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Table 1804.2; IRC Table R401.4.1

Proponent: Dennis Dickard, PE, MPE, RBO, Hamilton County Building Inspections, OH

THESE PROPOSALS ARE ON THE AGENDA OF THE IBC STRUCTURAL AND THE IRC BUILDING/ENERGY CODE DEVELOPMENT COMMITTEES AS 2 SEPARATE CODE CHANGES. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

PART I – IBC STRUCTURAL

Delete table and substitute as follows:

~~TABLE 1804.2~~
~~ALLOWABLE FOUNDATION AND LATERAL PRESSURE~~

~~CLASS OF MATERIAL~~	~~ALLOWABLE FOUNDATION PRESSURE (psf)[d]~~	~~LATERAL BEARING (psf/f below natural grade)~~	~~LATERAL SLIDING~~	
			~~Coefficient of friction[a]~~	~~Resistance (psf)[b]~~
~~1. Crystalline bedrock~~	~~12,000~~	~~1,200~~	~~0.70~~	–
~~2. Sedimentary and foliated rock~~	~~4,000~~	~~400~~	~~0.35~~	–
~~3. Sandy gravel and/or gravel (GW and GP)~~	~~3,000~~	~~200~~	~~0.35~~	–
~~4. Sand, silty sand, clayey sand, silty gravel and clayey gravel (SW, SP, SM, SC, GM and GC)~~	~~2,000~~	~~150~~	~~0.25~~	–
~~5. Clay, sandy clay, silty clay, clayey silt, silt and sandy silt (CL, ML, MH and CH)~~	~~1,500[c]~~	~~100~~	~~–~~	~~130~~

~~For SI: 1 pound per square foot = 0.0479 kPa, 1 pound per square foot per foot = 0.157 kPa/m.~~

~~a. Coefficient to be multiplied by the dead load.~~
~~b. Lateral sliding resistance value to be multiplied by the contact area, as limited by Section 1804.3.~~
~~c. Where the building official determines that in-place soils with an allowable bearing capacity of less than 1,500 psf are likely to be present at the site, the allowablebearing capacity shall be determined by a soils investigation.~~
~~d. An increase of one-third is permitted when using the alternate load combinations in Section 1605.3.2 that include wind or earthquake loads.~~

TABLE 1804.2
ALLOWABLE FOUNDATION AND LATERAL PRESSURES[c]

CLASS OF MATERIAL[d]	CONSISTENCY IN PLACE	ALLOWABLE FOUNDATION PRESSURE (psf)[e]	LATERAL BEARING (psf/f below natural grade)	LATERAL SLIDING	
				Coefficient of friction[a]	Resistance (psf)[b]
1. Crystalline bedrock	-----	12,000	1,200	0.70	-----
2. Sedimentary and foliated rock	-----	6,000	400	0.35	-----
3. Sandy gravel and/or gravel (GW and GP)	firm	6,000	300	0.35	-----
	loose	4,000	200	0.35	-----
4. Sand, silty sand, clayey sand, silty gravel and clayey gravel (SW,SP,SM,SC,GM and GC)	firm	4,000	200	0.25	-----
	loose	3,000	150	0.25	-----
5. Clay, sandy clay, silty clay, clayey silt, silt and Sandy silt (CL,ML,MH and CH)	medium	2,500	130	-----	130
	soft	2,000	100	-----	130

For SI: 1 pound per square foot = 0.0479 kPa, 1 pound per square foot per foot = 0.157 kPa/m.

a. Coefficient to be multiplied by the dead load.
b. Lateral sliding resistance value to be multiplied by the contact area, as limited by Section 1804.3
c. Where the building official determines that in-place soils with an allowable capacity less than 2,000 psf are likely to be present at the site, the allowable bearing capacity shall be determined by a soils investigation

d. Firm consistency of class 4 and the medium consistency of class 5 can be molded by strong finger pressure, and the firm consistency of class 3 is too compact to be excavated with a shovel.
e. An increase of one-third is permitted when using the alternate load combinations in Section 1605.3.2 that include wind or earthquake loads.

PART II – IRC BUILDING/ENERGY

Delete tables and substitute as follows:

~~TABLE R401.4.1~~
~~PRESUMPTIVE LOAD–BEARING VALUES OF FOUNDATION MATERIALS~~[a]

~~CLASS OF MATERIAL~~	~~LOAD-BEARING PRESSURE (pounds per square foot~~
~~Crystalline bedrock~~	~~12,000~~
~~Sedimentary and foliated rock~~	~~4,000~~
~~Sandy gravel and/or gravel (GW and GP)~~	~~3,000~~
~~Sand, silty sand, clayey sand, silty gravel and clayey gravel (SW, SP, SM, SC, GM and GC)~~	~~2,000~~
~~Clay, sandy clay, silty clay, clayey silt, silt and sandy silt (CL, ML, MH and CH)~~	~~1,500b~~

TABLE R401.4.1
ALLOWABLE FOUNDATION AND LATERAL PRESSURES[c]

				LATERAL SLIDING	
CLASS OF MATERIAL[d]	CONSISTENCY IN PLACE	ALLOWABLE FOUNDATION PRESSURE (psf)	LATERAL BEARING (psf/f below natural grade)	Coefficient of friction[a]	Resistance (psf)[b]
1. Crystalline bedrock	-----	12,000	1,200	0.70	-----
2. Sedimentary and foliated rock	-----	6,000	400	0.35	-----
3. Sandy gravel and/or gravel (GW and GP)	firm	6,000	300	0.35	-----
	loose	4,000	200	0.35	-----
4. Sand, silty sand, clayey sand, silty gravel and clayey gravel (SW,SP,SM,SC,GM and GC)	firm	4,000	200	0.25	-----
	loose	3,000	150	0.25	-----
5. Clay, sandy clay, silty clay, clayey silt, silt and Sandy silt (CL,ML,MH and CH)	medium	2,500	130	-----	130
	soft	2,000	100	-----	130

For SI: 1 pound per square foot = 0.0479 kPa, 1 pound per square foot per foot = 0.157 kPa/m.

a. Coefficient to be multiplied by the dead load.
b. Lateral sliding resistance value to be multiplied by the contact area. For clay, sandy clay, silty clay and clayey silt in no case shall the lateral sliding resistance exceed ½ the dead load.
c. Where the building official determines that in-place soils with an allowable capacity less than 2,000 psf are likely to be present at the site, the allowable bearing capacity shall be determined upon R401.4, Soil tests.
d. Firm consistency of class 4 and the medium consistency of class 5 can be molded by strong finger pressure, and the firm consistency of class 3 is too compact to be excavated with a shovel.

Reason: The purpose of this proposed code change is to substitute new material for a current provision of the code.
In my professional opinion, this change is justified for the following reasons.

1. To provide consistency between table 1804.2 of the IBC and table R401.4.1 of the IRC.
2. The limited range of allowable values places undo hardships on certain residential and commercial structures. To expand the range of allowable foundation pressures permits the building official to use discretionary judgment.
3. Adding lateral pressures accommodates calculations for retaining walls and miscellaneous wall designs.
4. Previous code editions had successfully utilized higher values in the past.

References:

1. Department of the Navy. Foundations and Earth Structures Design Manual 7.2. Alexandria: Naval Facilities Engineering Command, 1982.
2. Soil Conservation Services. Engineering Geology. Washington D.C: United States Department of Agriculture, 1978.
3. American Society of Agricultural Engineers. Post and Pole Foundation Design Standard. St. Joseph: ASAE, 1997.

Cost Impact: The code change proposal will not increase the cost of construction.

