

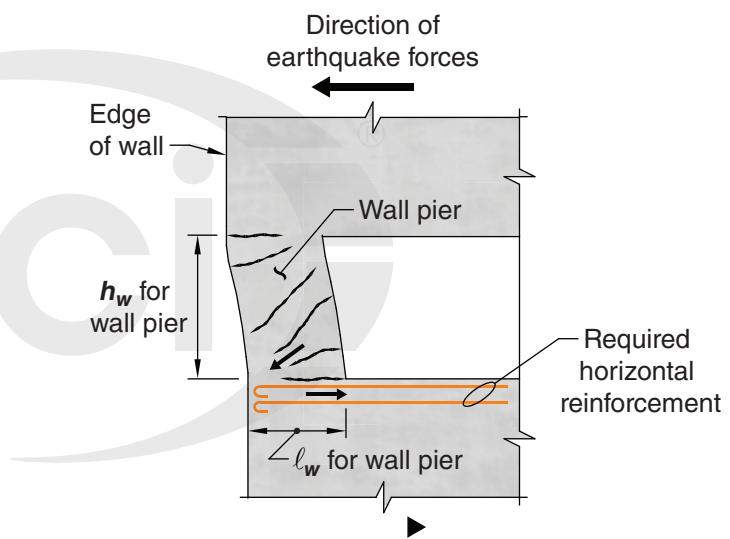
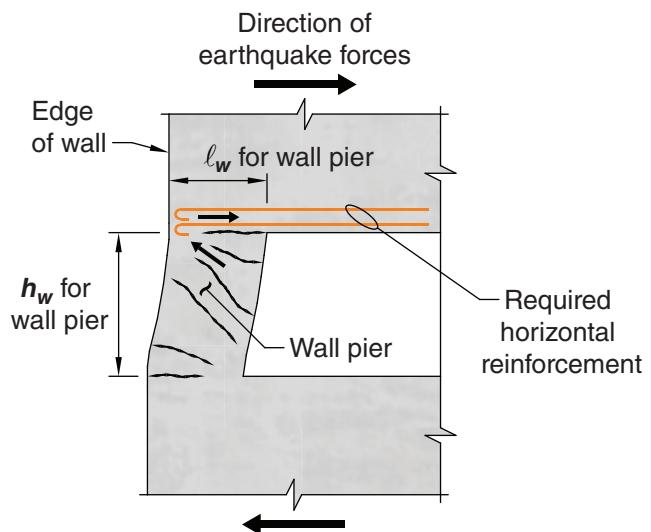
CODE**COMMENTARY**

Fig. R18.10.8—Required horizontal reinforcement in wall segments above and below wall piers at the edge of a wall.

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18.10.8.1 Wall piers shall satisfy the special moment frame requirements for columns of 18.7.4, 18.7.5, and 18.7.6, with joint faces taken as the top and bottom of the clear height of the wall pier. Alternatively, wall piers with $(\ell_w/b_w) > 2.5$ shall satisfy (a) through (f):

- (a) Design shear force shall be calculated in accordance with 18.7.6.1 with joint faces taken as the top and bottom of the clear height of the wall pier. If the general building code includes provisions to account for overstrength of the seismic-force-resisting system, the design shear force need not exceed Ω_o times the factored shear calculated by analysis of the structure for earthquake load effects.
- (b) V_n and distributed shear reinforcement shall satisfy 18.10.4.
- (c) Transverse reinforcement shall be hoops except it shall be permitted to use single-leg horizontal reinforcement parallel to ℓ_w where only one curtain of distributed shear reinforcement is provided. Single-leg horizontal reinforcement shall have 180-degree bends at each end that engage wall pier boundary longitudinal reinforcement.
- (d) Vertical spacing of transverse reinforcement shall not exceed 6 in.
- (e) Transverse reinforcement shall extend at least 12 in. above and below the clear height of the wall pier.
- (f) Special boundary elements shall be provided if required by 18.10.6.3.

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

18.10.9 Ductile coupled walls

R18.10.9 Ductile coupled walls

The aspect ratio limits and development length requirements for ductile coupled walls are intended to induce an energy dissipation mechanism associated with inelastic deformation reversal of coupling beams. Wall stiffness and strength at each end of coupling beams should be sufficient to develop this intended behavior.

18.10.9.1 Ductile coupled walls shall satisfy the requirements of this section.

18.10.9.2 Individual walls shall satisfy $h_{wes}/\ell_w \geq 2$ and the applicable provisions of 18.10 for special structural walls.

18.10.9.3 Coupling beams shall satisfy 18.10.7 and (a) through (c) in the direction considered.

- (a) Coupling beams shall have $\ell_n/h \geq 2$ at all levels of the building.
- (b) All coupling beams at a floor level shall have $\ell_n/h \leq 5$ in at least 90% of the levels of the building.

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(c) The requirements of 18.10.2.5 shall be satisfied at both ends of all coupling beams.

COMMENTARY**18.10.10 Construction joints**

18.10.10.1 Construction joints in structural walls shall be specified according to [26.5.6](#), and contact surfaces shall be roughened consistent with condition (b) of Table 22.9.4.2.

18.10.11 Discontinuous walls

18.10.11.1 Columns supporting discontinuous structural walls shall be reinforced in accordance with 18.7.5.6.

18.11—Special structural walls constructed using precast concrete**18.11.1 Scope**

18.11.1.1 This section shall apply to special structural walls constructed using precast concrete forming part of the seismic-force-resisting system.

18.11.2 General

18.11.2.1 Special structural walls constructed using precast concrete shall satisfy 18.10 and 18.5.2, except 18.10.2.4 shall not apply for precast walls where deformation demands are concentrated at the panel joints.

18.11.2.2 Special structural walls constructed using precast concrete and unbonded post-tensioning tendons and not satisfying the requirements of 18.11.2.1 are permitted provided they satisfy the requirements of [ACI CODE-550.6](#).

R18.11—Special structural walls constructed using precast concrete**R18.11.2 General****18.12—Diaphragms and trusses****18.12.1 Scope**

R18.11.2.2 Experimental and analytical studies ([Priestley et al. 1999](#); [Perez et al. 2003](#); [Restrepo 2002](#)) have demonstrated that some types of precast structural walls post-tensioned with unbonded tendons, and not satisfying the prescriptive requirements of Chapter 18, provide satisfactory seismic performance characteristics. ACI CODE-550.6 defines a protocol for establishing a design procedure, validated by analysis and laboratory tests, for such walls, with or without coupling beams.

[ACI CODE-550.7](#) defines design requirements for one type of special structural wall constructed using precast concrete and unbonded post-tensioning tendons, and validated for use in accordance with 18.11.2.2.

R18.12—Diaphragms and trusses**R18.12.1 Scope**

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

- (a) Support for building elements (such as walls, partitions, and cladding) resisting horizontal forces but not acting as part of the seismic-force-resisting system
- (b) Transfer of lateral forces from the point of application to the vertical elements of the seismic-force-resisting system

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18.12.1.1 This section shall apply to diaphragms and collectors forming part of the seismic-force-resisting system in structures assigned to SDC D, E, or F and to SDC C if 18.12.1.2 applies.

18.12.1.2 Section 18.12.11 shall apply to diaphragms constructed using precast concrete members and forming part of the seismic-force-resisting system for structures assigned to SDC C, D, E, or F.

18.12.1.3 Section 18.12.12 shall apply to structural trusses forming part of the seismic-force-resisting system in structures assigned to SDC D, E, or F.

