CODE

Flat plates and flat slabs: $0.25I_g$

(c) Area: $1.0A_g$

Alternatively, the moments of inertia of compression and flexural members I shall be permitted to be computed as follows:

Compression members:

$$I = \left(0.80 + 25\frac{A_{st}}{A_g}\right)\left(1 - \frac{M_u}{P_u h} - 0.5\frac{P_u}{P_o}\right)I_g \leq 0.875I_g \quad (10\text{-}11)$$

where P_u and M_u shall be from the particular load combination under consideration, or the combination of P_u and M_u resulting in the smallest value of I. I need not be taken less than $0.35I_g$.

Flexural members:

$$I = (0.10 + 25\rho)\left(1.2 - 0.2\frac{b_w}{d}\right)I_g \leq 0.5I_g \quad (10\text{-}12)$$

For continuous flexural members, I shall be permitted to be taken as the average of values obtained from Eq. (10-12) for the critical positive and negative moment sections. I need not be taken less than $0.25I_g$.

The cross-sectional dimensions and reinforcement ratio used in the above formulas shall be within 10 percent of the dimensions and reinforcement ratio shown on the contract documents or the stiffness evaluation shall be repeated.

10.10.4.2 When sustained lateral loads are present, I for compression members shall be divided by $(1 + \beta_{ds})$. The term β_{ds} shall be taken as the ratio of maximum factored sustained shear within a story to the maximum factored shear in that story associated with the same load combination, but shall not be taken greater than 1.0.

10.10.5 *Moment magnification procedure*

Columns and stories in structures shall be designated as nonsway or sway columns or stories. The design of columns

COMMENTARY

These two effects result in an overestimation of the second-order deflections on the order of 20 to 25 percent, corresponding to an implicit stiffness reduction of 0.80 to 0.85 on the stability calculation.

The moment of inertia of T-beams should be based on the effective flange width defined in 8.10. It is generally sufficiently accurate to take I_g of a T-beam as two times the I_g for the web, $2(b_w h^3/12)$.

If the factored moments and shears from an analysis based on the moment of inertia of a wall, taken equal to $0.70I_g$, indicate that the wall will crack in flexure, based on the modulus of rupture, the analysis should be repeated with $I = 0.35I_g$ in those stories where cracking is predicted using factored loads.

The values of the moments of inertia were derived for nonprestressed members. For prestressed members, the moments of inertia may differ depending on the amount, location, and type of the reinforcement and the degree of cracking prior to ultimate. The stiffness values for prestressed concrete members should include an allowance for the variability of the stiffnesses.

Section 10.10 provides requirements for strength and assumes frame analyses will be carried out using factored loads. Analyses of deflections, vibrations, and building periods are needed at various service (unfactored) load levels (Khuntia and Ghosh 2004a,b) to determine the serviceability of the structure and to estimate the wind forces in wind tunnel laboratories. The moments of inertia of the structural members in the service load analyses should be representative of the degree of cracking at the various service load levels investigated. Unless a more accurate estimate of the degree of cracking at design service load level is available, it is satisfactory to use 1.0/0.70 = 1.43 times the moments of inertia given here for service load analyses.

Equations (10-11) and (10-12) provide more refined values of EI considering axial load, eccentricity, reinforcement ratio, and concrete compressive strength as presented in Mirza et al. (1987) and Mirza (1990). The stiffnesses provided in these references are applicable for all levels of loading, including service and ultimate, and consider a stiffness reduction factor ϕ_K comparable to that included in 10.10.4.1(b). For use at load levels other than ultimate, P_u and M_u should be replaced with their appropriate values at the desired load level.

R10.10.4.2 The unusual case of sustained lateral loads might exist, for example, if there were permanent lateral loads resulting from unequal earth pressures on two sides of an environmental structure.

R10.10.5 *Moment magnification procedure*

This section describes an approximate design procedure that uses the moment magnifier concept to account for slenderness effects. Moments computed using an ordinary first-

CODE

in nonsway frames or stories shall be based on 10.10.6. The design of columns in sway frames or stories shall be based on 10.10.7.

10.10.5.1 It shall be permitted to assume a column in a structure is nonsway if the increase in column end moments due to second-order effects does not exceed 5 percent of the first-order end moments.