PART I – IBC STRUCTURAL

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

PART II – IRC BUILDING/ENERGY

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S149–07/08

1610, 1807 (New), 1805.5.2, 1805.5.4, 1805.5.5, 1805.6

Proponent: Edwin T. Huston, Smith & Huston, Inc., representing National Council of Structural Engineering Associations

1. Revise as follows:

SECTION 1610
SOIL LATERAL LOADS

1610.1 General. ~~Basement, foundation~~ Foundation walls and retaining walls shall be designed to resist lateral soil loads. Soil loads specified in Table 1610.1 shall be used as the minimum design lateral soil loads unless specified otherwise in a soil investigation report approved by the building official. ~~Basement~~ Foundation walls and other walls in which horizontal movement is restricted at the top shall be designed for at-rest pressure. Retaining walls free to move and rotate at the top ~~are~~ shall be permitted to be designed for active pressure. Design lateral pressure from surcharge loads shall be added to the lateral earth pressure load. Design lateral pressure shall be increased if soils ~~with expansion potential are present at the site~~ at the site are expansive. Foundation walls shall be designed to support the weight of the full hydrostatic pressure of undrained backfill unless a drainage system is installed in accordance with Sections 1807.4.2 and 1807.4.3

> **Exception:** ~~Basement~~ Foundation walls extending not more than 8 feet (2438 mm) below grade and ~~supporting~~ laterally supported at the top by flexible ~~floor systems~~ diaphragms shall be permitted to be designed for active pressure.

TABLE 1610.1
~~SOIL~~ LATERAL SOIL LOAD

DESCRIPTION OF BACKFILL MATERIAL [c]	UNIFIED SOIL CLASSIFICATION	DESIGN LATERAL SOIL LOAD [a] (pounds per square foot per foot of depth)	
		Active pressure	At-rest pressure
Well-graded, clean gravels; gravel-sand mixes	GW	30	60
Poorly graded clean gravels; gravel-sand mixes	GP	30	60
Silty gravels, poorly graded gravel-sand mixes	GM	40	60
Clayey gravels, poorly graded gravel-and-clay mixes	GC	45	60
Well-graded, clean sands; gravelly sand mixes	SW	30	60
Poorly graded clean sands; sand-gravel mixes	SP	30	60
Silty sands, poorly graded sand-silt mixes	SM	45	60
Sand-silt clay mix with plastic fines	SM-SC	45	100
Clayey sands, poorly graded sand-clay mixes	SC	60	100
Inorganic silts and clayey silts	ML	45	100
Mixture of inorganic silt and clay	ML-CL	60	100
Inorganic clays of low to medium plasticity	CL	60	100
Organic silts and silt clays, low plasticity	OL	Note b	Note b
Inorganic clayey silts, elastic silts	MH	Note b	Note b
Inorganic clays of high plasticity	CH	Note b	Note b
Organic clays and silty clays	OH	Note b	Note b

For SI: 1 pound per square foot per foot of depth = 0.157 kPa/m, 1 foot = 304.8 mm.

a. Design lateral soil loads are given for moist conditions for the specified soils at their optimum densities. Actual field conditions shall govern. Submerged or saturated soil pressures shall include the weight of the buoyant soil plus the hydrostatic loads.
b. Unsuitable as backfill material.
c. The definition and classification of soil materials shall be in accordance with ASTM D 2487.

2. Add new text as follows:

SECTION 1807
FOUNDATION WALLS, RETAINING WALLS, AND EMBEDDED POSTS AND POLES

1807.1 Foundation walls. Foundation walls shall be designed and constructed in accordance with Sections 1807.1.1 through 1807.1.6. Foundation walls shall be supported by foundations designed in accordance with Section 1808.

1807.1.1 Design lateral soil loads. Foundation walls shall be designed for the lateral soil loads set forth in Section 1610.

1807.1.2 Unbalanced backfill height. Unbalanced backfill height is the difference in height between the exterior finish ground level and the lower of the top of the concrete footing that supports the foundation wall or the interior finish ground level. Where an interior concrete slab on grade is provided and is in contact with the interior surface of the foundation wall, the unbalanced backfill height shall be permitted to be measured from the exterior finish ground level to the top of the interior concrete slab.

1807.1.3 Rubble stone foundation walls. Foundation walls of rough or random rubble stone shall not be less than 16 inches (406 mm) thick. Rubble stone shall not be used for foundation walls for structures assigned to Seismic Design Category C, D, E or F.

1807.1.4 Permanent wood foundation systems. Permanent wood foundation systems shall be designed and installed in accordance with AF&PA Technical Report No. 7. Lumber and plywood shall be treated in accordance with AWPA U1 (Commodity Specification A, Use Category 4B and Section 5.2) and shall be identified in accordance with Section 2303.1.8.1.

1807.1.5 Concrete and masonry foundation walls. Concrete and masonry foundation walls shall be designed in accordance with Chapter 19 or 21 as applicable.

> **Exception**: Concrete and masonry foundation walls shall be permitted to be designed and constructed in accordance with Section 1807.1.6

1807.1.6 Prescriptive design of concrete and masonry foundation walls. Concrete and masonry foundation walls that are laterally supported at the top and bottom shall be permitted to be designed and constructed in accordance with this section.

1807.1.6.1 Foundation wall thickness. The thickness of prescriptively designed foundation walls shall not be less than the thickness of the wall supported, except that foundation walls of at least 8 inch (203 mm) nominal width shall be permitted to support brick-veneered frame walls and 10-inch-wide (254 mm) cavity walls provided the requirements of Section 1807.1.6.2 or 1807.1.6.3 are met.

1807.1.6.2 Concrete foundation walls. Concrete foundation walls shall comply with the following:

1. The thickness shall comply with the requirements of Table 1807.1.6.2.
2. The size and spacing of vertical reinforcement shown in Table 1807.1.6.2 is based on the use of reinforcement with a minimum yield strength of 60,000 psi (414 Mpa). Vertical reinforcement with a minimum yield strength of 40,000 psi (276 Mpa) or 50,000 psi (345 Mpa) shall be permitted, provided the same size bar is used and the spacing shown in the table is reduced by multiplying the spacing by 0.67 or 0.83, respectively.
3. Vertical reinforcement, when required, shall be placed nearest the inside face of the wall a distance, d, from the outside face (soil face) of the wall. The distance, d, is equal to the wall thickness, t, minus 1.25 inches (32 mm) plus one-half the bar diameter, d_b, [$d = t - (1.25 + d_b / 2)$]. The reinforcement shall be placed within a tolerance of ± 3/8 inch (9.5 mm) where d is less than or equal to 8 inches (203 mm) or ± 1/2 inch (12.7 mm) where d is greater than 8 inches (203 mm).
4. In lieu of the reinforcement shown in Table 1807.1.6.2, smaller reinforcing bar sizes with closer spacings that provide an equivalent cross-sectional area of reinforcement per unit length shall be permitted.

5. Concrete cover for reinforcement measured from the inside face of the wall shall not be less than 3/4 inch (19.1 mm). Concrete cover for reinforcement measured from the outside face of the wall shall not be less than 1.5 inches (38 mm) for No. 5 bars and smaller, and not less than 2 inches (51 mm) for larger bars.
6. Concrete shall have a specified compressive strength, f_c', of not less than 2,500 psi (17.2 MPa) at 28 days.
7. The unfactored axial load per linear foot of wall shall not exceed 1.2 t f_c' where t is the specified wall thickness in inches.

1807.1.6.2.1 Seismic requirements. Based on the seismic design category assigned to the structure in accordance with Section 1613, concrete foundation walls designed using Table 1807.1.6.2 shall be subject to the following limitations:

1. Seismic Design Categories A and B. No additional seismic requirements, except provide not less than two No. 5 bars around window and door openings. Such bars shall extend at least 24 inches (610 mm) beyond the corners of the openings.
2. Seismic Design Category C, D, E and F. Tables shall not be used except as allowed for plain concrete members in Section 1908.1.15.

