2 must be met, along with the requirement that the shear capacity of the member must be adequate to carry forces induced by flexural yielding under the criteria of ACI Code 21.3.4 [see Fig. 20.16 and Eq. (20.28) in Section 20.7]. For members with factored gravity axial forces exceeding $A_g f_c'/10$, the longitudinal reinforcement ratio ρ_g must be within the range 0.01 to 0.06 and all requirements for transverse reinforcement and shear capacity specified for columns designed for earthquake-induced lateral loading must be satisfied. In addition, the transverse column reinforcement must be continued within the joints, as required by ACI Code 21.5.2 (see Section 20.5c) for frames in zones of high seismic risk.

20.6 ACI Provisions for Special Structural Walls, Coupling Beams, Diaphragms, and Trusses

ACI Code Chapter 21 includes requirements for stiff structural systems and members that carry earthquake forces or distribute earthquake forces between portions of structures that carry earthquake forces. Structural walls, coupling beams, diaphragms, trusses, struts, ties, chords, and collector elements are in this category. The general requirements for these members are presented in this section. The requirements for shear design are presented in Section 20.7c.

a. Structural Walls

To ensure adequate ductility, ACI Code 21.7.2 requires that structural walls have minimum shear reinforcement ratios in both the longitudinal and transverse directions ρ_v and ρ_n of 0.0025 and a maximum reinforcement spacing of 18 in. If the shear force assigned to a wall exceeds $2A_{cv}\sqrt{f_c'}$, where A_{cv} is the net area of the concrete section bounded by the web thickness and the length of the section in the direction of the factored shear force, at least two curtains of reinforcement must be used. If, however, the factored shear is not greater than $A_{cv}\sqrt{f_c'}$, the minimum reinforcement criteria of ACI Code 14.3 govern.

Boundary elements are added along the edges of structural walls and diaphragms to increase strength and ductility. The elements include added longitudinal and transverse reinforcement and may lie entirely within the thickness of the wall or may require a larger cross section, as shown in Fig. 20.15. Under certain conditions, openings must be bordered by boundary elements. For walls that are continuous from the base of the structure to the top of a wall, compression zones must be reinforced with special boundary elements when the depth to the neutral axis c exceeds the value given in Eq. (20.26).

$$c \geq \frac{l_w}{600(\delta_u/h_w)} \tag{20.26}$$

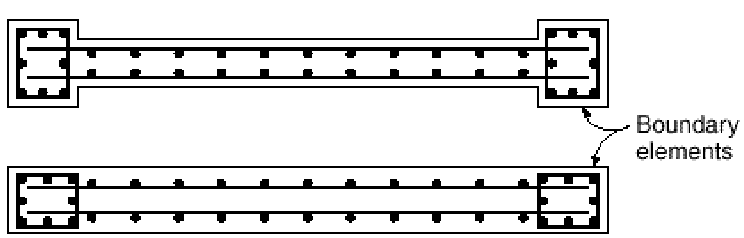

FIGURE 20.15
Cross sections of structural walls with boundary elements.

where l_w and h_w are the length and width of the wall, respectively, and δ_u is the design displacement. In Eq. (20.26), δ_u/h_w is not taken greater than 0.007. When special boundary elements are required based on Eq. (20.26), the reinforcement in the boundary element must be extended vertically from the critical section a distance equal to the greater of l_w or $M_u/4V_u$.