10.10.5.2 It also shall be permitted to assume a story within a structure is nonsway if

$$Q = \frac{\Sigma P_u \Delta_o}{V_{us} \ell_c} \leq 0.05 \quad (10\text{-}13)$$

where $\sum \boldsymbol{P_u}$ and $\boldsymbol{V_{us}}$ are the total factored vertical load and the horizontal story shear, respectively, in the story being evaluated, and Δ_o is the first-order relative lateral deflection between the top and the bottom of that story due to $\boldsymbol{V_{us}}$.

10.10.6 *Moment magnification procedure—nonsway*

Compression members shall be designed for factored axial force $\boldsymbol{P_u}$ and the factored moment amplified for the effects of member curvature $\boldsymbol{M_c}$ as follows

$$M_c = \delta M_2 \quad (10\text{-}14)$$

where

$$\delta = \frac{C_m}{1 - \dfrac{P_u}{0.75 P_c}} \geq 1.0 \quad (10\text{-}15)$$

and

$$P_c = \frac{\pi^2 EI}{(k \ell_u)^2} \quad (10\text{-}16)$$

COMMENTARY

order frame analysis are multiplied by a moment magnifier that is a function of the factored axial force $\boldsymbol{P_u}$ and the critical buckling load $\boldsymbol{P_c}$ for the column. Nonsway and sway frames are treated separately. A first-order frame analysis is an elastic analysis that does not include the internal force effects resulting from deflections.

The moment magnifier design method requires the designer to distinguish between nonsway frames, which are designed according to 10.10.6, and sway frames, which are designed according to 10.10.7. Frequently, this can be done by inspection by comparing the total lateral stiffness of the columns in a story to that of the bracing elements. A compression member may be assumed nonsway by inspection if it is located in a story in which the bracing elements (shearwalls, shear trusses, or other types of lateral bracing) have such substantial lateral stiffness to resist the lateral deflections of the story that any resulting lateral deflection is not large enough to affect the column strength substantially. If not readily apparent by inspection, 10.10.5.1 and 10.10.5.2 give two possible ways of doing this. In 10.10.5.1, a story in a frame is said to be nonsway if the increase in the lateral load moments resulting from $\boldsymbol{P}$-Δ effects does not exceed 5 percent of the first-order moments (Grossman 1990). Section 10.10.5.2 gives an alternative method of determining this based on the stability index for a story $\boldsymbol{Q}$. In computing $\boldsymbol{Q}$, $\sum \boldsymbol{P_u}$ should correspond to the lateral loading case for which $\sum \boldsymbol{P_u}$ is greatest. A frame may contain both nonsway and sway stories. This test would not be suitable if $\boldsymbol{V_{us}}$ is zero.

If the lateral load deflections of the frame have been computed using service loads and the service load moments of inertia given in 10.10.4, it is permissible to compute $\boldsymbol{Q}$ in Eq. (10-13) using 1.2 times the sum of the service gravity loads, the service load story shear, and 1.43 times the first-order service load story deflections.

R10.10.6 *Moment magnification procedure—nonsway*

The ϕ used in the design of slender columns represent two different sources of variability. First, the stiffness reduction ϕ_K accounts for the variability in the stiffness $\boldsymbol{EI}$ and the moment magnification analysis. Second, the strength reduction ϕ for tied and spiral columns accounts for the variability of the strength of the cross section. Studies reported in Lai and MacGregor (1983) indicate that the stiffness reduction factor ϕ_K and the cross-sectional strength reduction ϕ do not have the same values. These studies suggest the stiffness reduction factor ϕ_K for an isolated column should be 0.75 for both tied and spiral columns. The 0.75 factor in Eq. (10-15) is the stiffness reduction factor ϕ_K. The factor is based on the probability of understrength of a single isolated slender column. In the case of a multistory frame, the column and frame deflections depend on the average concrete strength which is higher than the strength of the concrete in the critical single understrength column. For this reason, the value of ϕ_K in 10.10.4 is 0.875.

CODE

10.10.6.1 ***EI*** shall be taken as

$$EI = \frac{(0.2E_cI_g + E_sI_{se})}{1+\beta_{dns}} \quad (10\text{-}17)$$

or

$$EI = \frac{0.4E_cI_g}{1+\beta_{dns}} \quad (10\text{-}18)$$

Alternatively, ***EI*** shall be permitted to be computed using the value of ***I*** from Eq. (10-11) divided by $(1 + \beta_{dns})$.