TABLE 1807.1.6.2
CONCRETE FOUNDATION WALLS [b, c]

MAXIMUM WALL HEIGHT (feet)	**MAXIMUM UNBALANCED BACKFILL HEIGHT[e] (feet)**	**VERTICAL REINFORCEMENT AND SPACING (inches)**								
		Design lateral soil load[a] (psf per foot of depth)								
		30 [d]			**45 [d]**			**60**		
		Minimum wall thickness (inches)								
		7.5	9.5	11.5	7.5	9.5	11.5	7.5	9.5	11.5
5	4	PC	PC	PC	PC	PC	PC	PC	PC	PC
	5	PC	PC	PC	PC	PC	PC	PC	PC	PC
6	4	PC	PC	PC	PC	PC	PC	PC	PC	PC
	5	PC	PC	PC	PC	PC	PC	PC	PC	PC
	6	PC	PC	PC	PC	PC	PC	PC	PC	PC
7	4	PC	PC	PC	PC	PC	PC	PC	PC	PC
	5	PC	PC	PC	PC	PC	PC	PC	PC	PC
	6	PC	PC	PC	PC	PC	PC	#5 at 48"	PC	PC
	7	PC	PC	PC	#5 at 46"	PC	PC	#6 at 48"	PC	PC
8	4	PC	PC	PC	PC	PC	PC	PC	PC	PC
	5	PC	PC	PC	PC	PC	PC	PC	PC	PC
	6	PC	PC	PC	PC	PC	PC	#5 at 43"	PC	PC
	7	PC	PC	PC	#5 at 41"	PC	PC	#6 at 43"	PC	PC
	8	#5 at 47"	PC	PC	#6 at 43"	PC	PC	#6 at 32"	#6 at 44"	PC
9	4	PC	PC	PC	PC	PC	PC	PC	PC	PC
	5	PC	PC	PC	PC	PC	PC	PC	PC	PC
	6	PC	PC	PC	PC	PC	PC	#5 at 39"	PC	PC
	7	PC	PC	PC	#5 at 37"	PC	PC	#6 at 38"	#5 at 37"	PC
	8	#5 at 41"	PC	PC	#6 at 38"	#5 at 37"	PC	#7 at 39"	#6 at 39"	#4 at 48"
	9[d]	#6 at 46"	PC	PC	#7 at 41"	#6 at 41"	PC	#7 at 31"	#7 at 41"	#6 at 39"
10	4	PC	PC	PC	PC	PC	PC	PC	PC	PC
	5	PC	PC	PC	PC	PC	PC	PC	PC	PC
	6	PC	PC	PC	PC	PC	PC	#5 at 37"	PC	PC
	7	PC	PC	PC	#6 at 48"	PC	PC	#6 at 35"	#6 at 48"	PC
	8	#5 at 38"	PC	PC	#7 at 47"	#6 at 47"	PC	#7 at 35"	#7 at 47"	#6 at 45"
	9[d]	#6 at 41"	#4 at 48"	PC	#7 at 37"	#7 at 48"	#4 at 48"	#6 at 22"	#7 at 37"	#7 at 47"
	10[d]	#7 at 45"	#6 at 45"	PC	#7 at 31"	#7 at 40"	#6 at 38"	#6 at 22"	#7 at 30"	#7 at 38"

For SI: 1 inch = 25.4 mm, 1 foot = 304.8 mm, 1 pound per square foot per foot = 0.157 kPa/m.

a. For design lateral soil loads see Section 1610.
b. Provisions for this table are based on design and construction requirements specified in Section 1807.1.6.2.
c. “PC” means plain concrete.
d. Where unbalanced backfill height exceeds 8 feet and design lateral soil loads from Table 1610.1 are used, the requirements for 30 and 45 psf per foot of depth are not applicable. See Section 1610.
e. For height of unbalanced backfill, see Section 1807.1.2 .

1807.1.6.3 Masonry foundation walls. Masonry foundation walls shall comply with the following:

1. The thickness shall comply with the requirements of Table 1807.1.6.3(1) for plain masonry walls or Table 1807.1.6.3(2), 1807.1.6.3(3) or 1807.1.6.3(4) for masonry walls with reinforcement.
2. Vertical reinforcement shall have a minimum yield strength of 60,000 psi (414 Mpa).
3. The specified location of the reinforcement shall equal or exceed the effective depth distance, d, noted in Tables 1807.1.6.3(2), 1807.1.6.3(3) and 1807.1.6.3(4) ~~1805.5(2), 1805.5(3) and 1805.5(4)~~ and shall be measured from the face of the exterior (soil) side of the wall to the center of the vertical reinforcement. The reinforcement shall be placed within the tolerances specified in ACI 530.1/ASCE 6/TMS 402, Article 3.4 B7 of the specified location.
4. Grout shall comply with Section 2103.12.
5. Concrete masonry units shall comply with ASTM C 90.
6. Clay masonry units shall comply with ASTM C 652 for hollow brick, except compliance with ASTM C 62 or C 216 shall be permitted where solid masonry units are installed in accordance with Table 1807.1.6.3(1) for plain masonry.
7. Masonry units shall be laid in running bond and installed with Type M or S mortar in accordance with Section 2103.8.
8. The unfactored axial load per linear foot of wall shall not exceed 1.2 t f'_m where t is the specified wall thickness in inches and f'_m is the specified compressive strength of masonry in pounds per square inch.
9. At least 4 inches (102 mm) of solid masonry shall be provided at girder supports at the top of hollow masonry unit foundation walls.
10. Corbeling of masonry shall be in accordance with Section 2104.2. Where an 8-inch (203 mm) wall is corbeled, the top corbel shall not extend higher than the bottom of the floor framing and shall be a full course of headers at least 6 inches (152 mm) in length or the top course bed joint shall be tied to the vertical wall projection. The tie shall be W2.8 (4.8 mm) and spaced at a maximum horizontal distance of 36 inches (914 mm). The hollow space behind the corbelled masonry shall be filled with mortar or grout.

1807.1.6.3.1 Alternative foundation wall reinforcement. In lieu of the reinforcement provisions for masonry foundation walls in Table 1807.1.6.3(2), 1807.1.6.3(3) or 1807.1.6.3(4) alternative reinforcing bar sizes and spacings having an equivalent cross-sectional area of reinforcement per linear foot (mm) of wall shall be permitted to be used, provided the spacing of reinforcement does not exceed 72 inches (1829 mm) and reinforcing bar sizes do not exceed No. 11.

1807.1.6.3.2 Seismic requirements. Based on the seismic design category assigned to the structure in accordance with Section 1613, masonry foundation walls designed using Tables 1807.1.6.3(1) through 1807.1.6.3(4) shall be subject to the following limitations:

1. Seismic Design Categories A and B. No additional seismic requirements.
2. Seismic Design Category C. A design using Tables 1807.1.6.3(1) through 1807.1.6.3(4) is subject to the seismic requirements of Section 2106.4.
3. Seismic Design Category D. A design using Tables 1807.1.6.3(2) through 1807.1.6.3(4) is subject to the seismic requirements of Section 2106.5.
4. Seismic Design Categories E and F. A design using Tables 1807.1.6.3(2) through 1807.1.6.3(4) is subject to the seismic requirements of Section 2106.6.

TABLE 1807.1.6.3(1)
PLAIN MASONRY FOUNDATION WALLS [a, b, c]

MAXIMUM WALL HEIGHT (feet)	MAXIMUM UNBALANCED BACKFILL HEIGHT[e] (feet)	MINIMUM NOMINAL WALL THICKNESS (inches) Design lateral soil load[a] (psf per foot of depth) 30 [f]	45 [f]	60
7	4 (or less)	8	8	8
	5	8	10	10
	6	10	12	10 (solid[c])
	7	12	10 (solid[c])	10 (solid[c])
8	4 (or less)	8	8	8
	5	8	10	12
	6	10	12	12 (solid[c])
	7	12	12 (solid[c])	Note d
	8	10 (solid[c])	12 (solid[c])	Note d
9	4 (or less)	8	8	8
	5	8	10	12
	6	12	12	12 (solid[c])
	7	12 (solid[c])	12 (solid[c])	Note d
	8	12 (solid[c])	Note d	Note d
	9 [f]	Note d	Note d	Note d

For SI: 1 inch = 25.4 mm, 1 foot = 304.8 mm, 1 pound per square foot per foot = 0.157 kPa/m.

a. For design lateral soil loads, see Section 1610.
b. Provisions for this table are based on design and construction requirements specified in Section 1807.1.6.3.
c. Solid grouted hollow units or solid masonry units.
d. A design in compliance with Chapter 21 or reinforcement in accordance with Table 1807.1.6.3(2) is required.
e. For height of unbalanced backfill, see Section 1807.1.2.
f. Where unbalanced backfill height exceeds 8 feet and design lateral soil loads from Table 1610.1 are used, the requirements for 30 and 45 psf per foot of depth are not applicable. See Section 1610.