Structural walls are also required to have boundary elements at boundaries and around openings where the maximum extreme fiber compressive stress under factored loads exceeds $0.2\sqrt{f'_c}$. Stresses are calculated based on a linear elastic model using the gross cross section [$\sigma = (P/A) \pm (My/I)$]. The boundary elements may be discontinued once the calculated compressive stress drops below $0.15f'_c$. The confinement provided by the boundary element increases both the ductility of the wall and its ability to carry repeated cycles of loading. When required, the boundary element must extend horizontally from the extreme compressive fiber a distance not less than $c - 0.1l_w$ or $c/2$, whichever is greater. When flanged sections are used, the boundary element is defined based on the effective flange width and extends at least 12 in. into the web. Transverse reinforcement within the boundary element must meet the requirements for columns in ACI Code 21.4.1 through 21.4.3 (discussed in Section 20.5b), but need not meet the requirements in Eq. (20.20). The transverse reinforcement within a boundary element must extend into the support a distance equal to at least the development length of the largest longitudinal reinforcement, except where the boundary element terminates at a footing or mat, in which case the transverse reinforcement must extend at least 12 in. into the foundation. Horizontal reinforcement in the wall web must be anchored within the confined core of the boundary element, a requirement that usually requires standard 90° hooks or mechanical anchorage.

When boundary elements are not required and when the longitudinal reinforcement ratio in the wall boundary is greater than $400/f_y$, the transverse reinforcement at the boundary must consist of hoops at the wall boundary with crossties or legs that are not spaced more than 14 in. on center extending into the wall a distance of $c - 0.1l_w$ or $c/2$, whichever is greater, at a spacing of not greater than 8 in. The transverse reinforcement in such cases must be anchored with a standard hook around the edge reinforcement, or the edge reinforcement must be enclosed in U stirrups of the same size and spacing as the transverse reinforcement. This requirement need not be met if the maximum shear force is less than $A_{cv}\sqrt{f'_c}$.

b. Coupling Beams

Coupling beams connect structural walls, as shown in Fig. 20.16*a*. Under lateral loading, they can increase the stiffness of the structure and dissipate energy. Deeper coupling beams can be subjected to significant shear, which is carried effectively by diagonal reinforcement. According to ACI Code 21.7.7, coupling beams with clear span to total depth ratios l_n/h of 4.0 or greater may be designed using the criteria for flexural members described in Section 20.5a. In this case, however, the limitations on width-to-depth ratio and total width for flexural members need not be applied if it can be shown by analysis that the beam has adequate lateral stability. Coupling beams with $l_n/h < 4$ may be reinforced using two intersecting groups of diagonally placed bars that are symmetrical about the midspan (Fig. 20.16*b*). Such reinforcement is not effective unless it is placed at a steep angle (Refs. 20.12 and 20.13) and, thus, is not permitted for coupling beams with $l_n/h \geq 4$. Coupling beams with l_n/h less than 2 and a factored shear

