CHAPTER 13—FOUNDATIONS

CODE

13.1—Scope

13.1.1 This chapter shall apply to the design of nonprestressed and prestressed foundations, including shallow foundations (a) through (e), deep foundations (f) through (i), cantilever retaining walls (j) and (k), and basement walls (l):

(a) Strip footings
(b) Isolated footings
(c) Combined footings
(d) Mat foundations
(e) Grade beams
(f) Pile caps
(g) Piles
(h) Drilled piers
(i) Caissons
(j) Cantilever retaining walls
(k) Counterfort and buttressed cantilever retaining walls
(l) Basement walls

COMMENTARY

R13.1—Scope

R13.1.1 Examples of foundation types covered by this chapter are illustrated in Fig. R13.1.1. Stepped and sloped footings are considered to be subsets of other footing types.

The Code contains provisions for the design of deep foundations. These provisions are based in part on similar provisions that were previously included in ASCE/SEI 7 and the IBC.

CODE

COMMENTARY

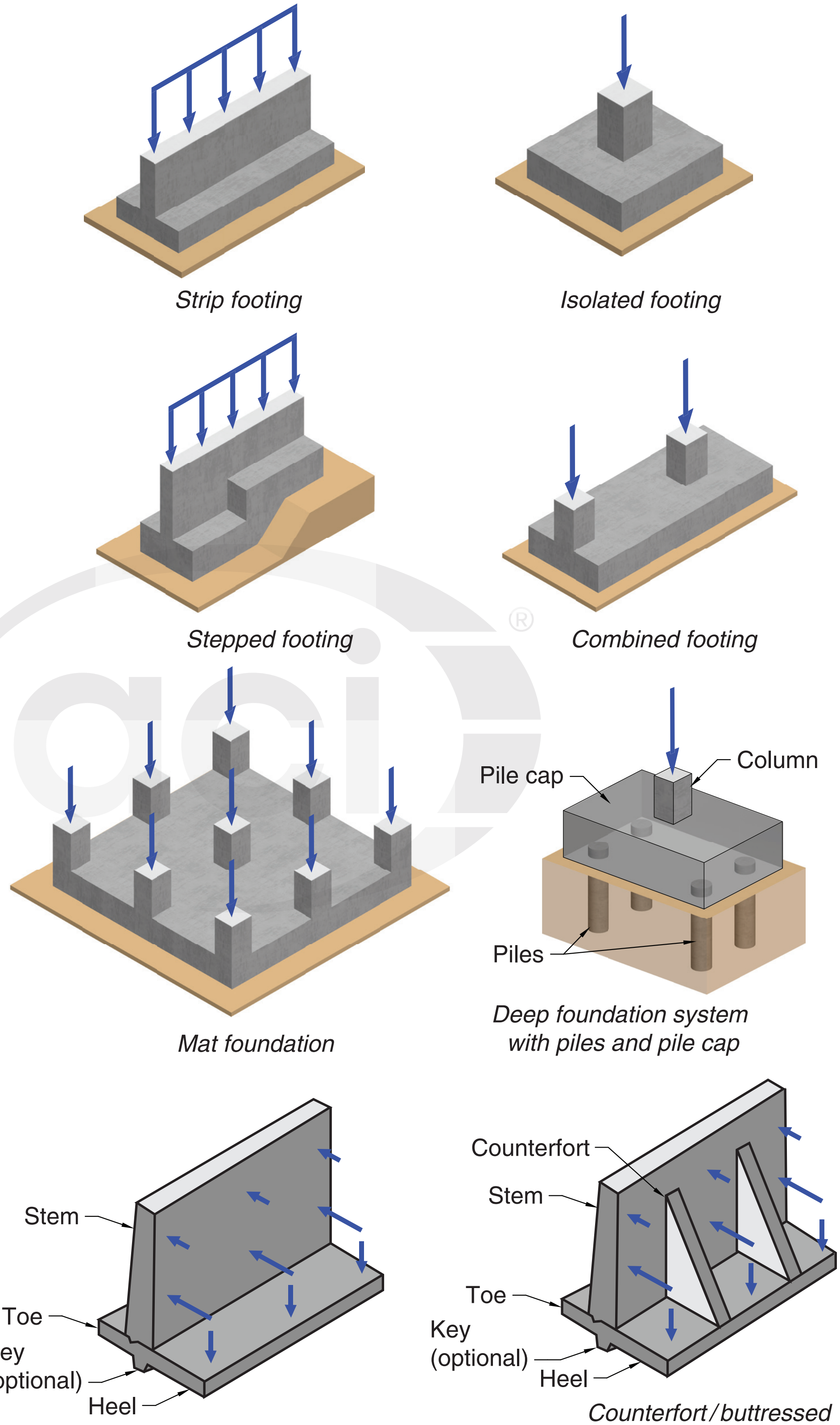

Fig. R13.1.1—Types of foundations.

13.1.2 Foundations excluded by 1.4.7 are excluded from this chapter.

CODE

13.2—General

13.2.1 *Materials*

13.2.1.1 Design properties for concrete shall be selected to be in accordance with Chapter 19.

13.2.1.2 Design properties for steel reinforcement shall be selected to be in accordance with Chapter 20.

13.2.1.3 Materials, design, and detailing requirements for embedments in concrete shall be in accordance with 20.6.

13.2.2 *Connection to other members*

13.2.2.1 Design and detailing of cast-in-place and precast column, pedestal, and wall connections to foundations shall be in accordance with 16.3.