18.12.2 Design forces**R18.12.2 Design forces**

18.12.2.1 The earthquake design forces for diaphragms shall be obtained from the general building code using the applicable provisions and load combinations.

R18.12.2.1 In the general building code, earthquake design forces for floor and roof diaphragms typically are not calculated directly during the lateral-force analysis that provides story forces and story shears. Instead, diaphragm design forces at each level are calculated by a formula that amplifies the story forces recognizing dynamic effects and includes minimum and maximum limits. These forces are used with the governing load combinations to design diaphragms for shear and moment.

For collector elements, the general building code in the United States specifies load combinations that amplify earthquake forces by a factor Ω_o . The forces amplified by Ω_o are also used for the local diaphragm shear forces resulting from the transfer of collector forces, and for local diaphragm flexural moments resulting from any eccentricity of collector forces. The specific requirements for earthquake design forces for diaphragms and collectors depend on which edition of the general building code is used. The requirements may also vary according to the SDC.

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

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design professional should consider providing increased diaphragm strength.

For reinforced concrete diaphragms, [ASCE/SEI 7](#) Sections 12.10.1 and 12.10.2 provide requirements to determine design forces for reinforced concrete diaphragms. For precast concrete diaphragms in buildings assigned to SDC C, D, E, or F, the provisions of ASCE/SEI 7 Section 12.10.3 apply.

18.12.3 Seismic load path

18.12.3.1 All diaphragms and their connections shall be designed and detailed to provide for transfer of forces to collector elements and to the vertical elements of the seismic-force-resisting system.

18.12.3.2 Elements of a structural diaphragm system that are subjected primarily to axial forces and used to transfer diaphragm shear or flexural forces around openings or other discontinuities shall satisfy the requirements for collectors in 18.12.7.6 and 18.12.7.7.

R18.12.3 Seismic load path

R18.12.3.2 This provision applies to strut-like elements that occur around openings, diaphragm edges, or other discontinuities in diaphragms. Figure R18.12.3.2 shows an example. Such elements can be subjected to earthquake axial forces in combination with bending and shear from earthquake or gravity loads.

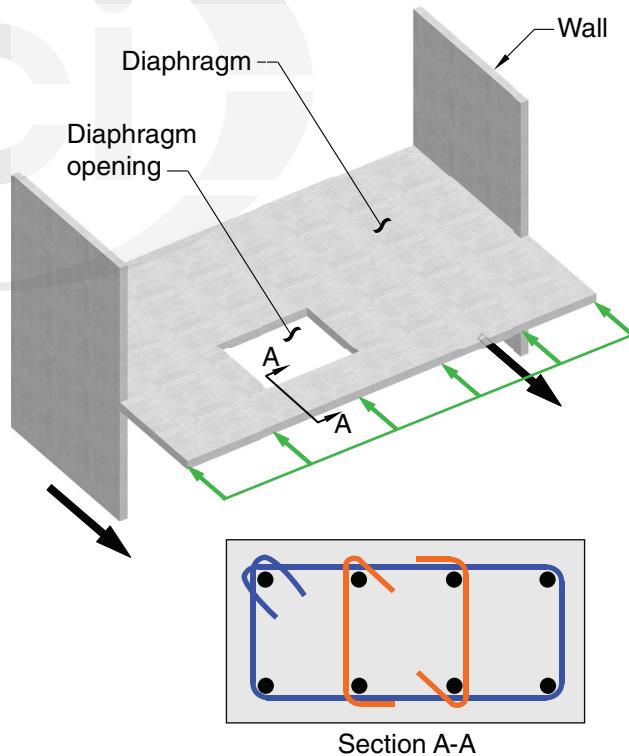


Fig. R18.12.3.2—Example of diaphragm subject to the requirements of 18.12.3.2 and showing an element having confinement as required by 18.12.7.6.

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18.12.4 Cast-in-place composite topping slab diaphragms

18.12.4.1 A cast-in-place composite topping slab on a precast floor or roof shall be permitted as a structural diaphragm, provided the cast-in-place topping slab is reinforced and the surface of the previously hardened concrete on which the topping slab is placed is clean, free of laitance, and intentionally roughened.

18.12.5 Cast-in-place noncomposite topping slab diaphragms

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

18.12.6 Minimum thickness of diaphragms

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

18.12.7 Reinforcement

18.12.7.1 Minimum reinforcement for diaphragms shall be in conformance with 12.6. Except for post-tensioned slabs, reinforcement spacing each way in floor or roof systems shall not exceed 18 in. Where welded wire reinforcement is used as the distributed reinforcement to resist shear in topping slabs placed over precast floor and roof elements, the wires parallel to the joints between the precast elements shall be spaced not less than 10 in. on center. Reinforcement provided for shear strength shall be continuous and shall be distributed uniformly across the shear plane.

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

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R18.12.4 Cast-in-place composite topping slab diaphragms

R18.12.4.1 A bonded topping slab is required so that the floor or roof system can provide restraint against slab buckling. Reinforcement is required to ensure the continuity of the shear transfer across precast joints. The connection requirements are introduced to promote a complete system with necessary shear transfers.

R18.12.5 Cast-in-place noncomposite topping slab diaphragms

R18.12.5.1 Composite action between the topping slab and the precast floor elements is not required, provided that the topping slab is designed to resist the design earthquake forces.

R18.12.6 Minimum thickness of diaphragms

R18.12.6.1 The minimum thickness of concrete diaphragms reflects current practice in joist and waffle systems and composite topping slabs on precast floor and roof systems. Thicker slabs are required if the topping slab is not designed to act compositely with the precast system to resist the design earthquake forces.

R18.12.7 Reinforcement

R18.12.7.1 The maximum spacing for reinforcement is intended to control the width of inclined cracks. Minimum average prestress requirements (refer to 24.4.4.1) are considered to be adequate to limit the crack widths in post-tensioned floor systems; therefore, the maximum spacing requirements do not apply to these systems.

The minimum spacing requirement for welded wire reinforcement in topping slabs on precast floor systems is to avoid fracture of the distributed reinforcement during an earthquake. Cracks in the topping slab open immediately above the boundary between the flanges of adjacent precast members, and the wires crossing those cracks are restrained by the transverse wires (Wood et al. 2000). Therefore, all the deformation associated with cracking should be accommodated in a distance not greater than the spacing of the transverse wires. A minimum spacing of 10 in. for the transverse wires is required to reduce the likelihood of fracture of the wires crossing the critical cracks during a design earthquake. The minimum spacing requirements do not apply to diaphragms reinforced with individual bars, because strains are distributed over a longer length.

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

18.12.7.4 Class G or Class S mechanical splices are required where mechanical splices are used in the region of the connection between the diaphragm and the vertical elements of the seismic-force-resisting system.

18.12.7.5 Longitudinal reinforcement for collectors shall be proportioned such that the average tensile stress over length (a) or (b) does not exceed ϕf_y where the value of f_y is limited to 60,000 psi.

- (a) Length between the end of a collector and location at which transfer of load to a vertical element begins
- (b) Length between two vertical elements

18.12.7.6 Collector elements with compressive stresses exceeding $0.2f'_c$ at any section shall have transverse reinforcement satisfying 18.7.5.2(a) through (e) and 18.7.5.3, except the spacing limit of 18.7.5.3(a) shall be one-third of the least dimension of the collector. The amount of transverse reinforcement shall be in accordance with Table 18.12.7.6. The specified transverse reinforcement is permitted to be discontinued at a section where the calculated compressive stress is less than $0.15f'_c$.