10.10.6.2 The term β_{dns} shall be taken as the ratio of maximum factored axial sustained load to maximum factored axial load associated with the same load combination but shall not be taken greater than 1.0.

10.10.6.3 The effective length factor ***k*** shall be permitted to be taken as 1.0.

10.10.6.4 For members without transverse loads between supports, C_m shall be taken as

$$C_m = 0.6 + 0.4\frac{M_1}{M_2} \quad (10\text{-}19)$$

where M_1/M_2 is positive if the column is bent in single curvature, and negative if the member is bent in double curvature. For members with transverse loads between supports, C_m shall be taken as 1.0.

10.10.6.5 Factored moment, M_2, in Eq. (10-14) shall not be taken less than

$$M_{2,min} = P_u(0.6 + 0.03h) \quad (10\text{-}20)$$

COMMENTARY

R10.10.6.1 In defining the critical load, the main problem is the choice of a stiffness ***EI*** that reasonably approximates the variations in stiffness due to cracking, creep, and nonlinearity of the concrete stress-strain curve. Either Eq. (10-17) or Eq. (10-18) may be used to compute ***EI***. Equation (10-17) was derived for small eccentricity ratios and high levels of axial load where slenderness effects are most pronounced. Equation (10-18) is a simplified approximation to Eq. (10-17) and is less accurate (Bianchini et al. 1960). For improved accuracy, ***EI*** can be approximated using the suggested ***E*** and ***I*** values provided by Eq. (10-11) divided by $(1 + \beta_{dns})$.

R10.10.6.2 Creep due to sustained load will increase the lateral deflections of a column and, hence, the moment magnification. This is approximated for design by reducing the stiffness ***EI*** used to compute P_c, and hence δ, by dividing ***EI*** by $(1 + \beta_{dns})$. Both the concrete and steel terms in Eq. (10-17) are divided by $(1 + \beta_{dns})$ to reflect the premature yielding of steel in columns subjected to sustained load. For simplification, it can be assumed that $\beta_{dns} = 0.6$. In this case, Eq. (10-18) becomes

$$EI = 0.25E_cI_g$$

R10.10.6.3 The effective length factor for a compression member considering braced behavior ranges from 0.5 and 1.0. While lower values can be justified, it is recommended that a ***k*** value of 1.0 be used. If lower values are used, the calculation of ***k*** should be based on analysis of the frame using E_c and ***I*** values given in 10.10.4. The Jackson and Moreland Alignment Charts (Fig. R10.10.1) can be used to estimate lower values of ***k*** (SP-17(97); MacGregor et al. 1970).

R10.10.6.4 The factor C_m is a correction factor relating the actual moment diagram to an equivalent uniform moment diagram. The derivation of the moment magnifier assumes that the maximum moment is at or near midheight of the column. If the maximum moment occurs at one end of the column, design should be based on an equivalent uniform moment C_mM_2 that would lead to the same maximum moment when magnified (MacGregor 1993).

In the case of compression members that are subjected to transverse loading between supports, it is possible that the maximum moment will occur at a section away from the end of the member. If this occurs, the value of the largest calculated moment occurring anywhere along the member should be used for the value of M_2 in Eq. (10-14). C_m is to be taken as 1.0 for this case.

R10.10.6.5 In the Code, slenderness is accounted for by magnifying the column end moments. If the factored column moments are very small or zero, the design of slender columns should be based on the minimum eccentricity given

CODE

about each axis separately, where 0.6 and h are in inches. For members in which $M_{2,min}$ exceeds M_2, the value of C_m in Eq. (10-19) shall either be taken equal to 1.0, or shall be based on the ratio of the computed end moments, M_1/M_2.

10.10.7 *Moment magnification procedure—sway*

Moments M_1 and M_2 at the ends of an individual compression member shall be taken as

$$M_1 = M_{1ns} + \delta_s M_{1s} \quad (10\text{-}21)$$

$$M_2 = M_{2ns} + \delta_s M_{2s} \quad (10\text{-}22)$$

where δ_s is computed according to 10.10.7.3 or 10.10.7.4.

10.10.7.1 Flexural members shall be designed for the total magnified end moments of the compression members at the joint.

10.10.7.2 The effective length factor k shall be determined using the values of E_c and I given in 10.10.4 and shall not be less than 1.0.