13.2.3 *Earthquake effects*

13.2.3.1 Structural members extending below the base of the structure that are required to transmit forces resulting from earthquake effects to the foundation shall be designed in accordance with 18.2.2.3.

13.2.3.2 For structures assigned to Seismic Design Category (SDC) C, D, E, or F, foundations resisting earthquake-induced forces or transferring earthquake-induced forces between structure and ground shall be designed in accordance with 18.13.

13.2.4 *Slabs-on-ground*

13.2.4.1 Slabs-on-ground that transmit vertical loads or lateral forces from other parts of the structure to the ground shall be designed and detailed in accordance with applicable provisions of this Code.

13.2.4.2 Slabs-on-ground that transmit lateral forces as part of the seismic-force-resisting system shall be designed in accordance with 18.13.

COMMENTARY

R13.2—General

R13.2.3 *Earthquake effects*

R13.2.3.1 The base of a structure, as defined in analysis, does not necessarily correspond to the foundation or ground level, or to the base of a building as defined in the general building code for planning (for example, for height limits or fire protection requirements). Details of columns and walls extending below the base of a structure to the foundation are required to be consistent with those above the base of the structure. For additional discussion of the design of foundations for earthquake effects, see R18.13.1.

R13.2.4 *Slabs-on-ground*

Slabs-on-ground often act as a diaphragm to hold the building together at the ground level and minimize the effects of out-of-phase ground motion that may occur over the footprint of the building. In these cases, the slab-on-ground should be adequately reinforced and detailed. As required in Chapter 26, construction documents should clearly state that these slabs-on-ground are structural members so as to prohibit sawcutting of such slabs.

CODE

13.2.5 *Plain concrete*

13.2.5.1 Plain concrete foundations shall be designed in accordance with Chapter 14.

13.2.6 *Design criteria*

13.2.6.1 Foundations shall be proportioned for bearing effects, stability against overturning and sliding at the soil-foundation interface in accordance with the general building code.

13.2.6.2 For shallow foundation members continuously supported by soil and designed based on the assumption of rigid behavior of the shallow member, (a) and (b) shall be permitted:

(a) For one-way shear strength, V_c shall be taken as:

$$V_c = 2\lambda\sqrt{f_c'}b_w d$$

(b) For two-way shear strength, the size effect factor, λ_s, specified in 22.6, shall be taken equal to 1.0.

13.2.6.3 Foundations members shall be designed to resist factored loads and corresponding induced reactions except as permitted by 13.4.2.

COMMENTARY

R13.2.6 *Design criteria*

R13.2.6.1 Permissible soil pressures or permissible deep foundation strengths are determined by principles of soil mechanics and in accordance with the general building code. The size of the base area of a footing on soil or the number and arrangement of deep foundation members are established by using allowable geotechnical strength and service-level load combinations or by using nominal geotechnical strength with resistance factor and factored load combinations.

Only the calculated end moments at the base of a column or pedestal require transfer to the footing. The minimum moment requirement for slenderness considerations given in 6.6.4.5 need not be considered for transfer of forces and moments to footings.

R13.2.6.2 Both (a) and (b) are permitted because shallow foundation members directly bearing on soil or supported on closely spaced piles, and designed under previous editions of the Code, have exhibited satisfactory behavior, without consideration of the influence of size effect or flexural reinforcement effect on shear strength. This differs from the observed shear behavior of concrete members not directly bearing on soil, which is the type of member investigated in the body of laboratory research that formed the basis of the size effect and flexural reinforcement effect provisions introduced in ACI CODE-318-19.

The reasons for such differences may be due to many factors. Influencing factors may include the overestimation of the design shear forces generated by soil bearing pressures, the development of stiffer and stronger shear transfer mechanisms including arching action, or the development of friction forces or passive pressures that mitigate cracking as a result of the interaction of the shallow foundation member with the soil. Approximate methods traditionally used to estimate soil bearing pressures acting on shallow foundation members, and the assumed uniform distribution of those pressures beneath the foundation, may overestimate design shear forces in common types of shallow foundations.

This provision is based on the assumption of rigid member behavior. Shallow foundation members or portions of these members that are designed considering the stiffness interaction between the soil and the member may not benefit from the conservative stress distributions resulting from the rigid foundation assumption.

R13.2.6.3 To design a footing or pile cap for strength, the induced reactions due to factored loads applied to the foundation should be determined. For a single concentrically-loaded spread footing, the soil pressure due to factored

CODE

COMMENTARY

loading is calculated as the factored load divided by the base area of the footing. For the case of footings or mats with eccentric loading, applied factored loads may be used to determine soil pressures. For pile caps or mats supported by deep foundations, applied factored loads may be used to determine member reactions. However, the resulting pressures or reactions may be incompatible with the geotechnical design resulting in unacceptable subgrade reactions or instability (Rogowsky and Wight 2010). In such cases, the design should be adjusted in coordination with the geotechnical engineer.

Only the calculated end moments at the base of a column or pedestal require transfer to the footing. The minimum moment requirements for slenderness considerations given in 6.6.4.5 need not be considered for transfer of forces and moments to footings.

13.2.6.4 Foundation systems shall be permitted to be designed by any procedure satisfying equilibrium and geometric compatibility.