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

Table 18.12.7.6—Transverse reinforcement for collector elements

Transverse reinforcement	Applicable expressions	
A_{sh}/sb_c for rectilinear hoop	$0.9 \frac{f'_c}{f_{yt}}$	(a)
ρ_s for spiral or circular hoop	$0.45 \left(\frac{A_g}{A_{ch}} - 1 \right) \frac{f'_c}{f_{yt}}$	(b)
	$0.12 \frac{f'_c}{f_{yt}}$	(c)

18.12.7.7 Longitudinal reinforcement detailing for collector elements at splices and anchorage zones shall satisfy (a) or (b):

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R18.12.7.3 Bar development and lap splices are designed according to requirements of [Chapter 25](#) for reinforcement in tension. Reductions in development or splice length for calculated stresses less than f_y are not permitted, as indicated in [25.4.10.2](#).

R18.12.7.4 Although the Ω_o factor is to be applied to the design forces transferred by the connection between the diaphragm and the vertical elements of the seismic-force-resisting system, significant yielding of reinforcement at the connection may occur due to inelastic displacements, including but not limited to displacements caused by rocking structural walls and displacements arising from flexural elongation of wall boundaries occurring over several stories. The application of the Ω_o factor does not limit this kind of yielding, and a Class S splice should be considered.

R18.12.7.5 Table 20.2.2.4(a) permits the maximum design yield strength to be 80,000 psi for portions of a collector, for example, at and near critical sections. The average stress in the collector is limited to control diaphragm cracking over the length of the collector. The calculation of average stress along the length is not necessary if the collector is designed for f_y of 60,000 psi even if Grade 80 reinforcement is specified.

R18.12.7.6 In documents such as the NEHRP Provisions ([FEMA P-750](#)), [ASCE/SEI 7](#), the [2024 IBC](#), and the Uniform Building Code ([ICBO 1997](#)), collector elements of diaphragms are designed for forces amplified by a factor Ω_o to account for the overstrength in the vertical elements of the seismic-force-resisting systems. The amplification factor Ω_o ranges between 2 and 3 for most concrete structures, depending on the document selected and on the type of seismic-force-resisting system. In some documents, the factor can be calculated based on the maximum forces that can be developed by the elements of the vertical seismic-force-resisting system.

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

R18.12.7.7 This section is intended to reduce the possibility of bar buckling and provide adequate bar development conditions in the vicinity of splices and anchorage zones.

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- (a) Center-to-center spacing of at least three longitudinal bar diameters, but not less than 1-1/2 in., and concrete clear cover of at least two and one-half longitudinal bar diameters, but not less than 2 in.
- (b) Area of transverse reinforcement, providing A_v , at least the greater of $0.75\sqrt{f'_c}(b_w s/f_{yt})$ and $50b_w s/f_{yt}$, except as required in 18.12.7.6

18.12.8 Combined moment and axial strength

18.12.8.1 Diaphragms and portions of diaphragms shall be designed for flexure in accordance with [Chapter 12](#). The effects of openings shall be considered.

COMMENTARY**R18.12.8 Combined moment and axial strength**

R18.12.8.1 Flexural strength of diaphragms is calculated using the same assumptions as for walls, columns, or beams. The design of diaphragms for moment and other actions uses the applicable load combinations of [5.3.1](#) to consider earthquake forces acting concurrently with gravity or other loads.

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

In the 1999 through 2005 editions of the Code, it was assumed that design moments for diaphragms were resisted entirely by chord forces acting at opposite edges of the diaphragm consistent with a beam idealization of the diaphragm. While this approach is still used, it is also acceptable to resist design moments using distributed reinforcement or precompression with proper consideration of applicable load combinations and analysis including compatibility of stresses and strains.

18.12.9 Shear strength**R18.12.9 Shear strength**

The shear strength requirements for diaphragms are similar to those for slender structural walls and are based on the shear provisions for beams. The term A_{cv} refers to the gross area of the diaphragm, but may not exceed the thickness times the width of the diaphragm. This corresponds to the gross area of the effective deep beam that forms the diaphragm. Distributed slab reinforcement ρ_t used to calculate shear strength of a diaphragm in Eq. (18.12.9.1) is positioned perpendicular to the diaphragm flexural reinforcement. Provision 18.12.9.2 limits the maximum shear strength of the diaphragm.

In addition to satisfying 18.12.9.1 and 18.12.9.2, cast-in-place topping slab diaphragms must also satisfy 18.12.9.3 and 18.12.9.4. Cast-in-place topping slabs on a precast floor or roof system tend to have shrinkage cracks that are aligned with the joints between adjacent precast members. Therefore, the additional shear strength requirements for topping slab diaphragms in 18.12.9.3 are based on a shear friction model ([Wood et al. 2000](#)), and the assumed crack plane corresponds to joints in the precast system along the direction of the applied shear, as shown in Fig. R22.9.4.3a. The coefficient of friction, μ , in the shear friction model is taken

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equal to 1.0 for normalweight concrete due to the presence of these shrinkage cracks.

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

Provision 18.12.9.4 limits the maximum shear that may be transmitted by shear friction within a topping slab diaphragm.

18.12.9.1 V_n of diaphragms shall not exceed:

$$V_n = A_{cv}(2\lambda\sqrt{f'_c} + \rho_f f_y) \quad (18.12.9.1)$$

For cast-in-place topping slab diaphragms on precast floor or roof members, A_{cv} shall be calculated using only the thickness of topping slab for noncomposite topping slab diaphragms and the combined thickness of cast-in-place and precast elements for composite topping slab diaphragms. For composite topping slab diaphragms, the value of f'_c used to calculate V_n shall not exceed the lesser of f'_c for the precast members and f'_c for the topping slab.

18.12.9.2 V_n of diaphragms shall not exceed $8\sqrt{f'_c} A_{cv}$.

18.12.9.3 Above joints between precast elements in noncomposite and composite cast-in-place topping slab diaphragms, V_n shall not exceed:

$$V_n = A_{vf} f_y \mu \quad (18.12.9.3)$$

where A_{vf} is the total area of shear friction reinforcement within the topping slab, including both distributed and boundary reinforcement, that is oriented perpendicular to joints in the precast system and coefficient of friction, μ , is 1.0 λ , where λ is given in 19.2.4. At least one-half of A_{vf} shall be uniformly distributed along the length of the potential shear plane. The area of distributed reinforcement in the topping slab shall satisfy 24.4.3.2 in each direction.

18.12.9.4 Above joints between precast elements in noncomposite and composite cast-in-place topping slab diaphragms, V_n shall not exceed the limits in 22.9.4.4, where A_c is calculated using only the thickness of the topping slab.



CODE**COMMENTARY****18.12.10 Construction joints**

18.12.10.1 Construction joints in diaphragms shall be specified according to 26.5.6, and contact surfaces shall be roughened consistent with condition (b) of Table 22.9.4.2.

18.12.11 Precast concrete diaphragms

18.12.11.1 Diaphragms and collectors constructed using precast concrete members with composite topping slab and not satisfying 18.12.4, and untopped precast concrete diaphragms, are permitted provided they satisfy the requirements of ACI CODE-550.5. Cast-in-place noncomposite topping slab diaphragms shall satisfy 18.12.5 and 18.12.6.