10.10.7.3 The moment magnifier δ_s shall be calculated as

$$\delta_s = \frac{1}{1-Q} \geq 1 \quad (10\text{-}23)$$

If δ_s calculated by Eq. (10-23) exceeds 1.5, δ_s shall be calculated using second-order elastic analysis or 10.10.7.4.

COMMENTARY

in this section. It is not intended that the minimum eccentricity be applied about both axes simultaneously.

The factored column end moments from the structural analysis are used in Eq. (10-19) in determining the ratio M_1/M_2 for the column when the design should be based on minimum eccentricity. This eliminates what would otherwise be a discontinuity between columns with computed eccentricities less than the minimum eccentricity and columns with computed eccentricities equal to or greater than the minimum eccentricity.

R10.10.7 *Moment magnification procedure—sway*

The analysis described in this section deals only with plane frames subjected to loads causing deflections in that plane. If torsional displacements are significant, a three-dimensional second-order analysis should be used.

R10.10.7.1 The strength of a sway frame is governed by the stability of the columns and by the degree of end restraint provided by the beams in the frame. If plastic hinges form in the restraining beam, the structure approaches a failure mechanism and its axial load capacity is drastically reduced. This section provides that the designer makes certain that the restraining flexural members have the capacity to resist the magnified column moments.

R10.10.7.3 The iterative P-Δ analysis for second-order moments can be represented by an infinite series. The solution of this series is given by Eq. (10-23) (Grossman 1990). Ospina and Alexander (1998) shows that Eq. (10-23) closely predicts the second-order moments in a sway frame until δ_s exceeds 1.5.

The P-Δ moment diagrams for deflected columns are curved, with Δ related to the deflected shape of the columns. Equation (10-23) and most commercially available second-order frame analyses have been derived assuming that the P-Δ moments result from equal and opposite forces of $P\Delta/\ell_c$ applied at the bottom and top of the story. These forces give a straight-line P-Δ moment diagram. The curved P-Δ moment diagrams lead to lateral displacements on the order of 15 percent larger than those from the straight-line P-Δ moment diagrams. This effect can be included in Eq. (10-23) by writing the denominator as $(1 - 1.15Q)$ rather than $(1 - Q)$. The 1.15 factor has been left out of Eq. (10-23) for simplicity.

If deflections have been calculated using service loads, Q in Eq. (10-23) should be calculated in the manner explained in R10.10.5.

CODE

COMMENTARY

The Q factor analysis is based on deflections calculated using the values of E_c and I from 10.10.4, which include the equivalent of a stiffness reduction factor ϕ_K. These values of E_c and I lead to a 20 to 25 percent overestimation of the lateral deflections that corresponds to a stiffness reduction factor ϕ_K between 0.80 and 0.85 on the P-Δ moments. As a result, no additional ϕ is needed. Once the moments are established using Eq. (10-23), selection of the cross sections of the columns involves the strength reduction factors ϕ from 9.3.2.2.

10.10.7.4 Alternatively, it shall be permitted to calculate δ_s as

$$\delta_s = \frac{1}{1 - \dfrac{\Sigma P_u}{0.75\Sigma P_c}} \geq 1 \qquad (10\text{-}24)$$

where $\sum P_u$ is the summation for all the factored vertical loads in a story and $\sum P_c$ is the summation for all sway-resisting columns in a story. P_c is calculated using Eq. (10-16) with k determined from 10.10.7.2 and EI from 10.10.6.1, where β_{ds} shall be substituted for β_{dns}.

R10.10.7.4 To check the effects of story stability, δ_s is computed as an averaged value for the entire story based on use of $\sum P_u/\sum P_c$. This reflects the interaction of all sway resisting columns in the story in the P-Δ effects because the lateral deflection of all columns in the story should be equal in the absence of torsional displacements about a vertical axis. In addition, it is possible that a particularly slender individual column in a sway frame could have substantial midheight deflections even if adequately braced against lateral end deflections by other columns in the story. Such a column should be checked using 10.10.6.

If the lateral load deflections involve a significant torsional displacement, the moment magnification in the columns farthest from the center of twist may be underestimated by the moment magnifier procedure. In such cases, a three-dimensional second-order analysis should be considered.

The 0.75 in the denominator of Eq. (10-24) is a stiffness reduction factor ϕ_K as explained in R10.10.6.