R13.2.6.4 Foundation design is permitted to be based directly on fundamental principles of structural mechanics, provided it can be demonstrated that all strength and serviceability criteria are satisfied. Design of the foundation may be achieved through the use of classic solutions based on a linearly elastic continuum, numerical solutions based on discrete elements, or yield-line analyses. In all cases, analyses and evaluation of the stress conditions at points of load application or pile reactions in relation to shear and torsion, as well as flexure, should be included.

13.2.6.5 Foundation design in accordance with the strut-and-tie method, Chapter 23, shall be permitted.

R13.2.6.5 An example of the application of this provision is a pile cap similar to that shown in Fig. R13.1.1. Pile caps may be designed using a three-dimensional strut-and-tie model satisfying Chapter 23 (Adebar et al. 1990) provided the shear force limits of 23.4.4 are also satisfied.

Figure R13.2.6.5 illustrates the application of the shear force limits of 23.4.4 and the provisions of 13.2.7.2 for one-way shear design of a spread footing using the strut-and-tie method. Soil pressure within d from the face of the column or wall does not contribute to shear across the critical crack (Uzel et al. 2011), but the soil pressure within d contributes to the bending moment at the face of the column or wall.

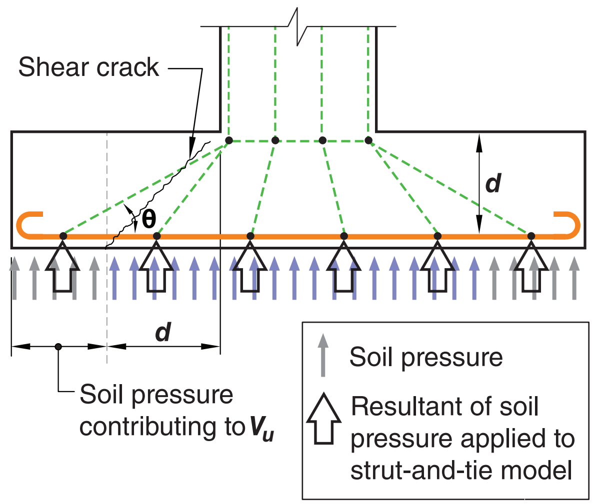

Fig. R13.2.6.5—One-way shear design of a spread footing using the strut-and-tie method.

13.2.6.6 External moment on any section of a strip footing, isolated footing, or pile cap shall be calculated by passing a vertical plane through the member and calculating the moment of the forces acting over the entire area of member on one side of that vertical plane.

13.2.7 *Critical sections for shallow foundations and pile caps*

R13.2.7 *Critical sections for shallow foundations and pile caps*

13.2.7.1 M_u at the supported member shall be permitted to be calculated at the critical section defined in accordance with Table 13.2.7.1.

Table 13.2.7.1—Location of critical section for M_u

Supported member	Location of critical section
Column or pedestal	Face of column or pedestal
Column with steel base plate	Halfway between face of column and edge of steel base plate
Concrete wall	Face of wall
Masonry wall	Halfway between center and face of masonry wall

13.2.7.2 The location of critical section for factored shear in accordance with 7.4.3 and 8.4.3 for one-way shear or 8.4.4.1 for two-way shear shall be measured from the location of the critical section for M_u in 13.2.7.1.

R13.2.7.2 The shear strength of a footing is determined for the more severe condition of 8.5.3.1.1 and 8.5.3.1.2. The critical section for shear is measured from the face of the supported member (column, pedestal, or wall), except for masonry walls and members supported on steel base plates.

Calculation of shear requires that the soil reaction be obtained from factored loads, and the design strength be in accordance with Chapter 22.

Where necessary, shear around individual piles may be investigated in accordance with 8.5.3.1.2. If shear perimeters overlap, the modified critical perimeter b_o should be taken as that portion of the smallest envelope of individual shear perimeters that will actually resist the critical shear for the group under consideration. One such situation is illustrated in Fig. R13.2.7.2.

CODE

COMMENTARY

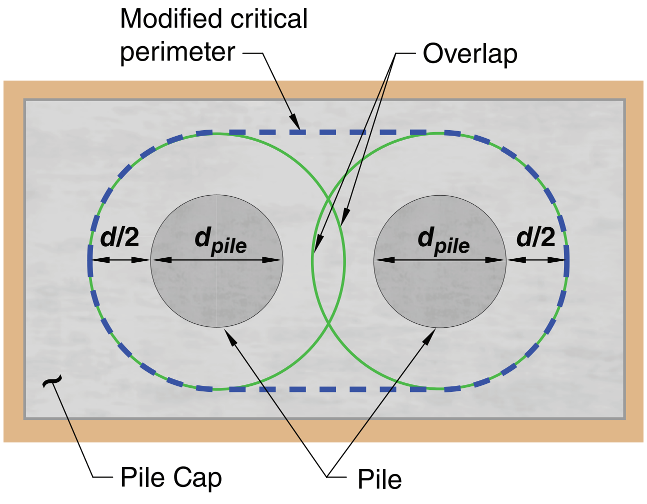

Fig. R13.2.7.2—Modified critical perimeter for shear with overlapping critical perimeters.

13.2.7.3 Circular or regular polygon-shaped concrete columns or pedestals shall be permitted to be treated as square members of equivalent area when locating critical sections for moment, shear, and development of reinforcement.

13.2.8 *Development of reinforcement in shallow foundations and pile caps*

13.2.8.1 Development of reinforcement shall be in accordance with Chapter 25.

13.2.8.2 Calculated tensile or compressive force in reinforcement at each section shall be developed on each side of that section.