18.12.11.2 Connections and reinforcement at joints used in the construction of precast concrete diaphragms satisfying 18.12.11.1 shall have been tested in accordance with ACI CODE-550.4.

18.12.11.3 Extrapolation of data on connections and reinforcement at joints to project details that result in larger construction tolerances than those used to qualify connections in accordance with ACI CODE-550.4 shall not be permitted.

18.12.12 Structural trusses

18.12.12.1 Structural truss elements with compressive stresses exceeding $0.2f'_c$ at any section shall have transverse reinforcement, in accordance with 18.7.5.2, 18.7.5.3, 18.7.5.7, and Table 18.12.12.1, over the length of the element.

R18.12.11 Precast concrete diaphragms

R18.12.11.1 ACI CODE-550.5 provides requirements for the design of precast concrete diaphragms with connections whose performance has been validated by ACI CODE-550.4 testing. ACI CODE-550.5 permits a maximum tolerance for positioning and completion of connections of 1/2 in., which can be difficult to achieve with normal construction practices. Section 26.13.1.3 requires continuous inspection of precast concrete diaphragm connections to verify that construction is performed properly and tolerances not greater than 1/2 in. for all connections are achieved. Results from ACI CODE-550.4 testing are not to be extrapolated to allow greater tolerances.

Topped precast concrete floors designed in accordance with Chapter 18 need careful consideration of support conditions to verify precast concrete members have sufficient seating for anticipated displacements and ability to accommodate relative rotations between beam supports and the member (Henry et al. 2017).

R18.12.12 Structural trusses

R18.12.12.1 The expressions for transverse reinforcement A_{sh} are based on ensuring compression capacity of an equivalent column section is maintained after spalling of cover concrete.

CODE**COMMENTARY****Table 18.12.12.1—Transverse reinforcement for structural trusses**

Transverse reinforcement	Applicable expressions		
A_{sh}/sb_c for rectilinear hoop	Greater of:	$0.3\left(\frac{A_g}{A_{ch}} - 1\right)\frac{f'_c}{f'_{yt}}$	(a)
		$0.09\frac{f'_c}{f'_{yt}}$	(b)
ρ_s for spiral or circular hoop	Greater of:	$0.45\left(\frac{A_g}{A_{ch}} - 1\right)\frac{f'_c}{f'_{yt}}$	(c)
		$0.12\frac{f'_c}{f'_{yt}}$	(d)

18.12.12.2 All continuous reinforcement in structural truss elements shall be developed or spliced for f_y in tension.

18.13—Foundations**18.13.1 Scope****R18.13—Foundations****R18.13.1 Scope**

Requirements for foundations supporting buildings assigned to SDC C, D, E, or F represent a consensus of a minimum level of good practice in designing and detailing concrete foundations. However, because repairs to foundations can be extremely difficult and expensive, it may be desirable that the elements of the foundation remain essentially elastic during strong ground motions. Methods to achieve this goal include designing the foundation to include an overstrength factor or an increased seismic demand level when compared to the superstructure, or comparing strengths to demands predicted by nonlinear response history analyses with appropriate consideration of uncertainty in demands (Klemencic et al. 2012).

18.13.1.1 This section shall apply to foundations resisting earthquake-induced forces or transferring earthquake-induced forces between structure and ground.

18.13.1.2 The provisions in this section for piles, drilled piers, caissons, and slabs-on-ground shall supplement other applicable Code design and construction criteria, including 1.4.7 and 1.4.8.

18.13.2 Footings, foundation mats, and pile caps

18.13.2.1 The provisions of this section shall apply to structures assigned to SDC D, E, or F.

18.13.2.2 Longitudinal reinforcement of columns and structural walls resisting forces induced by earthquake effects shall extend into the footing, mat, or pile cap, and shall develop f_y in tension at the interface.

18.13.2.3 Columns designed assuming fixed-end conditions at the foundation shall comply with 18.13.2.2 and, if hooks are required, longitudinal reinforcement resisting flexure shall have 90-degree hooks near the bottom of the

R18.13.2 Footings, foundation mats, and pile caps

R18.13.2.3 Tests (Nilsson and Losberg 1976) have demonstrated that flexural members terminating in a footing, slab, or beam (a T-joint or L-joint) should have their hooks turned inward toward the axis of the member for the joint to

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foundation with the free end of the bars oriented toward the center of the column.

18.13.2.4 Columns or boundary elements of special structural walls that have an edge within one-half the footing depth from an edge of the footing shall have transverse reinforcement in accordance with 18.7.5.2 through 18.7.5.4 provided below the top of the footing. This reinforcement shall extend into the footing, mat, or pile cap a length equal to the development length, calculated for f_y in tension, of the column or boundary element longitudinal reinforcement.

18.13.2.5 Where earthquake effects create uplift forces in boundary elements of special structural walls or columns, flexural reinforcement shall be provided in the top of the footing, mat, or pile cap to resist actions resulting from the factored load combinations, and shall be at least that required by 7.6.1 or 9.6.1.

18.13.2.6 Structural plain concrete in footings and basement walls shall be in accordance with 14.1.3.

18.13.2.7 Pile caps incorporating batter piles shall be designed to resist the full compressive strength of the batter piles acting as short columns. The slenderness effects of batter piles shall be considered for the portion of the piles in soil that is not capable of providing lateral support, or in air or water.

18.13.3 Grade beams and slabs-on-ground

18.13.3.1 For structures assigned to SDC D, E, or F, grade beams and beams that are part of a mat foundation subjected to flexure from columns that are part of the seismic-force-resisting system shall be in accordance with 18.6.

18.13.3.2 For structures assigned to SDC C, D, E, or F, slabs-on-ground that resist in-plane earthquake forces from walls or columns that are part of the seismic-force-resisting system shall be designed as diaphragms in accordance with 18.12. The construction documents shall clearly indicate that the slab-on-ground is a structural diaphragm and part of the seismic-force-resisting system.

18.13.4 Foundation seismic ties

18.13.4.1 For structures assigned to SDC C, D, E, or F, individual pile caps, piers, or caissons shall be interconnected by foundation seismic ties in orthogonal directions,

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be able to resist the flexure in the member forming the stem of the T or L.

R18.13.2.4 Columns or boundary members supported close to the edge of the foundation, as often occurs near property lines, should be detailed to prevent an edge failure of the footing, pile cap, or mat.

R18.13.2.5 The purpose of this section is to emphasize that top reinforcement in footings, mats, and pile caps may be required, in addition to other required reinforcement.

R18.13.2.6 Foundation and basement walls should be reinforced in buildings assigned to SDC D, E, or F.

R18.13.2.7 Batter piles typically attract higher lateral forces during earthquakes than vertical piles. Extensive structural damage has been observed at the junction of batter piles and building foundations. The pile cap and surrounding structure should be designed for the potentially large forces that can be developed in batter piles.

18.13.3 Grade beams and slabs-on-ground

For earthquake conditions, slabs-on-ground (soil-supported slabs) are often part of the lateral-force-resisting system and should be designed in accordance with this Code as well as other appropriate standards or guidelines (refer to 1.4.8).

R18.13.3.1 Grade beams resisting flexural stresses from column moments should have reinforcement details similar to the beams of the frame above the foundation.

R18.13.3.2 Slabs-on-ground often act as a diaphragm to tie 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. The construction documents should clearly state that these slabs-on-ground are structural members so as to prohibit saw cutting of the slab.