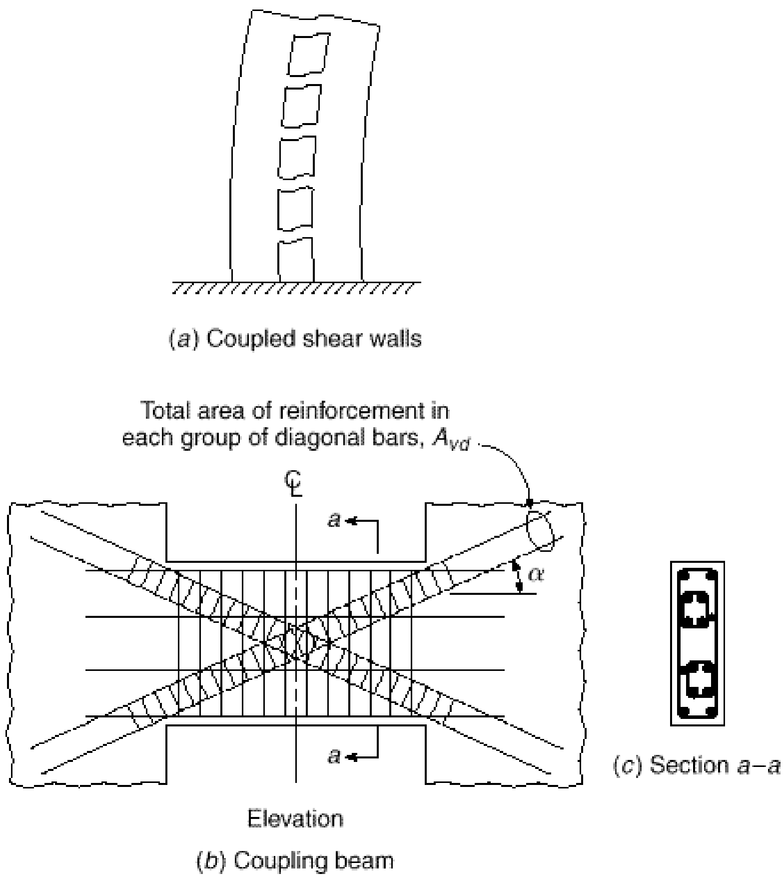

FIGURE 20.16
Coupled shear walls and coupling beam. (*b and c adapted from Ref. 20.10.*)

$V_u > 4\sqrt{f_c'}A_{cp}$, where A_{cp} is the concrete area resisting shear, must be reinforced with two intersecting groups of diagonal reinforcement, as shown in Fig. 20.16*b*, unless it can be shown that the loss of stiffness and strength in the beams will not impair the vertical load-carrying capacity of the structure, egress from the structure, or the integrity of nonstructural components and their connections to the structure. The criteria for shear reinforcement in coupling beams are discussed in Section 20.7c.

c. Structural Truss Elements, Struts, Ties, and Collector Elements

To provide adequate confinement and ductility, structural truss elements, struts, ties, and collector elements with compressive stresses greater than $0.2f_c'$ must meet the same transverse reinforcement requirements as columns in seismic load–resisting frames, but over the full length of the elements. The special transverse reinforcement may be discontinued at a section where the calculated compressive stress is less than $0.15f_c'$. Compressive stresses in these elements are calculated for the factored forces using a linear elastic model and the gross section properties of the elements. Continuous reinforcement in stiff structural systems must be anchored and spliced as required by ACI Code 21.5.4 [see Eqs. (20.14) and (20.15)].

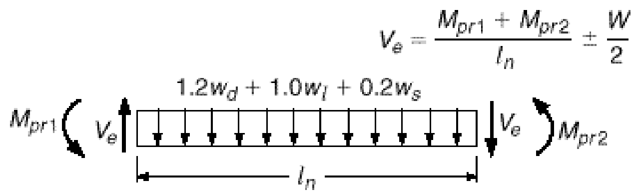

FIGURE 20.17
Forces considered in the shear design of flexural members subjected to seismic loading. $W/2$ is the shear corresponding to gravity loads based on $1.2D + 1.0L + 0.2S$.

d. Structural Diaphragms

Floors and roofs serve as structural diaphragms in buildings. In addition to supporting vertical dead, live, and snow load, they connect and transfer lateral forces between the members in the vertical lateral force–resisting system and support other building elements, such as partitions, that may resist horizontal forces but do not act as part of the vertical lateral force–resisting system. Floor and roof slabs that act as diaphragms may be monolithic with the other horizontal elements in the structures or may include a topping slab. ACI Code 21.9.4 requires that concrete slabs and composite topping slabs designed as structural diaphragms to transmit earthquake forces must be at least 2 in. thick. Topping slabs placed over precast floor or roof elements that do not rely on composite action must be at least $2\frac{1}{2}$ in. thick.

20.7 ACI Provisions for Shear Strength

a. Beams

A prime concern in the design of seismically loaded structures is the shear induced in members due to nonlinear behavior in flexure [Eq. (20.1)]. As discussed in Section 20.2, increasing the flexural strength of beams and columns may increase the shear in these members if the structure is subjected to severe lateral loading. As a result, the ACI Code requires that beams and columns in frames that are part of a lateral load–resisting system (including some members that are not designed to carry lateral loads) be designed for the combined effects of factored gravity load and shear induced by the formation of plastic hinges at the ends of the members.