TABLE 1807.1.6.3(2)
8-INCH MASONRY FOUNDATION WALLS WITH REINFORCEMENT WHERE d ≥ 5 INCHES [a, b, c]

MAXIMUM WALL HEIGHT (feet-inches)	MAXIMUM UNBALANCED BACKFILL HEIGHT[d] (feet-inches)	VERTICAL REINFORCEMENT Design lateral soil load[a] (psf per foot of depth) 30 [e]	45 [e]	60
7-4	4-0 (or less)	#4 at 48" o.c.	#4 at 48" o.c.	#4 at 48" o.c.
	5-0	#4 at 48" o.c.	#4 at 48" o.c.	#4 at 48" o.c.
	6-0	#4 at 48" o.c.	#5 at 48" o.c.	#5 at 48" o.c.
	7-4	#5 at 48" o.c.	#6 at 48" o.c.	#7 at 48" o.c.
8-0	4-0 (or less)	#4 at 48" o.c.	#4 at 48" o.c.	#4 at 48" o.c.
	5-0	#4 at 48" o.c.	#4 at 48" o.c.	#4 at 48" o.c.
	6-0	#4 at 48" o.c.	#5 at 48" o.c.	#5 at 48" o.c.
	7-0	#5 at 48" o.c.	#6 at 48" o.c.	#7 at 48" o.c.
	8-0	#5 at 48" o.c.	#6 at 48" o.c.	#7 at 48" o.c.
8-8	4-0 (or less)	#4 at 48" o.c.	#4 at 48" o.c.	#4 at 48" o.c.
	5-0	#4 at 48" o.c.	#4 at 48" o.c.	#5 at 48" o.c.
	6-0	#4 at 48" o.c.	#5 at 48" o.c.	#6 at 48" o.c.
	7-0	#5 at 48" o.c.	#6 at 48" o.c.	#7 at 48" o.c.
	8-8 [e]	#6 at 48" o.c.	#7 at 48" o.c.	#8 at 48" o.c.
9-4	4-0 (or less)	#4 at 48" o.c.	#4 at 48" o.c.	#4 at 48" o.c.
	5-0	#4 at 48" o.c.	#4 at 48" o.c.	#5 at 48" o.c.
	6-0	#4 at 48" o.c.	#5 at 48" o.c.	#6 at 48" o.c.
	7-0	#5 at 48" o.c.	#6 at 48" o.c.	#7 at 48" o.c.
	8-0	#6 at 48" o.c.	#7 at 48" o.c.	#8 at 48" o.c.
	9-4 [e]	#7 at 48" o.c.	#8 at 48" o.c.	#9 at 48" o.c.

MAXIMUM WALL HEIGHT (feet-inches)	MAXIMUM UNBALANCED BACKFILL HEIGHT[d] (feet-inches)	VERTICAL REINFORCEMENT Design lateral soil load[a] (psf per foot of depth)		
		30 [e]	45 [e]	60
10-0	4-0 (or less)	#4 at 48" o.c.	#4 at 48" o.c.	#4 at 48" o.c.
	5-0	#4 at 48" o.c.	#4 at 48" o.c.	#5 at 48" o.c.
	6-0	#4 at 48" o.c.	#5 at 48" o.c.	#6 at 48" o.c.
	7-0	#5 at 48" o.c.	#6 at 48" o.c.	#7 at 48" o.c.
	8-0	#6 at 48" o.c.	#7 at 48" o.c.	#8 at 48" o.c.
	9-0 [e]	#7 at 48" o.c.	#8 at 48" o.c.	#9 at 48" o.c.
	10-0 [e]	#7 at 48" o.c.	#9 at 48" o.c.	#9 at 48" o.c.

For SI: 1 inch = 25.4 mm, 1 foot = 304.8 mm, 1 pound per square foot per foot = 0.157 kPa/m.

a. For design lateral soil loads, see Section 1610
b. Provisions for this table are based on design and construction requirements specified in Section 1807.1.6.3
c. For alternative reinforcement, see Section 1807.1.6.3.1
d. For height of unbalanced backfill, see Section 1807.1.2
e. Where unbalanced backfill height exceeds 8 feet and design lateral soil loads from Table 1610.1 are used, the requirements for 30 and 45 psf per foot of depth are not applicable. See Section 1610.

TABLE 1807.1.6.3(3)
10-INCH MASONRY FOUNDATION WALLS WITH REINFORCEMENT WHERE d ≥ 6.75 INCHES [a, b, c]

MAXIMUM WALL HEIGHT (feet-inches)	MAXIMUM UNBALANCED BACKFILL HEIGHT[d] (feet-inches)	VERTICAL REINFORCEMENT Design lateral soil load[a] (psf per foot of depth)		
		30 [e]	45 [e]	60
7-4	4-0 (or less)	#4 at 56" o.c.	#4 at 56" o.c.	#4 at 56" o.c.
	5-0	#4 at 56" o.c.	#4 at 56" o.c.	#4 at 56" o.c.
	6-0	#4 at 56" o.c.	#4 at 56" o.c.	#5 at 56" o.c.
	7-4	#4 at 56" o.c.	#5 at 56" o.c.	#6 at 56" o.c.
8-0	4-0 (or less)	#4 at 56" o.c.	#4 at 56" o.c.	#4 at 56" o.c.
	5-0	#4 at 56" o.c.	#4 at 56" o.c.	#4 at 56" o.c.
	6-0	#4 at 56" o.c.	#4 at 56" o.c.	#5 at 56" o.c.
	7-0	#4 at 56" o.c.	#5 at 56" o.c.	#6 at 56" o.c.
	8-0	#5 at 56" o.c.	#6 at 56" o.c.	#7 at 56" o.c.
8-8	4-0 (or less)	#4 at 56" o.c.	#4 at 56" o.c.	#4 at 56" o.c.
	5-0	#4 at 56" o.c.	#4 at 56" o.c.	#4 at 56" o.c.
	6-0	#4 at 56" o.c.	#4 at 56" o.c.	#5 at 56" o.c.
	7-0	#4 at 56" o.c.	#5 at 56" o.c.	#6 at 56" o.c.
	8-8 [e]	#5 at 56" o.c.	#7 at 56" o.c.	#8 at 56" o.c.
9-4	4-0 (or less)	#4 at 56" o.c.	#4 at 56" o.c.	#4 at 56" o.c.
	5-0	#4 at 56" o.c.	#4 at 56" o.c.	#4 at 56" o.c.
	6-0	#4 at 56" o.c.	#5 at 56" o.c.	#5 at 56" o.c.
	7-0	#4 at 56" o.c.	#5 at 56" o.c.	#6 at 56" o.c.
	8-0	#5 at 56" o.c.	#6 at 56" o.c.	#7 at 56" o.c.
	9-4 [e]	#6 at 56" o.c.	#7 at 56" o.c.	#8 at 56" o.c.
10-0	4-0 (or less)	#4 at 56" o.c.	#4 at 56" o.c.	#4 at 56" o.c.
	5-0	#4 at 56" o.c.	#4 at 56" o.c.	#4 at 56" o.c.
	6-0	#4 at 56" o.c.	#5 at 56" o.c.	#5 at 56" o.c.
	7-0	#5 at 56" o.c.	#6 at 56" o.c.	#7 at 56" o.c.
	8-0	#5 at 56" o.c.	#7 at 56" o.c.	#8 at 56" o.c.
	9-0 [e]	#6 at 56" o.c.	#7 at 56" o.c.	#9 at 56" o.c.
	10-0 [e]	#7 at 56" o.c.	#8 at 56" o.c.	#9 at 56" o.c.

For SI: 1 inch = 25.4 mm, 1 foot = 304.8 mm, 1 pound per square foot per foot = 0.157 kPa/m.

a. For design lateral soil loads, see Section 1610.
b. Provisions for this table are based on design and construction requirements specified in Section 1807.1.6.3.
c. For alternative reinforcement, see Section 1807.1.6.3.
d. For height of unbalanced backfill, see Section 1807.1.2.
e. Where unbalanced backfill height exceeds 8 feet and design lateral soil loads from Table 1610.1 are used, the requirements for 30 and 45 psf per foot of depth are not applicable. See Section 1610.