13.2.8.3 Critical sections for development of reinforcement shall be assumed at the same locations as given in 13.2.7.1 for maximum factored moment and at all other vertical planes where changes of section or reinforcement occur.

13.2.8.4 Adequate embedment shall be provided for tension reinforcement where reinforcement stress is not directly proportional to moment, such as in sloped, stepped, or tapered foundations; or where tension reinforcement is not parallel to the compression face.

13.2.9 *Concrete cover*

13.2.9.1 Concrete cover for reinforcement in foundation members shall be in accordance with 20.5.1.3.

R13.2.9 *Concrete cover*

R13.2.9.1 Specific cover requirements for deep foundation members are given in 20.5.1.3.4.

13.3—Shallow foundations

13.3.1 *General*

13.3.1.1 Minimum base area of foundation shall be proportioned to not exceed the permissible bearing pressure when

R13.3—Shallow foundations

R13.3.1 *General*

R13.3.1.1 General discussion on the sizing of shallow foundations is provided in R13.2.6.1.

13 Foundations

CODE

subjected to forces and moments applied to the foundation. Permissible bearing pressures shall be determined through principles of soil or rock mechanics in accordance with the general building code, or other requirements as determined by the building official.

13.3.1.2 Overall depth of foundation shall be selected such that the effective depth of bottom reinforcement is at least 6 in.

13.3.1.3 In sloped, stepped, or tapered foundations, depth and location of steps or angle of slope shall be such that design requirements are satisfied at every section.

13.3.2 *One-way shallow foundations*

13.3.2.1 The design and detailing of one-way shallow foundations, including strip footings, combined footings, and grade beams, shall be in accordance with this section and the applicable provisions of Chapter 7 and Chapter 9.

13.3.2.2 Reinforcement shall be distributed uniformly across entire width of one-way footings.

13.3.3 *Two-way isolated footings*

13.3.3.1 The design and detailing of two-way isolated footings shall be in accordance with this section and the applicable provisions of Chapter 7 and Chapter 8.

13.3.3.2 In square two-way footings, reinforcement shall be distributed uniformly across entire width of footing in both directions.

13.3.3.3 In rectangular footings, reinforcement shall be distributed in accordance with (a) and (b):

(a) Reinforcement in the long direction shall be distributed uniformly across entire width of footing.
(b) For reinforcement in the short direction, a portion of the total reinforcement, $\gamma_s A_s$, shall be distributed uniformly over a band width equal to the length of short side of footing, centered on centerline of column or pedestal. Remainder of reinforcement required in the short direction, $(1 - \gamma_s)A_s$, shall be distributed uniformly outside the center band width of footing, where γ_s is calculated by:

$$\gamma_s = \frac{2}{(\beta + 1)} \tag{13.3.3.3}$$

where β is the ratio of long to short side of footing.

COMMENTARY

R13.3.1.3 Anchorage of reinforcement in sloped, stepped, or tapered foundations is addressed in 13.2.8.4.

R13.3.3 *Two-way isolated footings*

R13.3.3.3 To minimize potential construction errors in placing bars, a common practice is to increase the amount of reinforcement in the short direction by $2\beta/(\beta + 1)$ and space it uniformly along the long dimension of the footing (*CRSI Handbook* 1984; Fling 1987).

CODE

13.3.4 *Two-way combined footings and mat foundations*

13.3.4.1 The design and detailing of combined footings and mat foundations shall be in accordance with this section and the applicable provisions of Chapter 8.

13.3.4.2 The direct design method shall not be used to design combined footings and mat foundations.

13.3.4.3 Distribution of bearing pressure under combined footings and mat foundations shall be consistent with properties of the soil or rock and the structure, and with established principles of soil or rock mechanics.

13.3.4.4 Minimum reinforcement in nonprestressed mat foundations shall be in accordance with 8.6.1.1.

13.3.5 *Walls as grade beams*

13.3.5.1 The design of walls as grade beams shall be in accordance with the applicable provisions of Chapter 9.

13.3.5.2 If a grade beam wall is considered a deep beam in accordance with 9.9.1.1, design shall satisfy the requirements of 9.9.

13.3.5.3 Grade beam walls shall satisfy the minimum reinforcement requirements of 11.6.

13.3.6 *Wall components of cantilever retaining walls*

13.3.6.1 The stem of a cantilever retaining wall shall be designed as a one-way slab in accordance with the applicable provisions of Chapter 7.

13.3.6.1.1 It shall be permitted to calculate V_c for cantilever retaining walls as:

$$V_c = 2\lambda\sqrt{f_c'}b_w d$$

13.3.6.2 The stem of a counterfort or buttressed cantilever retaining wall shall be designed as a two-way slab in accordance with the applicable provisions of Chapter 8.

13.3.6.3 For walls of uniform thickness, the critical section for shear and flexure shall be at the interface between the stem and the footing. For walls with a tapered or varied thickness, shear and moment shall be investigated throughout the height of the wall.

COMMENTARY

R13.3.4 *Two-way combined footings and mat foundations*

R13.3.4.1 Detailed recommendations for design of combined footings and mat foundations are reported by ACI PRC-336.2. Also refer to Kramrisch and Rogers (1961).

R13.3.4.2 The direct design method is a method used for the design of two-way slabs. Refer to R6.2.4.1.

R13.3.4.3 Design methods using factored loads and strength reduction factors ϕ can be applied to combined footings or mat foundations, regardless of the bearing pressure distribution.