For members with axial loads less than $A_g f'_c/10$, ACI Code 21.3.4 requires that the design shear force V_e be based on the factored tributary gravity load along the span plus shear induced by moments of opposite sign corresponding to the "probable flexural strength" M_{pr}. Loading corresponding to this case is shown in Fig. 20.17. The probable flexural strength M_{pr} is based on the reinforcing steel achieving a stress of $1.25f_y$.

$$M_{pr} = 1.25A_s f_y \left(d - \frac{a}{2}\right) \tag{20.27a}$$

where

$$a = \frac{1.25 f_y A_s}{0.85 f_c b} \tag{20.27b}$$

The shear V_e is given by

$$V_e = \frac{M_{pr1} + M_{pr2}}{l_n} \pm \frac{W}{2} \tag{20.28}$$

where M_{pr1} and M_{pr2} = probable moment strengths at two ends of member when moments are acting in the same sense
l_n = length of member between faces of supports
$W \cdot 2$ = effect of factored gravity loads at each end of member (based on $1.2D + 1.0L + 0.2S$)

Equation (20.28) should be evaluated separately for moments at both ends acting in the clockwise and then counterclockwise directions.

To provide adequate ductility and concrete confinement, the transverse reinforcement over a length equal to twice the member depth from the face of the support, at both ends of the flexural member, must be designed based on a concrete shear capacity $V_c = 0$, when the earthquake-induced shear force in Eq. (20.28) $(M_{pr1} + M_{pr2}) \cdot l_n$ is one-half or more of the maximum required shear strength within that length and the factored axial compressive force in the member, including earthquake effects, is below $A_g f'_c \cdot 20$.

EXAMPLE 20.3

Beam shear design. An 18 in. wide by 24 in. deep reinforced concrete beam spans between two interior columns in a building frame designed for a region of high seismic risk. The clear span is 24 ft and the reinforcement at the face of the support consists of four No. 10 (No. 32) top bars and four No. 8 (No. 25) bottom bars. The effective depth is 21.4 in. for both top and bottom steel. The maximum factored shear $1.2V_d + 1.0V_l$ is 32 kips at each end of the beam. Materials strengths are $f'_c = 5000$ psi and $f_y = 60{,}000$ psi. Design the shear reinforcement for the regions adjacent to the column faces.

Solution. The probable moment strengths M_{pr} are based on a steel stress of $1.25 f_y$. For negative bending, the area of steel is $A_s = 5.08$ in^2 at both ends of the beam, the stress block depth is $a = 1.25 \times 5.08 \times 60 \cdot (0.85 \times 5 \times 18) = 4.98$ in., and the probable strength is

$$M_{pr1} = \frac{1.25 \times 5.08 \times 60}{12} \cdot 21.4 - \frac{4.98}{2} \cdot = 600 \text{ ft-kips}$$

For positive bending, the area of steel is $A_s = 3.16$ in^2, the effective width is 90 in., the stress block depth $a = 1.25 \times 3.16 \times 60 \cdot (0.85 \times 5 \times 90) = 0.62$ in., and the probable strength is

$$M_{pr2} = \frac{1.25 \times 3.16 \times 60}{12} \cdot 21.4 - \frac{0.62}{2} \cdot = 417 \text{ ft-kips}$$

The effect of factored gravity loads $W \cdot 2 = 1.2D + 1.0L = 32$ kips, giving a design shear force at each end of the beam, according to Eq. (20.28), of

$$V_e = \frac{600 + 417}{24} + 32 = 42 + 32 = 74 \text{ kips}$$

Since the earthquake-induced force, 42 kips, is greater than one-half of the maximum required shear strength, the transverse hoop reinforcement must be designed to resist the full value of V_e (i.e., $\cdot\ V_s \geq V_e$) over a length $2h = 48$ in. from the face of the column, in accordance with ACI Code 21.3.3. The maximum spacing of the hoops s is based on the smaller of $d \cdot 4 = 5.4$ in., $8d_b$ for the smallest longitudinal bars = 8 in., or $24d_b$ for the hoop bars [assumed to be No. 3 (No. 10) bars] = 9 in., or 12 in. A spacing $s = 5$ in. will be used.