18.13.4 Foundation seismic ties

R18.13.4.1 The foundation seismic ties should sufficiently interconnect foundations to act as a unit and be designed to minimize the relative movement of an individual column or tie relative to the foundation. This is essential where surface

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unless it can be demonstrated that equivalent restraint is provided by other means.

18.13.4.2 For structures assigned to SDC D, E, or F, individual spread footings founded on soil defined in ASCE/SEI 7 as Site Class E or F shall be interconnected by foundation seismic ties.

18.13.4.3 Where required, foundation seismic ties shall have a design strength in tension and compression at least equal to $0.1S_{DS}$ times the greater of the pile cap or column factored dead load plus factored live load unless it is demonstrated that equivalent restraint will be provided by (a), (b), (c), or (d):

- (a) Reinforced concrete beams within the slab-on-ground
- (b) Reinforced concrete slabs-on-ground
- (c) Confinement by competent rock, hard cohesive soils, or very dense granular soils
- (d) Other means approved by the building official

18.13.4.4 For structures assigned to SDC D, E, or F, grade beams designed to act as horizontal foundation seismic ties between pile caps or footings shall have continuous longitudinal reinforcement that shall be developed within or beyond the supported column or anchored within the pile cap or footing at all discontinuities and shall satisfy (a) and (b):

- (a) The smallest cross-sectional dimension of the grade beam shall be at least equal to the clear spacing between connected columns divided by 20, but need not exceed 18 in.
- (b) Closed tie transverse reinforcement shall be provided at a spacing not to exceed the lesser of 0.5 times the smallest orthogonal cross-sectional dimension and 12 in.

18.13.5 Deep foundations

18.13.5.1 This section shall apply to the following types of deep foundations

- (a) Uncased cast-in-place concrete drilled or augered piles
- (b) Metal cased concrete piles
- (c) Concrete filled pipe piles
- (d) Precast concrete piles

18.13.5.2 For structures assigned to SDC C, D, E, or F, piles, piers, or caissons resisting tension loads shall have continuous longitudinal reinforcement over their length to resist design tension forces.

18.13.5.3 For structures assigned to SDC C, D, E, or F, the minimum longitudinal and transverse reinforcement required by 18.13.5.7 through 18.13.5.10 shall be extended

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soils are soft enough to require deep foundations or where the site soils are susceptible to liquefaction.

R18.13.4.2 The ties between footings should have the same characteristics as the ties between pile caps in R18.13.4.1.

R18.13.4.3 The minimum foundation seismic tie design strength requirement based on a percentage of the factored dead plus live load provides a minimum connection between foundation elements. Other types of restraint can be used if substantiated as equivalent to the minimum tie design strength. The required design strength for the tie beam must be at least equal to $0.1S_{DS}$ times the larger force on either end of the tie beam, and that force is from the column or pile cap, whichever applies.

R18.13.5 Deep foundations

Adequate performance of piles and caissons for earthquake effects requires that these provisions be met in addition to other applicable standards or guidelines (refer to R1.4.7).

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over the entire unsupported length for the portion of deep foundation member in air or water, or in soil that is not capable of providing adequate lateral restraint to prevent buckling throughout this length.

18.13.5.4 For structures assigned to SDC C, D, E, or F, hoops, spirals, and ties in deep foundation members shall be terminated with seismic hooks.

18.13.5.5 For structures assigned to SDC D, E, or F or located in Site Class E or F, concrete deep foundation members shall have transverse reinforcement in accordance with 18.7.5.2, 18.7.5.3, and Table 18.7.5.4 Item (e) within seven member diameters above and below the interfaces between strata that are hard or stiff and strata that are liquefiable or soft.

18.13.5.6 For structures assigned to SDC D, E, or F, in foundations supporting one- and two-story stud bearing wall construction, concrete piles, piers or caissons, and foundation ties are exempt from the transverse reinforcement requirements of 18.13.5.3 through 18.13.5.5.

18.13.5.7 *Uncased cast-in-place drilled or augered concrete piles or piers*

18.13.5.7.1 For structures assigned to SDC C, D, E, or F, reinforcement shall be provided in uncased cast-in-place drilled or augered concrete piles where required by analysis and in accordance with the requirements in Table 18.13.5.7.1.

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support. For portions of the pile above ground, typically in air or water, or where soil is not capable of providing this lateral restraint, the minimum reinforced lengths should be increased, and the member should be designed as a column.

R18.13.5.5 During earthquakes, piles can be subjected to high flexural and shear demands at points of discontinuity, such as at interfaces between stiff and soft soil strata. ASCE/SEI 7 defines limits for soil strata. Transverse reinforcement is required in these regions to provide ductile behavior. In determining the portions of a pile with increased transverse reinforcement, accommodations are often made to the length of the reinforced zone for transverse reinforcement to account for variations in the driven pile tip elevations and variations in the interface elevations between stiff and soft soil strata.

R18.13.5.7 *Uncased cast-in-place drilled or augered concrete piles or piers*

R18.13.5.7.1 Longitudinal and transverse reinforcement requirements prescribed by this section result in ductility consistent with the applicable Seismic Design Category (SDC) to withstand ground deformation that occurs during earthquakes.

Where piles are subjected to significant uplift forces, the longitudinal reinforcement length required by analysis may exceed the minimum reinforcement length requirements.

Transverse reinforcement is required at the top of the pile to provide ductile performance where flexural yielding can potentially occur. For SDC D, E, and F and Site Classes A, B, C, and D, one-half of the transverse reinforcement for special moment frame columns is acceptable because some level of confinement is attributed to competent soils. For Site Class E and F, full column confinement is required because the soils are either liquefiable or not considered competent enough to provide confinement.

CODE**COMMENTARY****Table 18.13.5.7.1—Minimum reinforcement for uncased cast-in-place or augered concrete piles or piers**

Minimum reinforcement		SDC C – All Site Classes	SDC D, E, and F – Site Class A, B, C, and D	SDC D, E, and F – Site Class E and F
Minimum longitudinal reinforcement ratio (minimum number of bars)		0.0025 (minimum number of bars in accordance with 10.7.3.1)	0.005 (minimum number of bars in accordance with 10.7.3.1)	0.005 (minimum number of bars in accordance with 10.7.3.1)
Minimum reinforced pile length		Longest of (a) through (d): (a) 1/3 pile length (b) 10 ft (c) 3 times the pile diameter (d) Flexural length of pile - distance from bottom of pile cap to where $0.4M_{cr}$ exceeds M_u	Longest of (a) through (d): (a) 1/2 pile length (b) 10 ft (c) 3 times the pile diameter (d) Flexural length of pile - distance from bottom of pile cap to where $0.4M_{cr}$ exceeds M_u	Full length of pile except in accordance with [1] or [2].
Transverse confinement reinforcement zone	Length of reinforcement zone	3 times the pile diameter from the bottom of the pile cap	3 times the pile diameter from the bottom of the pile cap	7 times the pile diameter from the bottom of the pile cap
	Type of transverse reinforcement	Closed ties or spirals with a minimum 3/8 in. diameter	Minimum of No. 3 closed tie or 3/8 in. diameter spiral for piles ≤ 20 in. diameter Minimum No. 4 closed tie or 1/2 in. diameter spiral for piles > 20 in. diameter	In accordance with 18.7.5.2
	Spacing and amount of transverse reinforcement	Spacing shall not exceed lesser of 6 in. or 8 longitudinal bar diameters	In accordance with 18.7.5.3 and not less than one-half the requirement of Table 18.7.5.4 Item (e)	In accordance with 18.7.5.3 and not less than the requirement of Table 18.7.5.4 Item (e).
Transverse reinforcement in remainder of reinforced pile length	Type of transverse reinforcement	Closed ties or spirals with minimum 3/8 in. diameter	Minimum of No. 3 closed tie or 3/8 in. diameter spiral for piles ≤ 20 in. diameter Minimum of No. 4 closed tie or 1/2 in. diameter spiral for piles > 20 in. diameter	In accordance with 18.7.5.2
	Spacing and amount of transverse reinforcement	Maximum spacing of 16 longitudinal bar diameters	Spacing shall not exceed the least of (a) through (c): (a) 12 longitudinal bar diameters (b) 1/2 the pile diameter (c) 12 in.	