R13.3.4.4 To improve crack control due to thermal gradients and to intercept potential punching shear cracks with tension reinforcement, the licensed design professional should consider specifying continuous reinforcement in each direction near both faces of mat foundations.

R13.3.6 *Wall components of cantilever retaining walls*

R13.3.6.1.1 This provision is supported by a history of successful performance of cantilever retaining walls designed using the 2014 and earlier editions of the Code (Lew et al. 2010). This provision does not apply to wall footings.

R13.3.6.2 Counterfort or buttressed cantilever retaining walls tend to behave more in two-way action than in one-way action; therefore, additional care should be given to crack control in both directions.

R13.3.6.3 In general, the joint between the wall stem and the footing will be opening under lateral loads; therefore, the critical section should be at the face of the joint. If hooks are required to develop the wall flexural reinforcement, hooks should be located near the bottom of the footing with the free

CODE

13.3.7 *Basement walls*

13.3.7.1 The design of basement walls to resist out-of-plane lateral earth pressure shall satisfy (a) through (e):

(a) Basement walls shall be designed as one-way slabs in accordance with the applicable provisions of Chapter 7 or as two-way slabs in accordance with the applicable provisions of Chapter 8.
(b) Basement walls shall be designed to resist hydrostatic pressure, if applicable.
(c) It shall be permitted to calculate the one-way shear strength of concrete as $V_c = 2\lambda\sqrt{f_c'}b_w d$
(d) For two-way shear strength, the size effect factor λ_s, as specified in 22.6, shall be taken equal to 1.0.
(e) Basement walls shall satisfy the applicable provisions of Chapter 18.

13.3.7.2 For loads other than out-of-plane lateral earth pressure, basement walls shall satisfy the applicable provisions of Chapter 11.

13.4—Deep foundations

13.4.1 *General*

13.4.1.1 Number and arrangement of deep foundation members shall be determined such that forces and moments applied to the foundation do not exceed the permissible deep foundation strength. Permissible deep foundation strength shall be determined through principles of soil or rock mechanics in accordance with the general building code, or other requirements as determined by the building official.

13.4.1.2 Portions of deep foundation members in air, water, or soils not capable of providing adequate lateral restraint to prevent buckling shall be designed as a column or pedestal, in accordance with the applicable provisions of Chapter 10 and Chapter 14, where ϕ shall be determined in accordance with 13.4.3.2.

13.4.1.3 The least side dimension of precast deep foundation members shall not be smaller than 10 in. The diameter of cast-in-place deep foundation members shall not be smaller than 12 in., except 10 in. diameter members are permissible for residential and utility use and occupancy classifications with stud bearing wall construction two stories or less above grade.

13.4.1.4 Design shall consider the effects of potential mislocation of any deep foundation member by at least 3 in. Where the effects of mislocation are being considered, it shall be permitted to increase the axial compressive strength of the deep foundation members by 10%.

COMMENTARY

end of the bars oriented toward the opposite face of the wall (Nilsson and Losberg 1976).

R13.3.7 *Basement walls*

R13.3.7.1 Historically, in the design of basement walls, a two-dimensional analysis is performed and the design lateral earth pressure is not reduced to take advantage of soil arching. This practice has contributed to the successful performance of basement walls designed under previous editions of the Code (Lew et al. 2010). There is a lack of experimental data on the shear strength of basement walls under lateral earth pressure, and there is uncertainty as to the actual load demand on basement walls. If alternative analysis methods are used to reduce the design lateral earth pressure or to obtain a more favorable load distribution, it may be more appropriate to calculate shear strengths according to 22.5.5.

R13.4—Deep foundations

R13.4.1 *General*

R13.4.1.1 General discussion on selecting the number and arrangement of piles, drilled piers, and caissons is provided in R13.2.6.1.

R13.4.1.3 The least side dimension of 10 in. refers to that of a square, or the shortest side of a rectangular cross section, or, for the case of octagonal piles, the diameter of the largest circle that can be inscribed in the octagon.

R13.4.1.4 Due to subsurface obstructions or other reasons, deep foundation members are not always installed within the specified tolerance from their design location. In such cases, the axial load distribution in a group of deep foundation members may cause some of the members to be overloaded.

CODE

13.4.1.5 Design of deep foundation members shall be in accordance with 13.4.2 or 13.4.3.

13.4.2 *Allowable axial strength*

13.4.2.1 It shall be permitted to design a deep foundation member using load combinations for allowable stress design in ASCE/SEI 7, Section 2.4, and the allowable strength specified in Table 13.4.2.1 if (a) and (b) are satisfied:

(a) The deep foundation member is laterally supported for its entire height

(b) The applied forces cause bending moments in the deep foundation member less than the moment due to an accidental eccentricity of 5% of the member diameter or width

Table 13.4.2.1—Maximum allowable compressive strength for deep foundation members

Deep foundation member type	Maximum allowable compressive strength[1]	
Uncased cast-in-place concrete drilled or augered pile	$P_a = 0.3f_c'A_g + 0.4f_yA_s$	(a)
Cast-in-place concrete pile in rock or within a pipe, tube, or other permanent metal casing that does not satisfy 13.4.2.3	$P_a = 0.33f_c'A_g + 0.4f_yA_s$[2]	(b)
Metal cased concrete pile confined in accordance with 13.4.2.3	$P_a = 0.4f_c'A_g$	(c)
Precast nonprestressed concrete pile	$P_a = 0.33f_c'A_g + 0.4f_yA_s$	(d)
Precast prestressed concrete pile	$P_a = (0.33f_c' - 0.27f_{pc})A_g$	(e)

[1]A_g applies to the gross cross-sectional area. If a temporary or permanent casing is used, the inside face of the casing shall be considered the concrete surface.
[2]A_s does not include the steel casing, pipe, or tube.