The area of shear reinforcement within a distance s is

$$A_v = \frac{\cdot V_e \cdot \cdot\ \cdot s}{f_{yt} d} = \frac{\cdot 74 \cdot 0.75 \cdot 5}{60 \times 21.4} = 0.38 \text{ in}^2$$

Providing support for corner and alternate longitudinal bars, in accordance with ACI Code 21.3.3, leads to the use of overlapping hoop reinforcement, shown in Fig. 20.18, and a total area of transverse steel $A_v = 0.44$ in^2.

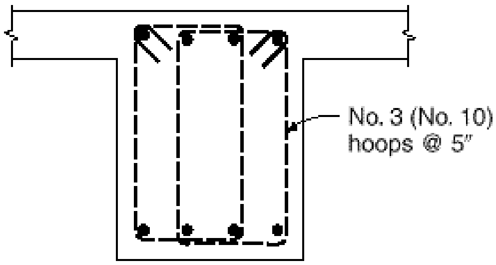

FIGURE 20.18
Configuration of hoop reinforcement for beam in Example 20.3.

The first hoop is placed 2 in. from the face of the column. The other hoops are spaced at 5 in. within 48 in. from each column face. Transverse reinforcement for the balance of the beam is calculated based on the value of V_e at that location and a nonzero concrete contribution V_c. The stirrups must have seismic hooks and a maximum spacing of $d/2$.

b. Columns

In accordance with ACI Code 21.4.5, shear provisions similar to those used for beams to account for the formation of inelastic hinges must also be applied to members with axial loads greater than $A_g f'_c/10$. In this case, the loading is illustrated in Fig. 20.19*a*, and the factored shear is

$$V_e = \frac{M_{pr1} + M_{pr2}}{l_u} \tag{20.29}$$

where l_u is the clear distance between beams, and M_{pr1} and M_{pr2} are based on a steel tensile strength of $1.25 f_y$.

In Eq. (20.29), M_{pr1} and M_{pr2} are the maximum probable moment strengths for the range of factored axial loads to which the column will be subjected, as shown in Fig. 20.19*b*; V_e, however, need not be greater than a value based on M_{pr} for the transverse members framing into the joint. For most frames, the latter will control. Of course, V_e may not be less than that obtained from the analysis of the structure under factored loads.

The ACI Code requires that the transverse reinforcement in a column over a length l_o (the greater of the depth of the member at the joint face, one-sixth of the clear span, or 18 in.) from each joint face must be proportioned to resist shear based on a concrete shear capacity $V_c = 0$ when (a) the earthquake-induced shear force is one-half or more of the maximum required shear strength within those lengths and (b) the factored axial compressive force, including earthquake effects, is less than $A_g f'_c/20$.

c. Walls, Coupling Beams, Diaphragms, and Trusses

According to ACI Code 21.7.3 and 21.9.6, the factored shear force V_u for walls, coupling beams, diaphragms, and trusses must be obtained from analysis based on the factored (including earthquake) loads.