TABLE 1807.1.6.3(4)
12-INCH MASONRY FOUNDATION WALLS WITH REINFORCEMENT WHERE d ≥ 8.75 INCHES [a, b, c]

MAXIMUM WALL HEIGHT (feet-inches)	MAXIMUM UNBALANCED BACKFILL HEIGHT[d] (feet-inches)	VERTICAL REINFORCEMENT Design lateral soil load[a] (psf per foot of depth) 30 [e]	45 [e]	60
7-4	4-0 (or less)	#4 at 72" o.c.	#4 at 72" o.c.	#4 at 72" o.c.
	5-0	#4 at 72" o.c.	#4 at 72" o.c.	#4 at 72" o.c.
	6-0	#4 at 72" o.c.	#4 at 72" o.c.	#5 at 72" o.c.
	7-4	#4 at 72" o.c.	#5 at 72" o.c.	#6 at 72" o.c.
8-0	4-0 (or less)	#4 at 72" o.c.	#4 at 72" o.c.	#4 at 72" o.c.
	5-0	#4 at 72" o.c.	#4 at 72" o.c.	#4 at 72" o.c.
	6-0	#4 at 72" o.c.	#4 at 72" o.c.	#5 at 72" o.c.
	7-0	#4 at 72" o.c.	#5 at 72" o.c.	#6 at 72" o.c.
	8-0	#5 at 72" o.c.	#6 at 72" o.c.	#7 at 72" o.c.
8-8	4-0 (or less)	#4 at 72" o.c.	#4 at 72" o.c.	#4 at 72" o.c.
	5-0	#4 at 72" o.c.	#4 at 72" o.c.	#4 at 72" o.c.
	6-0	#4 at 72" o.c.	#4 at 72" o.c.	#5 at 72" o.c.
	7-0	#4 at 72" o.c.	#5 at 72" o.c.	#6 at 72" o.c.
	8-8 [e]	#5 at 72" o.c.	#7 at 72" o.c.	#8 at 72" o.c.
9-4	4-0 (or less)	#4 at 72" o.c.	#4 at 72" o.c.	#4 at 72" o.c.
	5-0	#4 at 72" o.c.	#4 at 72" o.c.	#4 at 72" o.c.
	6-0	#4 at 72" o.c.	#5 at 72" o.c.	#5 at 72" o.c.
	7-0	#4 at 72" o.c.	#5 at 72" o.c.	#6 at 72" o.c.
	8-0	#5 at 72" o.c.	#6 at 72" o.c.	#7 at 72" o.c.
	9-4 [e]	#6 at 72" o.c.	#7 at 72" o.c.	#8 at 72" o.c.
10-0	4-0 (or less)	#4 at 72" o.c.	#4 at 72" o.c.	#4 at 72" o.c.
	5-0	#4 at 72" o.c.	#4 at 72" o.c.	#4 at 72" o.c.
	6-0	#4 at 72" o.c.	#5 at 72" o.c.	#5 at 72" o.c.
	7-0	#4 at 72" o.c.	#6 at 72" o.c.	#6 at 72" o.c.
	8-0	#5 at 72" o.c.	#6 at 72" o.c.	#7 at 72" o.c.
	9-0 [e]	#6 at 72" o.c.	#7 at 72" o.c.	#8 at 72" o.c.
	10-0 [e]	#7 at 72" o.c.	#8 at 72" o.c.	#9 at 72" o.c.

For SI: 1 inch = 25.4 mm, 1 foot = 304.8 mm, 1 pound per square foot per foot = 0.157 kPa/m.

a. For design lateral soil loads, see Section 1610.
b. Provisions for this table are based on design and construction requirements specified in Section 1807.1.6.3.
c. For alternative reinforcement, see Section 1807.1.6.3.1.
d. For height of unbalanced backfill, see Section 1807.1.2.
e. Where unbalanced backfill height exceeds 8 feet and design lateral soil loads from Table 1610.1 are used, the requirements for 30 and 45 psf per foot of depth are not applicable. See Section 1610.

1807.2 Retaining walls. Retaining walls shall be designed in accordance with Sections 1807.2.1 through 1807.2.3.

1807.2.1 General. Retaining walls shall be designed to ensure stability against overturning, sliding, excessive foundation pressure and water uplift.

1807.2.2 Design lateral soil loads. Retaining walls shall be designed for the lateral soil loads set forth in Section 1610.

1807.2.3 Safety factor. Retaining walls shall be designed to resist the lateral action of soil to produce sliding and overturning with a safety factor of 1.5. The load combinations of Section 1605.3 shall not apply to these requirements.

1807.3 Embedded posts and poles. Designs to resist both axial and lateral loads employing posts or poles as columns embedded in earth or embedded in concrete footings in the earth shall be in accordance with Sections 1807.3.1 through 1807.3.3.

1807.3.1 Limitations. The design procedures outlined in this section are subject to the following limitations:

1. The frictional resistance for structural walls and slabs on silts and clays shall be limited to one-half of the normal force imposed on the soil by the weight of the footing or slab.

2. Posts embedded in earth shall not be used to provide lateral support for structural or nonstructural materials such as plaster, masonry or concrete unless bracing is provided that develops the limited deflection required.

Wood poles shall be treated in accordance with AWPA U1 for sawn timber posts (Commodity Specification A, Use Category 4B), and for round timber posts (Commodity Specification B, Use Category 4B).

1807.3.2 Design criteria. The depth to resist lateral loads shall be determined using the design criteria established in Sections 1807.3.2.1 through 1807.3.2.3, or by other methods approved by the building official.

1807.3.2.1 Nonconstrained. The following formula shall be used in determining the depth of embedment required to resist lateral loads where no lateral constraint is provided at the ground surface, such as by a rigid floor or rigid ground surface pavement, and where no lateral constraint is provided above the ground surface, such as by a structural diaphragm.

$$d = 0.5 A \{1 + [1 + (4.36 h / A)]^{1/2}\} \qquad \textbf{(Equation 18-1)}$$

where:

A = $2.34 P / S_1 b$.
b = Diameter of round post or footing or diagonal dimension of square post or footing, feet (m).
d = Depth of embedment in earth in feet (m) but not over 12 feet (3658 mm) for purpose of computing lateral pressure.
h = Distance in feet (m) from ground surface to point of application of "P."
P = Applied lateral force in pounds (kN).
S_1 = Allowable lateral soil-bearing pressure as set forth in Section 1804.3 based on a depth of one-third the depth of embedment in pounds per square foot (psf) (kPa).

1807.3.2.2 Constrained. The following formula shall be used to determine the depth of embedment required to resist lateral loads where lateral constraint is provided at the ground surface, such as by a rigid floor or pavement.

$$d^2 = 4.25 (P h / S_3 b) \qquad \textbf{(Equation 18-2)}$$

or alternatively

$$d^2 = 4.25 (M_g / S_3 b) \qquad \textbf{(Equation 18-3)}$$

where:

M_g = Moment in the post at grade, in foot-pounds (kN-m).
S_3 = Allowable lateral soil-bearing pressure as set forth in Section 1804.3 based on a depth equal to the depth of embedment in pounds per square foot (kPa).

1807.3.2.3 Vertical load. The resistance to vertical loads shall be determined using the vertical foundation pressure set forth in Table 1804.2.

1807.3.3 Backfill. The backfill in the annular space around columns not embedded in poured footings shall be by one of the following methods:

1. Backfill shall be of concrete with an ultimate strength of 2,000 psi (13.8 MPa) at 28 days. The hole shall not be less than 4 inches (102 mm) larger than the diameter of the column at its bottom or 4 inches (102 mm) larger than the diagonal dimension of a square or rectangular column.
2. Backfill shall be of clean sand. The sand shall be thoroughly compacted by tamping in layers not more than 8 inches (203 mm) in depth.
3. Backfill shall be of controlled low-strength material (CLSM).