[1] For piles sufficiently embedded in firm soil or rock, reinforcement shall be permitted to be terminated a length above the tip equal to the lesser of 5 percent of the pile length and 33 percent of the length of the pile within rock or firm soil.

[2] In lieu of providing full length minimum flexural reinforcement, the deep foundation element shall be designed to withstand maximum imposed curvatures from the earthquake ground motions and structural response. Curvatures shall include free-field soil strains modified for soil-foundation-structure interaction coupled with foundation element deformations associated with earthquake loads imparted to the foundation by the structure. Minimum reinforced length shall not be less than the requirement for SDC D, E, or F; Site Class D.

18.13.5.7.2 Minimum longitudinal and transverse reinforcement shall be provided along minimum reinforced lengths measured from the top of the pile in accordance with Table 18.13.5.7.1.

18.13.5.7.3 Longitudinal reinforcement shall extend at least the development length, calculated for f_y in tension, beyond the flexural length of the pile, which is defined in Table 18.13.5.7.1 as the distance from the bottom of the pile cap to where $0.4M_{cr} > M_u$.

18.13.5.8 Metal-cased concrete piles

18.13.5.8.1 For structures assigned to SDC C, D, E, or F, longitudinal reinforcement requirements and minimum reinforced lengths for metal-cased concrete piles shall be the same as for uncased concrete piles in 18.13.5.7.

18.13.5.8.2 Metal-cased concrete piles shall have a spiral-welded metal casing of a thickness not less than 0.0747 in.

R18.13.5.7.3 Reinforcement should extend ℓ_d beyond the point where plain concrete is no longer adequate to resist the factored moment.

R18.13.5.8 Metal-cased concrete piles

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(No. 14 gauge) that is adequately protected from possible deleterious action due to soil constituents, changing water levels, or other factors indicated by boring records of site conditions.

18.13.5.9 Concrete-filled pipe piles

18.13.5.9.1 For structures assigned to SDC C, D, E or F, concrete-filled pipe piles shall have longitudinal reinforcement in the top of the pile with a total area of at least $0.01A_g$ and with a minimum length within the pile equal to two times the required embedment length into the pile cap, but not less than the development length, calculated for f_y in tension, of the reinforcement.

18.13.5.10 Precast concrete piles

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

18.13.5.10.2 Precast nonprestressed concrete piles for structures assigned to SDC C shall satisfy (a) through (d):

- (a) Minimum longitudinal steel reinforcement ratio shall be 0.01.
- (b) Longitudinal reinforcement shall be enclosed within a minimum of No. 3 closed ties or 3/8 in. diameter spirals, for up to 20 in. diameter piles, and No. 4 closed ties or 1/2 in. diameter spirals, for larger diameter piles.
- (c) Spacing of transverse reinforcement within a distance of 3 times the least cross-sectional dimension of the pile from the bottom of the pile cap shall not exceed the lesser of 8 times the diameter of the smallest longitudinal bar and 6 in.
- (d) Transverse reinforcement shall be provided throughout the length of the pile at a spacing not exceeding 6 in.

18.13.5.10.3 For structures assigned to SDC D, E, or F, precast nonprestressed concrete piles shall satisfy the requirements of 18.13.5.10.2 and the requirements for uncased cast-in-place or augered concrete piles in SDC D, E, or F in Table 18.13.5.7.1.

18.13.5.10.4 For structures assigned to SDC C, precast prestressed concrete piles shall satisfy (a) and (b):

- (a) If the transverse reinforcement consists of spirals or circular hoops, the volumetric ratio of transverse reinforcement, ρ_s , in the upper 20 ft shall not be less than that

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R18.13.5.8.2 Spiral-welded metal casing with the specified wall thickness provides confinement equivalent to closed ties or spirals required in an uncased concrete pile and eliminates the need for confinement ties.

R18.13.5.9 Concrete-filled pipe piles

R18.13.5.9.1 For resistance to uplift forces, concrete bond to the steel pipe is to be ignored in determining anchorage of the pile. Concrete shrinkage can be detrimental to bond, therefore shrinkage should be controlled, or force transfer via other methods such as headed studs or surface irregularities on the pipe should be considered. Reinforcement at the top of the pile is extended into the pile cap to tie the elements together and assist transfer of force to the pile cap.

R18.13.5.10 Precast concrete piles

R18.13.5.10.1 The potential for driving precast piles to a tip elevation different than that specified in the construction documents should be considered when detailing the pile. If the pile reaches refusal at a shallower depth, a longer length of pile will need to be cut off. If this possibility is not foreseen, the length of transverse reinforcement required by these provisions may not be provided after the excess pile length is cut off.

R18.13.5.10.4

(a) In a study of minimum confinement reinforcement for prestressed concrete piles (Sripathan et al. 2016), the relationship between curvature ductility demand on prestressed piles and overall system ductility demand was considered in the context of all soil profiles identified in ASCE/SEI 7. It was concluded that Eq. (18.13.5.10.4b)

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calculated by Eq. (18.13.5.10.4a) or calculated from a more detailed analysis by Eq. (18.13.5.10.4b):

$$0.15 \left(\frac{f'_c}{f'_{yt}} \right) \quad (18.13.5.10.4a)$$

$$0.04 \left(\frac{f'_c}{f'_{yt}} \right) \left(2.8 + \frac{2.3P_u}{f'_c A_g} \right) \quad (18.13.5.10.4b)$$

and f'_{yt} shall not be taken greater than 100,000 psi.

(b) A minimum of one-half of the volumetric ratio of spiral reinforcement required by Eq. (18.13.5.10.4a) or Eq. (18.13.5.10.4b) shall be provided for the remaining length of the pile.

18.13.5.10.5 For structures assigned to SDC D, E, or F, precast-prestressed concrete piles shall satisfy (a) through (e) and the ductile pile region shall be defined as the length of pile measured from the bottom of the pile cap to the point of zero curvature plus 3 times the least pile dimension, but not less than 35 ft. If the total pile length in the soil is 35 ft or less, the ductile pile region shall be taken as the entire length of the pile:

(a) In the ductile pile region, the center-to-center spacing of spirals or hoop reinforcement shall not exceed the least of 0.2 times the least pile dimension, 6 times the diameter of the longitudinal strand, and 6 in.

(b) Spiral reinforcement shall be spliced by lapping one full turn, by welding, or by the use of a mechanical splice. If spiral reinforcement is lap spliced, the ends of the spiral shall terminate in a seismic hook. Mechanical and welded splices of deformed bars shall comply with 25.5.7.