COMMENTARY

Members installed exceeding this mislocation limit may require further remediation such as installation of additional deep foundation members. When deep foundation members are installed out of position, the resulting eccentricity can produce additional bending moments in the deep foundation member, the structure above, or both. Design should consider how the mislocation of deep foundation members will impact the deep foundation members and any components of the superstructure above.

R13.4.1.5 In addition to the provisions in the Code, recommendations for concrete piles are given in ACI PRC-543, recommendations for drilled piers are given in ACI PRC-336.3, and recommendations for precast prestressed concrete piles are given in “Recommended Practice for Design, Manufacture, and Installation of Prestressed Concrete Piling” (PCI 2019).

R13.4.2 *Allowable axial strength*

R13.4.2.1 Potential changes to lateral support of the deep foundation member due to liquefaction, excavation, scour, or other causes, should be considered.

The values in Table 13.4.2.1 represent an upper bound for well understood soil conditions and quality workmanship. A lower value for the maximum allowable compressive strength may be appropriate, depending on soil conditions and the construction and quality control procedures used. For augered cast-in-place piles, where grout is placed through the stem of a hollow-stem auger as it is withdrawn from the soil, the strength coefficient of 0.3 is based on a strength reduction factor of 0.6. The designer should carefully consider the reliable grout strength, grout strength testing methods, and the minimum cross-sectional area of the member, accounting for soil conditions and construction procedures. Additional information is provided in ACI PRC-543.

13 Foundations

13.4.2.2 If 13.4.2.1(a) or 13.4.2.1(b) is not satisfied, a deep foundation member shall be designed using strength design in accordance with 13.4.3.

13.4.2.3 Metal cased cast-in-place concrete deep foundation members shall be considered to be confined if (a) through (f) are satisfied:

(a) Design shall not use the casing to resist any portion of the axial load imposed.
(b) Casing shall have a sealed tip and shall be mandrel-driven.
(c) Thickness of the casing shall not be less than manufacturer's standard gauge No. 14 (0.068 in.).
(d) Casing shall be seamless, or provided with seams of strength equal to the basic material, and be of a configuration that will provide confinement to the cast-in-place concrete.
(e) Ratio of yield strength of the steel casing to f_c' shall be at least 6, and yield strength shall be at least 30,000 psi.
(f) Nominal diameter of the member shall be less than or equal to 16 in.

R13.4.2.3 The basis for this allowable strength is the added strength provided to the concrete by the confining action of the steel casing. This strength applies only to non-axial load-bearing steel where the stress in the steel is taken in hoop tension instead of axial compression. In this Code, steel pile casing is not to be considered in the design of the pile to resist a portion of the pile axial load. Provisions for members designed to be composite with steel pipe or casing are covered in ANSI/AISC 360.

Potential corrosion of the metal casing should be considered; provision is based on a non-corrosive environment.

13.4.2.4 The use of allowable strengths greater than those specified in Table 13.4.2.1 shall be permitted if accepted by the building official in accordance with 1.10 and justified by load tests.

R13.4.2.4 Geotechnical and load test requirements for deep foundation members can be found in the IBC.

13.4.3 *Strength design*

R13.4.3 *Strength design*

13.4.3.1 Strength design in accordance with this section is permitted for all deep foundation members.

13.4.3.2 The strength design of deep foundation members shall be in accordance with 10.5 using the compressive strength reduction factors of Table 13.4.3.2 for axial load without moment, and the strength reduction factors of Table 21.2.1 for tension, shear, and combined axial force and moment. The provisions of 22.4.2.4 and 22.4.2.5 shall not apply to deep foundations.

R13.4.3.2 The strength design of deep foundation members is discussed in detail in ACI PRC-543.

If cast-in-place concrete drilled or augered piles are subject to flexure, shear, or tension loads, the strength reduction factors should be adjusted accordingly, considering the soil conditions, quality-control procedures that will be implemented, likely workmanship quality, and local experience. Guidance for adjustment factors is provided in ACI PRC-543.

CODE

Table 13.4.3.2—Compressive strength reduction factors ϕ for deep foundation members

Deep foundation member type	Compressive strength reduction factors ϕ	
Uncased cast-in-place concrete drilled or augered pile[1]	0.55	(a)
Cast-in-place concrete pile in rock or within a pipe, tube,[2] or other permanent casing that does not satisfy 13.4.2.3	0.60	(b)
Cast-in-place concrete-filled steel pipe pile[3]	0.70	(c)
Metal cased concrete pile confined in accordance with 13.4.2.3	0.65	(d)
Precast-nonprestressed concrete pile	0.65	(e)
Precast-prestressed concrete pile	0.65	(f)

[1]The factor of 0.55 represents an upper bound for well understood soil conditions with quality workmanship. A lower value for the strength reduction factor may be appropriate, depending on soil conditions and the construction and quality control procedures used.
[2]For wall thickness of the steel pipe or tube less than 0.25 in.
[3]Wall thickness of the steel pipe shall be at least 0.25 in.