In accordance with ACI Code 21.7.4, the nominal shear strength V_n of structural walls and diaphragms is taken as

$$V_n = A_{cv}\left(\alpha_c \sqrt{f'_c} + \rho_n f_y\right) \tag{20.30}$$

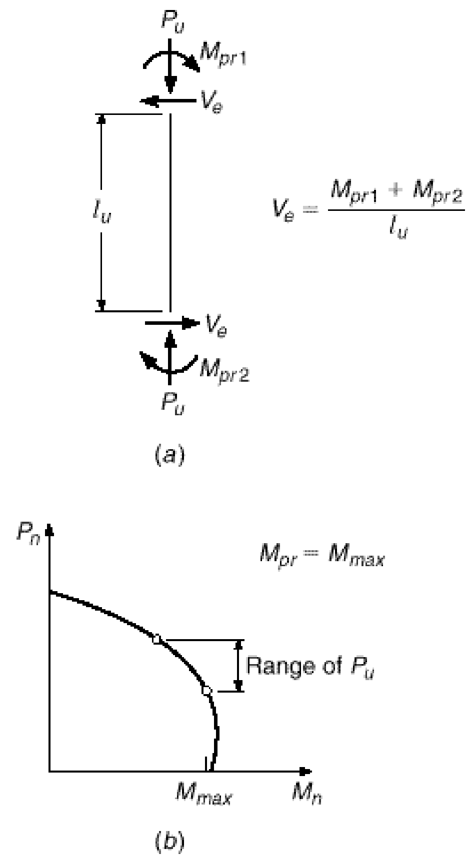

FIGURE 20.19
(*a*) Forces considered in the shear design of columns subjected to seismic loading. (*b*) Column interaction diagram used to determine maximum probable moment strengths. Note that M_{pr} for columns is usually governed by M_{pr} of the girders framing into a joint, rather than M_{max}.

where A_{cv} = net area of concrete section bounded by the web thickness and length of the section in the direction of shear force

ρ_n = ratio of distributed shear reinforcement on a plane perpendicular to the plane of A_{cv}

α_c = 3.0 for $h_w/l_w \leq 1.5$, = 2.0 for $h_w/l_w \geq 2.0$, and varies linearly for intermediate values of h_w/l_w

The values of h_w and l_w used to calculate α_c are the height and length, respectively, of the entire wall or diaphragm or segments of the wall or diaphragm. l_w is measured in the direction of the shear force. In applying Eq. (20.30), the ratio h_w/l_w is the larger of the ratios for the entire member or the segment of the member being considered. The use of α_c greater than 2.0 is based on the higher shear strength observed for walls with low aspect ratios.

As described in Section 20.6, ACI Code 21.7.2 requires that walls and diaphragms contain distributed shear reinforcement in two orthogonal directions in the plane of the member. For $h_w/l_w \leq 2.0$, the reinforcement ratio for steel crossing the plane of A_{cv}, ρ_v, must at least equal ρ_n. The nominal shear strength of all wall piers (vertical regions of a wall separated by openings) that together carry the lateral force is limited to a maximum value of $8A_{cv}\sqrt{f'_c}$, with no individual pier assumed to carry greater than $10A_{cp}\sqrt{f'_c}$, where A_{cv} is the total cross-sectional area and A_{cp} is the cross-sectional area of an individual pier. The nominal shear strength of horizontal wall segments (regions of a wall bounded by openings above and below) and coupling beams is limited to $10A_{cp}\sqrt{f'_c}$.

For coupling beams reinforced with two intersecting groups of diagonally placed bars symmetrical about the midspan (Fig. 20.16*b*), each group of the diagonally placed bars must consist of at least four bars assembled in a core, with sides measured to the outside of the transverse reinforcement that are no smaller than $0.5b_w$ perpendicular to the plane of the beam and $0.2b_w$ in the plane of the beam and perpendicular to the diagonal bars. The nominal strength provided by the diagonal bars is given by

$$V_n = 2A_{vd} f_y \sin\alpha \le 10\sqrt{f'_c}A_{cp} \tag{20.31}$$

where A_{vd} = total area of longitudinal reinforcement in an individual diagonal
A_{cp} = area of concrete section resisting shear
α = angle between diagonal reinforcement and longitudinal axis of coupling beam