(c) If the transverse reinforcement consists of spirals, or circular hoops, the volumetric ratio of transverse reinforcement, ρ_s , in the ductile pile region shall not be less than that calculated by Eq. (18.13.5.10.5a) or calculated from a more detailed analysis by Eq. (18.13.5.10.5b), and the required volumetric ratio shall be permitted to be obtained by providing an inner and outer spiral.

$$0.2 \left(\frac{f'_c}{f'_{yt}} \right) \quad (18.13.5.10.5a)$$

$$0.06 \left(\frac{f'_c}{f'_{yt}} \right) \left(2.8 + \frac{2.3P_u}{f'_c A_g} \right) \quad (18.13.5.10.5b)$$

and f'_{yt} shall not be taken as greater than 100,000 psi.

(d) Outside of the ductile pile region, spiral or hoop reinforcement shall be provided with a volumetric ratio not less than one-half of that required within the ductile pile region, and the maximum spacing shall be in accordance with Table 13.4.5.6(b).

(e) If transverse reinforcement consists of rectangular hoops and crossties, the total cross-sectional area of lateral transverse reinforcement in the ductile region shall be the greater of Eq. (18.13.5.10.5c) and Eq. (18.13.5.10.5d). The hoops and crossties shall be equivalent to deformed

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results in adequate deformation capacity for structures assigned to SDC C. The factored axial force on a pile should be determined from Eq. (5.3.1c) and Eq. (5.3.1g) with 5.3.7 and 5.3.8 as applicable.

R18.13.5.10.5 Observed damage from earthquakes and concerns about the accuracy of calculated pile demands have led to prescriptive requirements for confinement of potential yielding regions of piles. The required confinement is intended to provide adequate ductility capacity for structures assigned to SDC D, E, and F (Sriharan et al. 2016).

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bars not less than No. 3 in size, and rectangular hoop ends shall terminate at a corner with seismic hooks.

$$A_{sh} = 0.3s b_c \left(\frac{f'_c}{f_{yt}} \right) \left(\frac{A_g}{A_{ch}} - 1.0 \right) \left(0.5 + \frac{1.4P_u}{f'_c A_g} \right)$$

(18.13.5.10.5c)

$$A_{sh} = 0.12s b_c \left(\frac{f'_c}{f_{yt}} \right) \left(0.5 + \frac{1.4P_u}{f'_c A_g} \right)$$

(18.13.5.10.5d)

and f_{yt} shall not be taken as greater than 100,000 psi.

18.13.5.10.6 For structures assigned to SDC C, D, E, or F, the maximum factored axial load that can be applied to precast, prestressed piles subjected to a combination of earthquake lateral force and axial load shall not exceed the following values:

- (a) $0.2f'_c A_g$ for square piles with side dimension of 14 in. or less
- (b) $0.25f'_c A_g$ for square piles with side dimension greater than 14 in.
- (c) $0.4f'_c A_g$ for circular or octagonal piles less than or equal to 24 in. in diameter
- (d) $0.45f'_c A_g$ for circular or octagonal piles greater than 24 in. in diameter

18.13.6 Anchorage of piles, piers, and caissons

18.13.6.1 For structures assigned to SDC C, D, E, or F, the longitudinal reinforcement in piles, piers, or caissons resisting tension loads shall be detailed to transfer tension forces within the pile cap to supported structural members.

18.13.6.2 For structures assigned to SDC C, D, E, or F, concrete piles and concrete filled pipe piles shall be connected to the pile cap by embedding the pile reinforcement in the pile cap a distance equal to the development length or by the use of field-placed dowels anchored in the concrete pile. For deformed bars, the compression development length is used if the pile is in compression. In the case of uplift, the tension development length is used without reduction in length for excess reinforcement.

18.13.6.3 For structures assigned to SDC D, E, or F, if tension forces induced by earthquake effects are transferred between pile cap or mat foundation and precast pile by reinforcing bars grouted or post-installed in the top of the pile, the grouting system shall have been demonstrated by testing to develop at least $1.25f_y$ of the bar.

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R18.13.5.10.6 The axial load in precast prestressed piles is limited to preclude spalling of the concrete cover prior to the pile section experiencing flexural cracking, as this will result in a significant loss in pile resistance (Sriharan et al. 2016).

Research (Ryan and Mays 2021) indicates an increase in axial load may be permitted for precast prestressed concrete piles with larger cross-sections; hence, the distinction in the axial load limit as a function of pile size.

R18.13.6 Anchorage of piles, piers, and caissons

R18.13.6.1 A load path is necessary at pile caps to transfer tension forces from the reinforcing bars in the column or boundary element through the pile cap to the reinforcement of the pile or caisson. Examples of different types of pile connections to pile caps are available in ASCE/COPRI Standard for the Seismic Design of Piers and Wharves (61-14).

R18.13.6.2 Development length is determined according to requirements of Chapter 25. Embedment lengths less than the development length for calculated stresses less than f_y are not permitted, as indicated in 25.4.10.2. Development of the pile longitudinal reinforcement into the pile cap is intended to enable the capacity of the pile to pile cap connection to meet or exceed the pile section strength.

R18.13.6.3 Grouted dowels in a blockout in the top of a precast concrete pile need to be developed, and testing is a practical means of demonstrating strength. Alternatively, reinforcing bars can be cast in the upper portion of the pile, exposed by chipping of concrete and mechanically spliced or welded to an extension.

CODE**18.14—Members not designated as part of the seismic-force-resisting system****COMMENTARY****R18.14—Members not designated as part of the seismic-force-resisting system**

This section applies only to structures assigned to SDC D, E, or F. For those SDCs, all structural members not designated as a part of the seismic-force-resisting system are required to be designed to support gravity loads and the load effects of vertical ground motion, while subjected to the design displacement. For concrete structures, the provisions of this section satisfy this requirement for columns, beams, slabs, and wall piers of the gravity system.

Design displacement is defined in [Chapter 2](#). Models used to determine design displacement of buildings should be chosen to produce results that conservatively bound the values expected during the design earthquake and should include, as appropriate, effects of concrete cracking, foundation flexibility, and deformation of floor and roof diaphragms.

The provisions of 18.14 are intended to enable ductile flexural yielding of columns, beams, slabs, and wall piers under the design displacement, by providing sufficient confinement and shear strength in elements that yield.

18.14.1 Scope

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

18.14.2 Design actions

18.14.2.1 Members not designated as part of the seismic-force-resisting system shall be evaluated for gravity load combinations of [5.3](#) including the effect of vertical ground motion acting simultaneously with the design displacement δ_u .

18.14.3 Cast-in-place beams, columns, and joints

18.14.3.1 Cast-in-place beams, columns, and joints shall be detailed in accordance with 18.14.3.2 or 18.14.3.3 depending on the magnitude of moments and shears induced in those members when subjected to the design displacement δ_u . If effects of δ_u are not explicitly checked, the provisions of 18.14.3.3 shall be satisfied.

18.14.3.2 Where the induced moments and shears do not exceed the design moment and shear strength of the frame member, (a) through (d) shall be satisfied:

- (a) Beams shall satisfy 18.6.3.1. Transverse reinforcement satisfying [25.7](#) shall be provided throughout the length of the beam at spacing not to exceed $d/2$. Where factored axial force exceeds $A_{gf_c}/10$, transverse reinforcement shall be hoops satisfying 18.7.5.2 at a spacing not to exceed the lesser of $6d_b$ of the smallest enclosed longitudinal bar and 6 in.