COMMENTARY

13.4.4 *Cast-in-place deep foundations*

R13.4.4 *Cast-in-place deep foundations*

13.4.4.1 Cast-in-place deep foundation members subject to uplift or $M_u \geq 0.4M_{cr}$ shall be reinforced, unless enclosed by a structural steel pipe or tube. Where required, reinforcement shall be provided in accordance with 13.4.4.1.1 and 13.4.4.2.

13.4.4.1.1 Minimum reinforcement for deep foundation members supporting structures assigned to SDC A or B shall be in accordance with (a) through (f), except as permitted by 13.4.4.1.2.

(a) Minimum number of longitudinal bars shall be in accordance with 10.7.3.1.
(b) The area of longitudinal reinforcement shall be at least 0.0025 A_g unless both the design axial and flexural strength are at least 4/3 of the required strengths and $M_u \leq 0.4M_{cr}$.
(c) Longitudinal reinforcement shall extend from the top of the deep foundation member to a distance of 10 ft or 3 times the member diameter but shall not exceed the length of the member.
(d) Transverse reinforcement shall be provided over the minimum longitudinal reinforcement length required by (c).
(e) Transverse reinforcement shall be ties or spirals with a minimum 3/8 in. diameter. Anchorage of ties shall be in accordance with 25.7.2.4.1, welded lap splices in accordance with 25.5.7, or Class B tension lap splices in

R13.4.4.1.1 Minimum longitudinal reinforcement is necessary to provide some flexural strength in situations such as: 1) where the lateral strength is decreased near the top of the deep foundation member in disturbed soil; 2) where accidental eccentricity is caused by installation tolerances; and 3) where the deep foundation member may be susceptible to accidental damage caused during excavation to the pile cut-off elevation.

It is common for deep foundation members to be installed with a diameter larger than the specified diameter. The minimum reinforcement requirements given in 13.4.4.1.1 apply to the specified diameter, and do not need to be increased if the installed diameter exceeds the specified diameter. Longitudinal reinforcement equal to $0.0025A_g$ provides a moment capacity (without axial load) that is at least equal to the cracking moment of a circular section considering a modulus of rupture equal to $7.5\sqrt{f_c'}$.

This limit was originally required by ATC 3-06 (1978) for high Seismic Design Category C. It is intended to provide a ductile response after flexural cracking for members with normal strength concrete. Although flexural tests on lightly reinforced deep foundation members are not available, the licensed design professional should be aware that other

13 Foundations

CODE

accordance with 25.5.2. Continuously wound ties shall be anchored with one extra turn of the bar or wire at each end. Anchorage of spirals shall be in accordance with 25.7.3.4.

(f) If longitudinal reinforcement is required for compression and if A_{st} exceeds $0.01A_g$, spacing of transverse reinforcement shall be in accordance with 25.7.2.1.

13.4.4.1.2 For members 24 in. in nominal diameter or smaller, if $P_u < 0.75\phi P_n$ or if the member is designed in accordance with 13.4.2 and the axial load resulting from the allowable stress load combinations is smaller than $0.75P_a$, minimum reinforcement shall be permitted to be in accordance with (a) and (b).

(a) One or more longitudinal bars placed at the center of the member with $A_{st} \geq 0.0025A_g$

(b) Longitudinal reinforcement shall extend from the top of the deep foundation member to a distance of at least 10 ft but shall not exceed the length of the member.

13.4.4.2 Placement of reinforcement in deep foundation members shall be in accordance with 26.5.8.

13.4.5 *Precast concrete piles*

13.4.5.1 Precast concrete piles supporting buildings assigned to SDC A or B shall satisfy the requirements of 13.4.5.2 through 13.4.5.6.

13.4.5.2 Longitudinal reinforcement shall be arranged in a symmetrical pattern.

13.4.5.3 For precast nonprestressed piles, longitudinal reinforcement shall be provided according to (a) and (b):

(a) Minimum of 4 bars

(b) Minimum area of $0.008A_g$

13.4.5.4 For precast prestressed piles, the effective prestress in the pile shall provide a minimum average compressive stress in the concrete in accordance with Table 13.4.5.4.

Table 13.4.5.4—Minimum compressive stress in precast prestressed piles

Pile length, ft	Minimum compressive stress, psi
Pile length ≤ 30	400
30 < Pile length ≤ 50	550
Pile length > 50	700

13.4.5.5 For precast prestressed piles, the effective prestress in the pile shall be calculated based on an assumed total loss of 30,000 psi in the prestressed reinforcement.

COMMENTARY

members with small amounts of flexural reinforcement and subjected to tensile stresses caused by moment or combinations of axial force and moment have been observed to behave in a brittle fashion, tending to fail when the concrete first cracks (Leonhardt 1961; Puranam and Pujol 2017).

Minimum requirements for cast-in-place deep foundation members supporting structures assigned to SDC C, D, E, and F are provided in 18.13.5.

R13.4.4.1.2 For small diameter piles with low axial loads, a single center bar can provide resistance to bending attributable to accidental eccentricity. A full-length center bar with a centering device is recommended as additional quality control to verify installed pile length and diameter.

R13.4.5 *Precast concrete piles*

CODE

13.4.5.6 The longitudinal reinforcement shall be enclosed by transverse reinforcement according to Table 13.4.5.6(a) and shall be spaced according to Table 13.4.5.6(b):

Table 13.4.5.6(a)—Minimum transverse reinforcement size

Least horizontal pile dimension *h*, in.	Minimum wire size transverse reinforcement[1]
$h \leq 16$	W4, D4
$16 < h < 20$	W4.5, D5
$h \geq 20$	W5.5, D6

[1]If bars are used, minimum of No. 3 bar applies to all values of *h*.