3. Delete without substitute:

~~**1805.4.6 Wood foundations.** Wood foundation systems shall be designed and installed in accordance with AF&PA Technical Report No. 7. Lumber and plywood shall be treated in accordance with AWPA U1 (Commodity Specification A, Use Category 4B and Section 5.2) and shall be identified in accordance with Section 2303.1.8.1.~~

~~**1805.5 Foundation walls.** Concrete and masonry foundation walls shall be designed in accordance with Chapter 19 or 21, respectively. Foundation walls that are laterally supported at the top and bottom and within the parameters of Tables 1805.5(1) through 1805.5(5) are permitted to be designed and constructed in accordance with Sections 1805.5.1 through 1805.5.5.~~

~~**1805.5.1 Foundation wall thickness.** The minimum thickness of concrete and masonry foundation walls shall comply with Sections 1805.5.1.1 through 1805.5.1.3.~~

~~**1805.5.1.1 Thickness at top of foundation wall.** The thickness of foundation walls shall not be less than the thickness of the wall supported, except that foundation walls of at least 8-inch (203 mm) nominal width are permitted to support brick-veneered frame walls and 10-inch-wide (254 mm) cavity walls provided the requirements of Section 1805.5.1.2 are met. Corbeling of masonry shall be in accordance with Section 2104.2. Where an 8-inch (203 mm) wall is corbeled, the top corbel shall not extend higher than the bottom of the floor framing and shall be a full course of headers at least 6 inches (152 mm) in length or the top course bed joint shall be tied to the vertical wall projection. The tie shall be W2.8 (4.8 mm) and spaced at a maximum horizontal distance of 36 inches (914 mm); the hollow space behind the corbelled masonry shall be filled with mortar or grout.~~

~~**1805.5.1.2 Thickness based on soil loads, unbalanced backfill height and wall height.** The thickness of foundation walls shall comply with the requirements of Table 1805.5(5) for concrete walls, Table 1805.5(1) for plain masonry walls or Table 1805.5(2), 1805.5(3) or 1805.5(4) for masonry walls with reinforcement. When using the tables, masonry shall be laid in running bond and the mortar shall be Type M or S.~~

~~**1805.5.1.3 Rubble stone.** Foundation walls of rough or random rubble stone shall not be less than 16 inches (406 mm) thick. Rubble stone shall not be used for foundations for structures in Seismic Design Category C, D, E or F.~~

~~Unbalanced backfill height is the difference in height between the exterior finish ground level and the lower of the top of the concrete footing that supports the foundation wall or the interior finish ground level. Where an interior concrete slab on grade is provided and is in contact with the interior surface of the foundation wall, the unbalanced backfill height is permitted to be measured from the exterior finish ground level to the top of the interior concrete slab.~~

~~**1805.5.2 Foundation wall materials.** Concrete foundation walls constructed in accordance with Table 1805.5(5) shall comply with Section 1805.5.2.1. Masonry foundation walls constructed in accordance with Table 1805.5(1), 1805.5(2), 1805.5(3) or 1805.5(4) shall comply with Section 1805.5.2.2.~~

~~**1805.5.4 Hollow masonry walls**. At least 4 inches (102 mm) of solid masonry shall be provided at girder supports at the top of hollow masonry unit foundation walls.~~

~~**1805.5.5 Seismic requirements**. Tables 1805.5(1) through 1805.5(5) shall be subject to the following limitations in Sections 1805.5.5.1 and 1805.5.5.2 based on the seismic design category assigned to the structure as defined in Section 1613.~~

~~**1805.5.6 Foundation wall drainage.** Foundation walls shall be designed to support the weight of the full hydrostatic pressure of undrained backfill unless a drainage system is installed in accordance with Sections 1807.4.2 and 1807.4.3.~~

~~**1805.6 Foundation plate or sill bolting**. Wood foundation plates or sills shall be bolted or strapped to the foundation or foundation wall as provided in Chapter 23.~~

~~**1805.7 Designs employing lateral bearing.** Designs to resist both axial and lateral loads employing posts or poles as columns embedded in earth or embedded in concrete footings in the earth shall conform to the requirements of Sections 1805.7.1 through 1805.7.3.~~

~~**1805.7.1 Limitations.** The design procedures outlined in this section are subject to the following limitations:~~

1. ~~The frictional resistance for structural walls and slabs on silts and clays shall be limited to one-half of the normal force imposed on the soil by the weight of thefooting or slab.~~
2. ~~Posts embedded in earth shall not be used to provide lateral support for structural or nonstructural materials such as plaster, masonry or concrete unless bracing is provided that develops the limited deflection required.~~

~~Wood poles shall be treated in accordance with AWPA U1 for sawn timber posts (Commodity Specification A, Use Category 4B) and for round timber posts (Commodity Specification B, Use Category 4B).~~

~~**1805.7.2 Design criteria.** The depth to resist lateral loads shall be determined by the design criteria established in Sections 1805.7.2.1 through 1805.7.2.3, or by other methods approved by the building official.~~

~~**1805.7.2.1 Nonconstrained.** The following formula shall be used in determining the depth of embedment required to resist lateral loads where no constraint is provided at the ground surface, such as rigid floor or rigid ground surface pavement, and where no lateral constraint is provided above the ground surface, such as a structural diaphragm.~~

~~$d = 0.5A\{1 + [1 + (4.36h/A)]1/2\}$ **(Equation 18-1)**~~

~~where:~~

~~$A = 2.34P/S1\ b$.~~
~~b = Diameter of round post or footing or diagonal dimension of square post or footing, feet (m).~~
~~d = Depth of embedment in earth in feet (m) but not over 12 feet (3658 mm) for purpose of computing lateral pressure.~~
~~h = Distance in feet (m) from ground surface to point of application of “P.”~~
~~P = Applied lateral force in pounds (kN).~~
~~$S1$ = Allowable lateral soil-bearing pressure as set forth in Section 1804.3 based on a depth of one-third the depth of embedment in pounds per square foot (psf) (kPa).~~

~~**1805.7.2.2 Constrained.** The following formula shall be used to determine the depth of embedment required to resist lateral loads where constraint is provided at the ground surface, such as a rigid floor or pavement.~~

~~$d2 = 4.25(Ph/S3\ b)$ **(Equation 18-2)**~~

~~or alternatively~~

~~$d2 = 4.25\ (Mg/S3\ b)$ **(Equation 18-3)**~~

~~where:~~

~~Mg = Moment in the post at grade, in foot-pounds (kN-m).~~

~~$S3$ = Allowable lateral soil-bearing pressure as set forth in Section 1804.3 based on a depth equal to the depth of embedment in pounds per square foot (kPa).~~

~~**1805.7.2.3 Vertical load.** The resistance to vertical loads shall be determined by the allowable soil-bearing pressure set forth in Table 1804.2.~~

~~**1805.7.3 Backfill.** The backfill in the annular space around columns not embedded in poured footings shall be by one of the following methods:~~

1. ~~Backfill shall be of concrete with an ultimate strength of 2,000 psi (13.8 MPa) at 28 days. The hole shall not be less than 4 inches (102 mm) larger than the diameter of the column at its bottom or 4 inches (102 mm) larger than the diagonal dimension of a square or rectangular column.~~
2. ~~Backfill shall be of clean sand. The sand shall be thoroughly compacted by tamping in layers not more than 8 inches (203 mm) in depth.~~
3. ~~Backfill shall be of controlled low-strength material (CLSM).~~

~~SECTION 1806~~
~~RETAINING WALLS~~

~~**1806.1 General.** Retaining walls shall be designed to ensure stability against overturning, sliding, excessive foundation pressure and water uplift. Retainingwalls shall be designed for a safety factor of 1.5 against lateral sliding and overturning.~~

Reason: Code clarification.

Makes terminology in 1610.1 consistent with Chapter 18; the IBC does not recognize “basement walls,” instead calling them “foundation walls.” Moves the loading requirement of Section 1805.5.6 to 1610.1, where the related loading requirements occur. Makes the scoping language of the Section 1610.1 exception consistent with the sections concerning foundation walls (current Section 1805.5); the key condition is that walls are “laterally supported at the top”, not that the walls are “supporting” a floor. Also, “flexible floor systems” are undefined, but “flexible diaphragms” are defined.