R18.14.3 Cast-in-place beams, columns, and joints

R18.14.3.1 Cast-in-place columns and beams are assumed to yield if the combined effects of factored gravity loads and design displacements exceed the strengths specified, or if the effects of design displacements are not calculated. Requirements for transverse reinforcement and shear strength vary with member type and whether the member yields under the design displacement.

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(b) Columns shall satisfy 18.7.4.1 and 18.7.6. Spiral reinforcement satisfying 25.7.3 or hoop reinforcement satisfying 25.7.4 shall be provided over the full length of the column with spacing not to exceed the lesser of $6d_b$ of the smallest enclosed longitudinal bar and 6 in. Transverse reinforcement satisfying 18.7.5.2(a) through (e) shall be provided over a length ℓ_o , as defined in 18.7.5.1, from each joint face.

(c) Columns with factored gravity axial forces exceeding $0.35P_o$ shall satisfy 18.14.3.2(b) and 18.7.5.7. The minimum amount of transverse reinforcement provided shall be, for rectilinear hoops, one-half the greater of Table 18.7.5.4 parts (a) and (b) and, for spiral or circular hoops, one-half the greater of Table 18.7.5.4 parts (d) and (e). This transverse reinforcement shall be provided over a length ℓ_o , as defined in 18.7.5.1, from each joint face.

(d) Joints shall satisfy Chapter 15.

18.14.3.3 Where the induced moments or shears exceed ϕM_n or ϕV_n of the frame member, or if induced moments or shears are not calculated, (a) through (d) shall be satisfied:

(a) Materials, mechanical splices, and welded splices shall satisfy the requirements for special moment frames in 18.2.5 through 18.2.8.

(b) Beams shall satisfy 18.14.3.2(a) and 18.6.5.

(c) Columns shall satisfy 18.7.4, 18.7.5, and 18.7.6, except 18.7.4.3 need not be satisfied.

(d) Joints shall satisfy 18.4.4.1.

18.14.4 Precast beams and columns

18.14.4.1 Precast concrete frame members assumed not to contribute to lateral resistance, including their connections, shall satisfy (a) through (d):

(a) Requirements of 18.14.3

(b) Ties specified in 18.14.3.2(b) over the entire column height, including the depth of the beams

(c) Structural integrity reinforcement, in accordance with 4.10

(d) Bearing length at the support of a beam shall be at least 2 in. longer than determined from 16.2.6

18.14.5 Slab-column connections

18.14.5.1 For slab-column connections of two-way slabs without beams, slab shear reinforcement satisfying the requirements of 18.14.5.3 and either 8.7.6 or 8.7.7 shall be provided at any slab critical section defined in 22.6.4.1 for the following conditions:

COMMENTARY**R18.14.4 Precast beams and columns**

R18.14.4.1 Damage to some buildings with precast concrete gravity systems during the 1994 Northridge earthquake was attributed to several factors addressed in this section. Columns should contain ties over their entire height, frame members not proportioned to resist earthquake forces should be tied together, and longer bearing lengths should be used to maintain integrity of the gravity system during ground motion. The 2 in. increase in bearing length is based on an assumed 4 percent story drift ratio and 50 in. beam depth, and is considered to be conservative for the ground motions expected for structures assigned to SDC D, E, or F. In addition to this provision, precast frame members assumed not to contribute to lateral resistance should also satisfy the requirements for cast-in-place construction addressed in 18.14.3, as applicable.

R18.14.5 Slab-column connections

R18.14.5.1 Provisions for shear reinforcement at slab-column connections are intended to reduce the likelihood of slab punching shear failure if the design story drift ratio exceeds the value specified.

No calculation of induced moments is required, based on research (Megally and Ghali 2002; Moehle 1996; Kang and

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- (a) Non prestressed slabs where $\Delta_x/h_{sx} \geq 0.035 - (1/20)(v_{uv}/\phi v_c)$
- (b) Unbonded post-tensioned slabs with f_{pc} in each direction meeting the requirements of 8.6.2.1, where $\Delta_x/h_{sx} \geq 0.040 - (1/20)(v_{uv}/\phi v_c)$

The load combinations to be evaluated for v_{uv} shall only include those with E . The value of (Δ_x/h_{sx}) shall be taken as the greater of the values of the adjacent stories above and below the slab-column connection, v_c shall be calculated in accordance with 22.6.5; and, for unbonded post-tensioned slabs, the value of V_p shall be taken as zero when calculating v_c .

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Wallace 2006; Kang et al. 2007) that identifies the likelihood of punching shear failure considering the story drift ratio and shear stress v_{uv} due to gravity loads and the vertical component of earthquake loads, without moment transfer, about the slab critical section. Figure R18.14.5.1 illustrates the requirement for non prestressed and unbonded post-tensioned slab-column connections. The requirement can be satisfied by adding slab shear reinforcement, increasing slab thickness, changing the design to reduce the design story drift ratio, or a combination of these.

If column capitals, drop panels, shear caps, or other changes in slab thickness are used, the requirements of 18.14.5 are evaluated at all potential critical sections, as required by 22.6.5.1.

Post-tensioned slab-column connections with f_{pc} in each direction not meeting the requirements of 8.6.2.1 can be designed as non prestressed slab-column connections in accordance with 8.2.3.

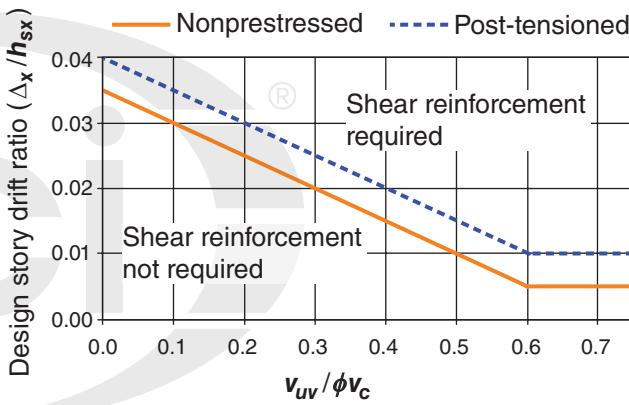


Fig. R18.14.5.1—Illustration of the criteria of 18.14.5.1.

18.14.5.2 The shear reinforcement requirements of 18.14.5.1 need not be satisfied if (a) or (b) is met:

- (a) $\Delta_x/h_{sx} \leq 0.005$ for non prestressed slabs
- (b) $\Delta_x/h_{sx} \leq 0.01$ for unbonded post-tensioned slabs with f_{pc} in each direction meeting the requirements of 8.6.2.1

18.14.5.3 Required slab shear reinforcement shall provide $v_s \geq 3.5\sqrt{f'_c}$ at the slab critical section and shall extend at least four times the slab thickness from the face of the support adjacent to the slab critical section.

18.14.6 Wall piers

18.14.6.1 Wall piers not designated as part of the seismic-force-resisting system shall satisfy the requirements of 18.10.8. Where the general building code includes provisions to account for overstrength of the seismic-force-resisting system, it shall be permitted to calculate the design shear force as Ω_o times the shear induced under design displacements, δ_u .

R18.14.6 Wall piers

R18.14.6.1 Section 18.10.8 requires that the design shear force be determined according to 18.7.6.1, which in some cases may result in unrealistically large forces. As an alternative, the design shear force can be determined as the product of an overstrength factor and the shear induced when the wall pier is displaced by δ_u . The overstrength factor Ω_o included in FEMA P-749, ASCE/SEI 7, and the 2021 IBC can be used for this purpose.