In the calculation of EI, β_{ds} will normally be zero for a sway frame because the lateral loads are generally of short duration. Sway deflections due to short-term loads such as wind or earthquake are a function of the short-term stiffness of the columns following a period of sustained gravity load. For this case, the definition of β_{ds} in 10.10.4.2 gives β_{ds} $= 0$. In the unusual case of a sway frame where the lateral loads are sustained, β_{ds} will not be zero. This might occur if a building on a sloping site is subjected to earth pressure on one side but not on the other.

10.11—Axially loaded members supporting slab system

Axially loaded members supporting a slab system included within the scope of 14.1 shall be designed as provided in Chapter 10 and in accordance with the additional requirements of Chapter 14.

10.12—Transmission of column loads through floor system

If f_c' of a column is greater than 1.4 times that of the floor system, transmission of load through the floor system shall be provided by 10.12.1, 10.12.2, or 10.12.3.

R10.12—Transmission of column loads through floor system

The requirements of this section are based on a paper on the effect of floor concrete strength on column strength (Everard and Cohen 1964). The provisions mean that where the column concrete strength does not exceed the floor concrete strength by more than 40 percent, no special precautions need be taken. For higher column concrete strengths, methods in 10.12.1 or 10.12.2 should be used for corner

CODE

COMMENTARY

or edge columns. Methods in 10.12.1, 10.12.2, or 10.12.3 should be used for interior columns with adequate restraint on all four sides.

10.12.1 Concrete of strength specified for the column shall be placed in the floor at the column location. Top surface of the column concrete shall extend 2 ft into the slab from face of column. Column concrete shall be well integrated with floor concrete and shall be placed in accordance with 7.1.3.1 and 7.1.3.2.

R10.12.1 Application of the concrete placement procedure described in 10.12.1 requires the placing of two different concrete mixtures in the floor system. The lower-strength mixture should be placed while the higher-strength concrete is still plastic and should be adequately vibrated to ensure the concretes are well integrated. This requires careful coordination of the concrete deliveries and the possible use of retarders. In some cases, additional inspection services will be required when this procedure is used. It is important that the higher-strength concrete in the floor in the region of the column be placed before the lower-strength concrete in the remainder of the floor to prevent accidental placing of the low-strength concrete in the column area. It is the responsibility of the licensed design professional to indicate on the contract documents where the high- and low-strength concretes are to be placed.

With the ACI 318-83, the amount of column concrete to be placed within the floor is expressed as a simple 2 ft extension from face of column. Because the concrete placement requirement should be carried out in the field, it is now expressed in a way that is directly evident to workers. The new requirement will also locate the interface between column and floor concrete farther out into the floor, away from regions of very high shear.

10.12.2 Strength of a column through a floor system shall be based on the lower value of concrete strength with vertical dowels and spirals as required.

10.12.3 For columns laterally supported on four sides by beams of approximately equal depth or by slabs, it shall be permitted to base strength of the column on an assumed concrete strength in the column joint equal to 75 percent of column concrete strength plus 35 percent of floor concrete strength. In the application of 10.12.3, the ratio of column concrete strength shall not be taken greater than 2.5 for design.

R10.12.3 Research (Hawkins 1968) has shown that heavily loaded slabs do not provide as much confinement as lightly loaded slabs when ratios of column concrete strength to slab concrete strength exceed about 2.5. Consequently, a limit is placed on the concrete strength ratio assumed in design.

10.13—Composite compression members

10.13.1 Composite compression members shall include all such members reinforced longitudinally with structural steel shapes, pipe, or tubing with or without longitudinal bars.

R10.13—Composite compression members

R10.13.1 Composite columns are defined without reference to classifications of combination, composite, or concrete-filled pipe column. Reference to other metals used for reinforcement has been omitted because they are seldom used with concrete in construction.

10.13.2 Strength of a composite member shall be computed for the same limiting conditions applicable to ordinary reinforced concrete members.

R10.13.2 The same rules used for computing the load-moment interaction strength for reinforced concrete sections can be applied to composite sections. Interaction charts for concrete-filled tubing would have a form identical to those of the SP-17(97) chapter on Columns, but with γ slightly greater than 1.0.

CODE

10.13.3 Any axial load strength assigned to concrete of a composite member shall be transferred to the concrete by members or brackets in direct bearing on the composite member concrete.

10.13.4 All axial load strength not assigned to concrete of a composite member shall be developed by direct connection to the structural steel shape, pipe, or tube.