Editorial reorganization to group requirements related to laterally loaded elements (foundation walls, retaining walls, and embedded posts and poles). Recognizes the distinction between foundation walls and foundations. Moves reference to design lateral soil loads from footnotes to text. Moves the definition of unbalanced backfill height from the prescriptive design requirements to a general section. Moves the general requirements related to rubble stone out of the prescriptive design requirements where it does not apply (as rubble stone is not laid in running bond).

Moves requirements related to permanent wood foundation systems to this section on foundation walls. According to AF&PA PWF-06, such systems are "engineered to support lateral soil pressures as well as dead, live, snow, wind, and seismic loads," where the foundation wall behavior is primary and vertical load behavior is secondary. That document also repeated refers to "foundation walls" (for instance, see Figures 3 and 4).

Makes footnotes for prescriptively designed concrete and masonry foundation walls more consistent. Separates prescriptive design requirements for concrete and masonry foundation walls.

In each of the tables, clarifies the important footnote related to applicability of 30 and 45 psf design loads. Where the unbalanced backfill height exceeds 8 feet, Section 1610.1 requires that foundation walls be designed for at-rest pressures. Where lateral soil loads from Table 1610.1 are used, values of 30 and 45 psf occur only for active pressure. Therefore, where both of these conditions are satisfied, the prescriptive design table entries for 30 and 45 psf cannot be used.

For prescriptively designed masonry foundation walls, the revision to item 7 (running bond) was in 1805.5.1.2, item 9 was in Section 1805.5.4, and item 10 was in Section 1805.5.1.1.

Section 1805.6 is not needed. As that section indicates, the requirements for sill plate anchorage appear in Chapter 23 (Sections 2305.3.11 and 2308.6).

The safety factor for stability of retaining walls predates modern load combinations. The revised text makes clear that load combinations do not apply to consideration of retaining wall sliding and overturning.

Correlation note: In Section 1704.5 exception 2 change "Table 1805.5(1), 1805.5(2), 1805.5(3) or 1805.5(4)" to "Table 1807.1.6.3(1), 1807.1.6.3(2), 1807.1.6.3(3) or 1807.1.6.3(4)".

Bibliography:
Composite of Chapter 18 reorganization assuming all of proponent's proposals are approved.

Cost Impact: The code change proposal will not increase the cost of construction.

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

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1801.2, 1801.2.1, 1805, 1808, 1809, 1809.2..1, 1908.2.2.5, 1809.2.3.5, 1810.1.1, 1810.1.3, 1810.2.5, 1810.3.4, 1810.4.4, 1810.6.5, 1812.3, 1812.5, 1812.10, 1812.7, 1808.2.23.2, 1809.2.3.1, 1810.3.4, 1810.5.4, 1810.6.4, 1810.7.2, 1810.7.4, 1810.7.6, 1810.8.2

Proponent: Edwin T. Huston, Smith & Huston, Inc., representing the National Council of Structural Engineering Associations

1. Revise as follows:

1801.2 Design basis. Allowable bearing pressures, allowable stresses and design formulas provided in this chapter shall be used with the allowable stress design load combinations specified in Section 1605.3. The quality and design of materials used structurally in excavations~~, footings~~ and foundations shall comply with ~~conform~~ to the requirements specified in Chapters 16, 19, 21, 22 and 23 of this code. Excavations and fills shall also comply with Chapter 33.

2. Delete without substitution:

~~**1801.2.1 Foundation design for seismic overturning.** Where the foundation is proportioned using the load combinations of Section 1605.2, and the computation of the seismic overturning moment is by the equivalent lateral-force method or the modal analysis method, the proportioning shall be in accordance with Section 12.13.4 of ASCE 7.~~

~~**1805.1 (Supp) General.** Footings and foundations shall be designed and constructed in accordance with Sections 1805.1 through 1805.9. Footings and foundations shall be built on undisturbed soil, compacted fill material or controlled low strength material (CLSM). Compacted fill material shall be placed in accordance with Section 1803.5. CLSM shall be placed in accordance with Section 1803.6.~~

~~The top surface of footings shall be level. The bottom surface of footings is permitted to have a slope not exceeding one unit vertical in 10 units horizontal (10-percent slope). Footings shall be stepped where it is necessary to change the elevation of the top surface of the footing or where the surface of the ground slopes more than one unit vertical in 10 units horizontal (10-percent slope).~~

~~**1805.2 Depth of footings.** The minimum depth of footings below the undisturbed ground surface shall be 12 inches (305 mm). Where applicable, the depth of footings shall also conform to Sections 1805.2.1 through 1805.2.3~~

~~**1805.2.1 Frost protection.** Except where otherwise protected from frost, foundation walls, piers and other permanent supports of buildings and structures shall be protected by one or more of the following methods:~~

~~1. Extending below the frost line of the locality;~~
~~2. Constructing in accordance with ASCE 32; or~~
~~3. Erecting on solid rock.~~

~~**Exception:** Free-standing buildings meeting all of the following conditions shall not be required to be protected:~~

~~1. Classified in Occupancy Category I, in accordance with Section1604.5;~~
~~2. Area of 600 square feet (56 m_2) or less for light-frame construction or 400 square feet (37m^2) or less for other than light-frame construction; and~~
~~3. Eave height of 10 feet (3048 mm) or less. Footings shall not bear on frozen soil unless such frozen condition is of a permanent character.~~

~~**1805.2.2 Isolated footings.** Footings on granular soil shall be so located that the line drawn between the lower edges of adjoining footings shall not have a slope steeper than 30 degrees (0.52 rad) with the horizontal, unless the material supporting the higher footing is braced or retained or otherwise laterally supported in an approved manner or a greater slope has been properly established by engineering analysis.~~

~~**1805.2.3 Shifting or moving soils.** Where it is known that the shallow subsoils are of a shifting or moving character, footings shall be carried to a sufficient depth to ensure stability.~~

~~**1805.3 Footings on or adjacent to slopes.** The placement of buildings and structures on or adjacent to slopes steeper than one unit vertical in three units horizontal (33.3-percent slope) shall conform to Sections 1805.3.1 through 1805.3.5~~

~~**1805.3.1 Building clearance from ascending slopes.** In general, buildings below slopes shall be set a sufficient distance from the slope to provide protection from slope drainage, erosion and shallow failures. Except as provided in Section 1805.3.5 and Figure 1805.3.1 the following criteria will be assumed to provide this protection. Where the existing slope is steeper than one unit vertical in one unit horizontal (100-percent slope), the toe of the slope shall be assumed to be at the intersection of a horizontal plane drawn from the top of the foundation and a plane drawn tangent to the slope at an angle of 45 degrees (0.79 rad) to the horizontal. Where a retaining wall is constructed at the toe of the slope, the height of the slope shall be measured from the top of the wall to the top of the slope.~~

~~**1805.3.2 Footing setback from descending slope surface.** Footings on or adjacent to slope surfaces shall be founded in firm material with an embedment and set back from the slope surface sufficient to provide vertical and lateral support for the footing without detrimental settlement. Except as provided for in Section 1805.3.5 and Figure 1805.3.1, the following setback is deemed adequate to meet the criteria. Where the slope is steeper than 1 unit vertical in 1 unit horizontal (100-percent slope), the required setback shall be measured from an imaginary plane 45 degrees (0.79 rad) to the horizontal, projected upward from the toe of the slope.~~

~~**1805.3.3 Pools.** The setback between pools regulated by this code and slopes shall be equal to one-half the building footing setback distance required by this section. That portion of the pool wall within a horizontal distance of 7 feet (2134 mm) from the top of the slope shall be capable of supporting the water in the pool without soil support.~~

~~**1805.3.4 Foundation elevation.** On graded sites, the top of any exterior foundation shall extend above the elevation of the street gutter at point of discharge or the inlet of an approved drainage device a minimum of 12 inches (305 mm) plus 2 percent. Alternate elevations are permitted subject to the approval of the building official, provided it can be demonstrated that required drainage to the point of discharge and away from the structure is provided at all locations on the site.~~