The upper limit in Eq. (20.31) is a safe upper bound based on the experimental observation that coupling beams remain ductile at shear forces exceeding this value (Ref. 20.13). Each group of the diagonally placed bars must be enclosed in transverse reinforcement meeting the requirements for columns in ACI Code 21.4.4.1 through 21.4.4.3, discussed in Section 20.5b. The diagonal bars must be developed for tension in the wall and must be considered when calculating the nominal flexural strength of the coupling beam. In this case, the horizontal component of the bar force $A_{vd} f_y \cos\alpha$ should be used to calculate M_n. Longitudinal and transverse reinforcement must be added, as shown in Fig. 20.16*b* to satisfy the requirements for distributed horizontal and vertical reinforcement specified for deep beams in ACI Code 11.8.4 and 11.8.5 (see Section 10.4d).

According to ACI Code 21.9.7, the maximum nominal shear strength of diaphragms is given by Eq. (20.30) with $\alpha_c = 2.0$. For diaphragms consisting of either cast-in-place composite or noncomposite topping slabs, the maximum shear force may not exceed

$$V_n = A_{cv}\rho_n f_y \tag{20.32}$$

where A_{cv} is based on the thickness of the topping slab. Web reinforcement in the diaphragm is distributed uniformly in both directions. Finally, V_n may not exceed $8A_{cv}\sqrt{f'_c}$, where A_{cv} is the gross cross-sectional area of the diaphragm.

20.8 ACI Provisions for Intermediate Moment Frames in Regions of Moderate Seismic Risk

ACI Code 21.12 governs the design of frames for moderate seismic risk. The requirements include specified loading and detailing requirements. Unlike regions of high seismic risk, two-way slab systems without beams are allowed to serve as lateral load–resisting systems. Walls, diaphragms, and trusses in regions of moderate seismic risk are designed using the main part of the Code.

ACI Code 21.12.3 offers two options for the shear design of frame members. The first option is similar to that illustrated in Figs. 20.17 and 20.19 and Eqs. (20.28) and (20.29), with the exception that the probable strengths M_{pr} are replaced by the nominal strengths M_n. For beams, f_y is substituted for $1.25f_y$ in Eq. (20.27). For columns, the moments used at the top and bottom of the column [Fig. 20.15 and Eq. (20.29)] are based on the capacity of the column alone (not considering the moment capacity of the beams framing into the joints) and are based on the factored axial load P_u that results in the maximum nominal moment capacity.

As an alternative to designing for shear induced by the formation of hinges at the ends of the members, ACI Code 21.12.3 allows shear design to be based on load combinations that include an earthquake effect that is twice that required by the governing building code. Thus, Eq. (20.4) becomes

$$U = 1.2D + 2.0E + 1.0L + 0.2S \tag{20.33}$$

For beams and columns, the Code prescribes detailing requirements that are not as stringent as those used in regions of high seismic risk but that provide greater confinement and increased ductility compared to those used in structures not designed for earthquake loading. For beams, the positive-moment strength at the face of a joint must be at least one-third of the negative-moment strength at the joint, in accordance with ACI Code 21.12.4. Both the positive and negative moment strength along the full length of a beam must be at least one-fifth of the maximum moment strength at the face of either joint. Hoops are required at both ends of beams over a length equal to twice the member depth; the first hoop must be placed within 2 in. of the face of the support, and the maximum spacing in this region may not exceed one-fourth of the effective depth, 8 times the diameter of the smallest longitudinal bar, 24 times the stirrup diameter, or 12 in. The maximum stirrup spacing elsewhere in beams is one-half of the effective depth.

For columns, within length l_o from the joint face, the tie spacing s_o may not exceed 8 times the diameter of the smallest longitudinal bar, 24 times the diameter of the tie bar, one-half of the smallest cross-sectional dimension of the column, or 12 in., in accordance with ACI Code 21.8.5. The length l_o must be greater than one-sixth of the column clear span, the maximum cross-sectional dimension of the member, or 18 in. The first tie must be located not more than $s_o \cdot 2$ from the joint face, and the tie spacing may not exceed twice the spacing s_o anywhere in the member. In accordance with ACI Code 21.12.5 and 11.11.2, lateral joint reinforcement with an area as specified in Eq. (4.13) must be provided within the column for a depth not less than the depth of the deepest flexural member framing into the joint.