10.13.5 For evaluation of slenderness effects, radius of gyration, r, of a composite section shall be not greater than the value given by

$$r = \sqrt{\frac{(E_c I_g/5) + E_s I_{sx}}{(E_c A_g/5) + E_s A_{sx}}} \tag{10-25}$$

and, as an alternative to a more accurate calculation, EI in Eq. (10-16) shall be taken either as Eq. (10-17) or

$$EI = \frac{(E_c I_g/5)}{1+\beta_{dns}} + E_s I_{sx} \tag{10-26}$$

10.13.6 *Structural steel-encased concrete core*

10.13.6.1 For a composite member with a concrete core encased by structural steel, the thickness of the steel encasement shall be not less than

$$b\sqrt{\frac{f_y}{3E_s}} \text{ for each face of width } b$$

nor

$$h\sqrt{\frac{f_y}{8E_s}} \text{ for circular sections of diameter } h$$

10.13.6.2 Longitudinal bars located within the encased concrete core shall be permitted to be used in computing A_{sx} and I_{sx}.

10.13.7 *Spiral reinforcement around structural steel core*

A composite member with spirally reinforced concrete around a structural steel core shall conform to 10.13.7.1 through 10.13.7.4.

10.13.7.1 Design yield strength of structural steel core shall be the specified minimum yield strength for the grade of structural steel used but not to exceed 50,000 psi.

10.13.7.2 Spiral reinforcement shall conform to 10.9.3.

COMMENTARY

R10.13.3 and R10.13.4 Direct bearing or direct connection for transfer of forces between steel and concrete can be developed through lugs, plates, or reinforcing bars welded to the structural shape or tubing before the concrete is cast. Flexural compressive stress need not be considered a part of direct compression load to be developed by bearing. A concrete encasement around a structural steel shape may stiffen the shape, but it would not necessarily increase its strength.

R.10.13.5 Equation (10-25) is given because the rules of 10.10.1.2 for estimating the radius of gyration are overly conservative for concrete-filled tubing and are not applicable for members with enclosed structural shapes.

In reinforced concrete columns subject to sustained loads, creep transfers some of the load from the concrete to the steel, thus increasing the steel stresses. In the case of lightly reinforced columns, this load transfer may cause the compression steel to yield prematurely, resulting in a loss in the effective EI. Accordingly, both the concrete and steel terms in Eq. (10-17) are reduced to account for creep. For heavily reinforced columns or for composite columns in which the pipe or structural shape makes up a large percentage of the cross section, the load transfer due to creep is not significant. Accordingly, Eq. (10-26) was revised in the 1980 ACI 318 Building Code supplement so that only the EI of the concrete is reduced for sustained load effects.

R10.13.6 *Structural steel-encased concrete core*

Steel-encased concrete sections should have a metal wall thickness large enough to attain longitudinal yield stress before buckling outward.

R10.13.7 *Spiral reinforcement around structural steel core*

Concrete that is laterally contained by a spiral has increased strength, and the size of spiral required can be regulated on the basis of the strength of the concrete outside the spiral by means of the same reasoning that applies for columns reinforced only with longitudinal bars. The radial pressure provided by the spiral ensures interaction between concrete, reinforcing bars, and steel core such that longitudinal bars will both stiffen and strengthen the cross section.

CODE

10.13.7.3 Longitudinal bars located within the spiral shall be not less than 0.01 nor more than 0.08 times net area of concrete section.

10.13.7.4 Longitudinal bars located within the spiral shall be permitted to be used in computing A_{sx} and I_{sx}.

10.13.8 *Tie reinforcement around structural steel core*

A composite member with transversely tied concrete around a structural steel core shall conform to 10.13.8.1 through 10.13.8.7.

10.13.8.1 Design yield strength of structural steel core shall be the specified minimum yield strength for grade of structural steel used but not to exceed 50,000 psi.

10.13.8.2 Transverse ties shall extend completely around the structural steel core.

10.13.8.3 Transverse ties shall have a diameter not less than 0.02 times the greatest side dimension of composite member, except that ties shall not be smaller than No. 3 and are not required to be larger than No. 5. Welded-wire reinforcement of equivalent area shall be permitted.

10.13.8.4 Vertical spacing of transverse ties shall not exceed 16 longitudinal bar diameters, 48 tie bar diameters, or 0.5 times the least side dimension of the composite member.

10.13.8.5 Longitudinal bars located within the ties shall be not less than 0.01 nor more than 0.08 times net area of concrete section.