Table 13.4.5.6(b)—Maximum transverse reinforcement spacing

Reinforcement location in the pile	Maximum center-to-center spacing, in.
First five ties or spirals at each end of pile	1
24 in. from each end of pile	4
Remainder of pile	6

COMMENTARY

R13.4.5.6 The minimum transverse reinforcement required in this section is typically sufficient for driving and handling stresses. These provisions for precast concrete piles in SDC A and B are based on information from the 2024 IBC. Minimum reinforcement requirements for precast concrete piles supporting buildings assigned to SDC C, D, E, and F are defined in 18.13.5.10.

13.4.6 *Pile caps*

R13.4.6 *Pile caps*

13.4.6.1 Pile caps shall meet the requirements of 13.4.6.2 through 13.4.6.7. Requirements for pile caps apply to all foundation elements to which deep foundation members connect, including grade beams and mats, except as permitted by 13.4.6.3.1.

13.4.6.2 Overall depth of pile cap shall be selected such that the effective depth of bottom reinforcement is at least 12 in.

13.4.6.3 Deep foundation members shall be embedded into a pile cap at least 3 in. The pile cap shall extend beyond the edge of the deep foundation member by at least 4 in.

R13.4.6.3 ACI PRC-543 contains further guidance on pile-to-pile cap connection considerations.

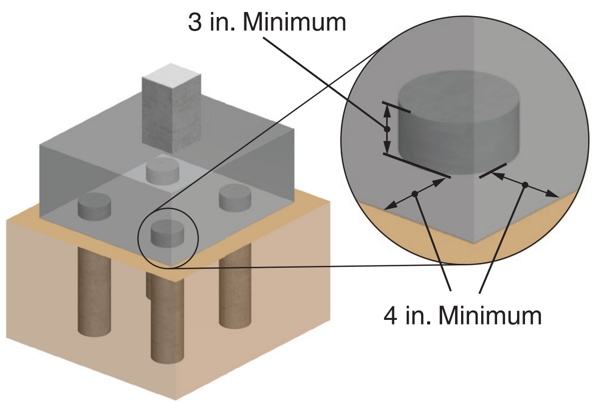

Fig. R13.4.6.3—Deep foundation member embedment in pile cap.

13 Foundations

CODE

13.4.6.3.1 Foundation elements supported on deep foundation members shall be exempt from the requirements of 13.4.6.3, provided that analysis demonstrates that the forces and moments calculated with the factored load combinations of Chapter 5, including design for potential mislocation of deep foundation members in accordance with 13.4.1.4, can be adequately transferred from the foundation elements to the deep foundation members.

13.4.6.4 Factored moments and shears shall be permitted to be calculated with the reaction from any pile assumed to be concentrated at the centroid of the pile section

13.4.6.5 Except for pile caps designed in accordance with 13.2.6.5, the pile cap shall be designed such that (a) is satisfied for one-way foundations and (a) and (b) are satisfied for two-way foundations.

(a) $\phi V_n \geq V_u$, where V_n shall be calculated in accordance with 22.5 for one-way shear, V_u shall be calculated in accordance with 13.4.6.7, and ϕ shall be in accordance with 21.2
(b) $\phi v_n \geq v_u$, where v_n shall be calculated in accordance with 22.6 for two-way shear, v_u shall be calculated in accordance with 13.4.6.7, and ϕ shall be in accordance with 21.2

13.4.6.6 If the pile cap is designed in accordance with the strut-and-tie method as permitted in 13.2.6.5, the effective concrete compressive strength of the struts, f_{ce}, shall be calculated in accordance with 23.4.3.

13.4.6.7 Calculation of factored shear on any section through a pile cap shall be in accordance with (a) through (c):

(a) Entire reaction from any pile with its center located $d_{pile}/2$ or more outside the section shall be considered as producing shear on that section.
(b) Reaction from any pile with its center located $d_{pile}/2$ or more inside the section shall be considered as producing no shear on that section.
(c) For intermediate positions of pile center, the portion of the pile reaction to be considered as producing shear on the section shall be based on a linear interpolation between full value at $d_{pile}/2$ outside the section and zero value at $d_{pile}/2$ inside the section.

13.4.6.8 For pile caps with piles spaced at 4 pile diameters or less, and for mat foundations with piles spaced at 5 pile diameters or less, (a) and (b) shall be permitted:

(a) For one-way shear strength, calculate V_c as:
$V_c = 2\lambda\sqrt{f_c'}b_w d$
(b) For two-way shear strength, calculate v_c as specified in 22.6, with the size effect factor, λ_s, taken equal to 1.0.

COMMENTARY

R13.4.6.6 It is typically necessary to take the effective concrete compressive strength from expression (d) or (f) in Table 23.4.3(a) because it is generally not practical to provide confining reinforcement satisfying 23.5 in a pile cap.

R13.4.6.7 If piles are located inside the critical sections d or $d/2$ from face of column, for one-way or two-way shear, respectively, an upper limit on the shear strength at a section adjacent to the face of the column should be considered. The *CRSI Handbook* (1984) offers guidance for this situation.

R13.4.6.8 Pile caps and mat foundations with closely-spaced piles exhibit increased shear strength compared with those with widely-spaced piles due to arching action.