For two-way slabs without beams, ACI Code 21.12.6 requires design for earthquake effects using Eqs. (20.4) and (20.5). Under these loading conditions, the reinforcement provided to resist the unbalanced moment transferred between the slab and the column M_s (M_u in Section 13.11) must be placed within the column strip. Reinforcement to resist the fraction of the unbalanced moment M_s defined by Eq. (13.16*a*), $M_{ub} = \cdot_f M_u = \cdot_f M_s$, but not less than one-half of the reinforcement in the column strip at the support, must be concentrated near the column. This reinforcement is placed within an effective slab width located between lines 1.5h on either side of the column or column capital, where h is the total thickness of the slab or drop panel.

To ensure ductile behavior throughout two-way slabs without beams, at least one-quarter of the top reinforcement at the support in column strips must be continuous throughout the span, as must bottom reinforcement equal to at least one-third of the top reinforcement at the support in column strips. A minimum of one-half of all bottom reinforcement at midspan in both column and middle strips must be continuous and develop its yield strength at the face of the support. For discontinuous edges of the slab, both the top and bottom reinforcement must be developed at the face of the support. Finally, at critical sections for two-way shear at columns (Section 13.10a), V_u may not exceed $0.4 \cdot V_c$. This provision may be waived if the earthquake-induced factored shear stress transferred by eccentricity of shear at the point of maximum shear stress does not exceed one-half of the design shear stress $\cdot v_n$ (see Section 13.11).

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Problems

20.1. An interior column joint in a reinforced concrete frame located in a region of high seismic risk consists of 28 in. wide by 20 in. deep beams and 36 in. wide by 20 in. deep girders framing into a 28 × 28 in. column. The slab thickness is 5 in., and the effective overhanging flange width on either side of the web of the flexural members is 40 in. Girder reinforcement at the joint consists of five No. 10 (No. 32) top bars and five No. 8 (No. 25) bottom bars. Beam reinforcement consists of four No. 10 (No. 32) top bars and four No. 8 (No. 25) bottom bars. As the flexural steel crosses the joint, the top and bottom girder bars rest on the respective top and bottom beam bars. Column reinforcement consists of 12 No. 9 (No. 29) bars evenly spaced around the perimeter of the column, similar to the placement shown in Fig. 20.12. Clear cover on the outermost main flexural and column longitudinal reinforcement is 2 in. Assume No. 4 (No. 13) stirrups and ties. For earthquake loading, the maximum factored axial load on the upper column framing into the joint is 1098 kips and the maximum factored axial load on the lower column is 1160 kips. For a frame story height of 13 ft, determine if the nominal flexural strengths of the columns exceed those of the beams and girders by at least 20 percent, and determine the minimum transverse reinforcement required in the columns adjacent to the beams. Use $f'_c = 4000$ psi and $f_y = 60{,}000$ psi.

20.2. Design the joint and the transverse column reinforcement for the members described in Problem 20.1. The factored shears due to earthquake load are

29 kips in the upper column and 31 kips in the lower column. Minimum factored axial loads are 21 and 25 kips below the forces specified in Problem 20.1 for the upper and lower columns, respectively.

20.3. In Example 20.1, the columns are spaced 28 ft on center in the direction of the spandrel beams. The total dead load on the spandrel beam (including self-weight) is 2 kips/ft and the total live load is 0.93 kips/ft. Design the spandrel beam transverse reinforcement for a building subject to high seismic risk.

20.4. Repeat Problem 20.3 for a frame subject to moderate/intermediate seismic risk.