10.13.8.6 A longitudinal bar shall be located at every corner of a rectangular cross section, with other longitudinal bars spaced not farther apart than one-half the least side dimension of the composite member.

10.13.8.7 Longitudinal bars located within the ties shall be permitted to be used in computing A_{sx} and I_{sx}.

10.14—Bearing strength

10.14.1 Design bearing strength of concrete shall not exceed $\boldsymbol{\phi(0.85f_c'A_1)}$, except when the supporting surface is wider on all sides than the loaded area, then the design bearing strength of the loaded area shall be permitted to be multiplied by $\sqrt{A_2/A_1}$ but by not more than 2.

COMMENTARY

R10.13.8 *Tie reinforcement around structural steel core*

The design yield strength of the steel core should be limited to that which would not generate spalling of the concrete. It has been assumed that axially compressed concrete will not spall at strains less than 0.0018. The yield strength of 0.0018 × 29,000,000, or 52,000 psi, represents an upper limit of the useful maximum steel stress.

Research[10.49] has shown that the required amount of tie reinforcement around the structural steel core is sufficient for the longitudinal steel bars to be included in the flexural stiffness of the composite column.

R10.14—Bearing strength

R10.14.1 This section deals with bearing strength on concrete supports. The permissible bearing stress of $\mathbf{0.85}\boldsymbol{f_c'}$ is based on tests reported in Reference 10.50 (refer also to 16.8).

When the supporting area is wider than the loaded area on all sides, the surrounding concrete confines the bearing area, resulting in an increase in bearing strength. No minimum depth is given for a supporting member. The minimum depth of support will be controlled by the shear requirements of 11.11.

When the top of the support is sloped or stepped, advantage may still be taken of the condition that the supporting member is larger than the loaded area, provided the supporting member does not slope at too great an angle.

CODE

10.14.2 Section 10.14 does not apply to post-tensioning anchorages.

COMMENTARY

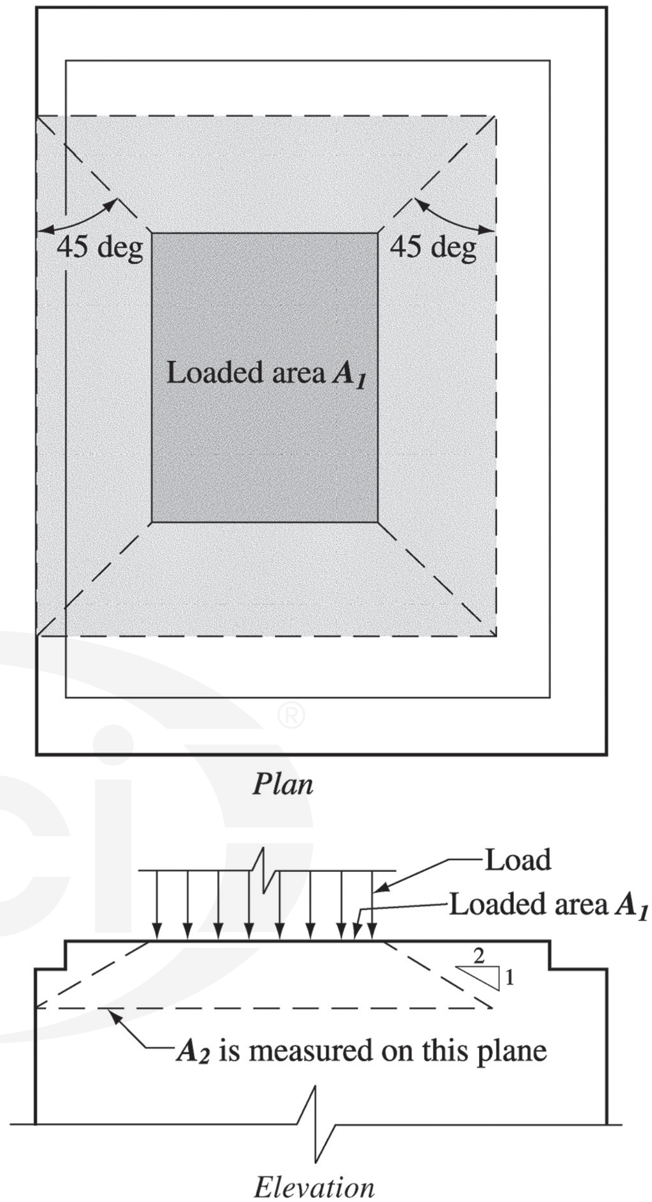

Fig. R10.14.1—Application of frustum to find A_2 *in stepped or sloped supports.*

Figure R10.14.1 illustrates the application of the frustum to find A_2. The frustum should not be confused with the path by which a load spreads out as it travels downward through the support. Such a load path would have steeper sides. The frustum described, however, has somewhat flat side slopes to ensure that there is concrete immediately surrounding the zone of high stress at the bearing. A_1 is the loaded area but not greater than the bearing plate or bearing cross-sectional area.

CODE

CHAPTER 11—SHEAR AND TORSION

11.1—Shear strength

11.1.1 Except for members designed in accordance with Appendix B, design of cross sections subject to shear shall be based on

$$\phi V_n \geq V_u \tag{11-1}$$

where V_u is the factored shear force at the section considered and V_n is nominal shear strength computed by

$$V_n = V_c + V_s/S_d \tag{11-2}$$

where V_c is nominal shear strength provided by concrete calculated in accordance with 11.2, 11.3, or 11.11, and V_s is nominal shear strength provided by shear reinforcement calculated in accordance with 11.4, 11.9.9, or 11.11.

11.1.1.1 In determining V_n, effect of any openings in members shall be considered.

11.1.1.2 In determining V_c, whenever applicable, effects of axial tension due to creep and shrinkage in restrained members shall be considered and effects of inclined flexural compression in variable depth members shall be permitted to be included.

11.1.2 The values of $\sqrt{f_c'}$ used in this chapter shall not exceed 100 psi except as allowed in 11.1.2.1.

11.1.2.1 Values of $\sqrt{f_c'}$ greater than 100 psi shall be permitted in computing V_c, V_{ci}, and V_{cw} for reinforced or prestressed concrete beams and concrete joist construction

COMMENTARY

CHAPTER R11—SHEAR AND TORSION

R11.1—Shear strength

This chapter includes shear and torsion provisions for both nonprestressed and prestressed concrete members. The shear-friction concept (11.6) is particularly applicable to design of reinforcement details in precast structures. Special provisions are included for deep flexural members (11.7), brackets and corbels (11.8), and shear walls (11.9). Shear provisions for slabs and footings are given in 11.11.

The shear strength is based on an average shear stress on the full effective cross section b_wd. In a member without shear reinforcement, shear is assumed to be carried by the concrete web. In a member with shear reinforcement, a portion of the shear is assumed to be provided by the concrete and the remainder by the shear reinforcement.

The shear strength provided by concrete V_c is assumed to be the same for beams with and without shear reinforcement and is taken as the shear causing significant inclined cracking. These assumptions are discussed in Joint ACI-ASCE Committee 426 (1973), MacGregor and Hanson (1969), and Joint ACI-ASCE Committee 326 (1962).

Appendix B allows the use of strut-and-tie models in the shear design of disturbed regions. The traditional shear design procedures, which ignore D-regions, are acceptable in shear spans that include B-regions.

R11.1.1.1 Openings in the web of a member can reduce its shear strength. The effects of openings are discussed in Section 4.7 of Joint ACI-ASCE Committee (1973) as well as in Barney et al. (1977) and Schlaich et al. (1987).

R11.1.1.2 In a member of variable depth, the internal shear at any section is increased or decreased by the vertical component of the inclined flexural stresses. Computation methods are outlined in various textbooks and in the 1940 Joint Committee Report (Joint Committee 1940).

R11.1.2 Because of a lack of test data and practical experience with concretes having compressive strengths greater than 10,000 psi, ACI 318-89 imposed a maximum value of 100 psi on $\sqrt{f_c'}$ for use in the calculation of shear strength of concrete beams, joists, and slabs. Exceptions to this limit were permitted in beams and joists when the transverse reinforcement satisfied an increased value for the minimum amount of web reinforcement. There are limited test data on the two-way shear strength of high-strength concrete slabs. Until more experience is obtained for two-way slabs built with concretes that have strengths greater than 10,000 psi, it is prudent to limit $\sqrt{f_c'}$ to 100 psi for the calculation of shear strength.

R11.1.2.1 Based on the test results in Mphonde and Frantz (1984), Elzanaty et al. (1986), Roller and Russell (1990), and Johnson and Ramirez (1989), an increase in the minimum amount of transverse reinforcement is required for high-