~~**1805.3.5 Alternate setback and clearance.** Alternate setbacks and clearances are permitted, subject to the approval of the building official. The building official is permitted to require an investigation and recommendation of a registered design professional to demonstrate that the intent of this section has been satisfied. Such an investigation shall include consideration of material, height of slope, slope gradient, load intensity and erosion characteristics of slope material.~~

~~**1805.4 Footings.** Footings shall be designed and constructed in accordance with Sections 1805.4.1 through 1805.4.6.~~

~~**1805.4.1 Design.** Footings shall be so designed that the allowable bearing capacity of the soil is not exceeded, and that differential settlement is minimized. The minimum width of footings shall be 12 inches (305 mm). Footings in areas with expansive soils shall be designed in accordance with the provisions of Section 1805.8.~~

~~**1805.4.1.1 Design loads.** Footings shall be designed for the most unfavorable effects due to the combinations of loads specified in Section 1605.2 or 1605.3. The dead load is permitted to include the weight of foundations, footings and overlying fill. Reduced live loads, as specified n Sections 1607.9 and 1607.11, are permitted to be used in the design of footings.~~

~~**1805.4.1.2 Vibratory loads.** Where machinery operations or other vibrations are transmitted through the foundation, consideration shall be given in the footing design to prevent detrimental disturbances of the soil.~~

~~**1805.4.2 Concrete footings.** The design, materials and construction of concrete footings shall comply with Sections 1805.4.2.1 through 1805.4.2.6 and the provisions of Chapter 19.~~

> ~~**Exception:** Where a specific design is not provided, concrete footings supporting walls of light-frame construction are permitted to be designed in accordance with Table 1805.4.2.~~

~~**TABLE 1805.4.2**~~
~~**FOOTINGS SUPPORTING WALLS OF LIGHT-FRAME CONSTRUCTION** [a, b, c, d, e]~~

~~NUMBER OF FLOORS SUPPORTED BY THE FOOTING~~ [f]	~~WIDTH OF FOOTING (inches)~~	~~THICKNESS OF FOOTING (inches)~~
~~1~~	~~12~~	~~6~~
~~2~~	~~15~~	~~6~~
~~3~~	~~18~~	~~8~~ [g]

~~For SI: 1 inch = 25.4 mm, 1 foot = 304.8 mm.~~
~~a. Depth of footings shall be in accordance with Section 1805.2.~~
~~b. The ground under the floor is permitted to be excavated to the elevation of the top of the footing.~~
~~c. Interior-stud-bearing walls are permitted to be supported by isolated footings. The footing width and length shall be twice the width shown in this table, and footings shall be spaced not more than 6 feet on center.~~
~~d. See Section 1908 for additional requirements for footings of structures assigned to Seismic Design Category C, D, E or F.~~
~~e. For thickness of foundation walls, see Section 1805.5.~~
~~f. Footings are permitted to support a roof in addition to the stipulated number of floors. Footings supporting roof only shall be as required for supporting one floor.~~
~~g. Plain concrete footings for Group R-3 occupancies are permitted to be 6 inches thick.~~

~~**1805.4.2.1 Concrete strength.** Concrete in footings shall have a specified compressive strength (f'_c) of not less than 2,500 pounds per square inch (psi) (17 237 kPa) at 28 days.~~

~~**1805.4.2.2 Footing seismic ties.** Where a structure is assigned to Seismic Design Category D, E or F in accordance with Section 1613, individual spread footings founded on soil defined in Section 1613.5.2 as Site Class E or F shall be interconnected by ties. Ties shall be capable of carrying, in tension or compression, a force equal to the product of the larger footing load times the seismic coefficient, S_{DS}, divided by 10 unless it is demonstrated that equivalent restraint is provided by reinforced concrete beams within slabs on grade or reinforced concrete slabs on grade.~~

~~**1805.4.2.3 Plain concrete footings.** The edge thickness of plain concrete footings supporting walls of other than light-frame construction shall not be less than 8 inches (203 mm) where placed on soil.~~

> ~~**Exception:** For plain concrete footings supporting Group R-3 occupancies, the edge thickness is permitted to be 6 inches (152 mm), provided that the footing does not extend beyond a distance greater than the thickness of the footing on either side of the supported wall.~~

~~**1805.4.2.4 Placement of concrete.** Concrete footings shall not be placed through water unless a tremie or other method approved by the building official is used. Where placed under or in the presence of water, the concrete shall be deposited by approved means to ensure minimum segregation of the mix and negligible turbulence of the water.~~

~~**1805.4.2.5 Protection of concrete.** Concrete footings shall be protected from freezing during depositing and for a period of not less than five days thereafter. Water shall not be allowed to flow through the deposited concrete.~~

~~**1805.4.2.6 Forming of concrete.** Concrete footings are permitted to be cast against the earth where, in the opinion of the building official, soil conditions do not require forming. Where forming is required, it shall be in accordance with Chapter 6 of ACI 318.~~

~~**1805.4.3 Masonry-unit footings.** The design, materials and construction of masonry-unit footings shall comply with Sections 1805.4.3.1 and 1805.4.3.2, and the provisions of Chapter 21.~~

> ~~**Exception:** Where a specific design is not provided, masonry-unit footings supporting walls of light-frame construction are permitted to be designed in accordance with Table 1805.4.2.~~

~~**1805.4.3.1 Dimensions.** Masonry-unit footings shall be laid in Type M or S mortar complying with Section 2103.8 and the depth shall not be less than twice the projection beyond the wall, pier or column. The width shall not be less than 8 inches (203 mm) wider than the wall supported thereon.~~

~~**1805.4.3.2 Offsets.** The maximum offset of each course in brick foundation walls stepped up from the footings shall be 1.5 inches (38 mm) where laid in single courses, and 3 inches (76 mm) where laid in double courses.~~

~~**1805.4.4 Steel grillage footings.** Grillage footings of structural steel shapes shall be separated with approved steel spacers and be entirely encased in concrete with at least 6 inches (152 mm) on the bottom and at least 4 inches (102 mm) at all other points. The spaces between the shapes shall be completely filled with concrete or cement grout.~~

~~**1805.4.5 Timber footings.** Timber footings are permitted for buildings of Type V construction and as otherwise approved by the building official. Such footings shall be treated in accordance with AWPA U1 (Commodity Specification A, Use Category 4B). Treated timbers are not required where placed entirely below permanent water level or where used as capping for wood piles that project above the water level over submerged or marsh lands. The compressive stresses perpendicular to the grain in untreated timber footings supported upon treated piles shall not exceed 70 percent of the allowable stresses for the species and grade of timber as specified in the AF&PA NDS.~~

(Renumber subsequent sections)

~~**1805.5.7 (Supp) Pier and curtain wall foundations.** Except in Seismic Design Categories D, E and F, pier and curtain wall foundations are permitted to be used to support light-frame construction not more than two stories above grade plane, provided the following requirements are met:~~

1. ~~All load-bearing walls shall be placed on continuous concrete footings bonded integrally with the exterior wall footings.~~
2. ~~The minimum actual thickness of a load-bearing masonry wall shall not be less than 4 inches (102 mm) nominal or 3 5/8 inches (92 mm) actual thickness, and shall be bonded integrally with piers spaced 6 feet (1829 mm) on center (o.c.).~~
3. ~~Piers shall be constructed in accordance with Chapter 21 and the following:~~
   - ~~3.1. The unsupported height of the masonry piers shall not exceed 10 times their least dimension.~~
   - ~~3.2. Where structural clay tile or hollow concrete masonry units are used for piers supporting beams and girders, the cellular spaces shall be filled solidly with concrete or Type M or S mortar.~~

     ~~**Exception:** Unfilled hollow piers are permitted where the unsupported height of the pier is not more than four times its least dimension.~~

   - ~~3.3. Hollow piers shall be capped with 4 inches (102 mm) of solid masonry or concrete or the cavities of the top course shall be filled with concrete or grout.~~
4. ~~The maximum height of a 4-inch (102mm) load-bearing masonry foundation wall supporting wood frame walls and floors shall not be more than 4 feet (1219 mm) in height.~~
5. ~~The unbalanced fill for 4-inch (102 mm) foundation walls shall not exceed 24 inches (610 mm) for solid masonry, nor 12 inches (305mm) for hollow masonry.~~

(Renumber subsequent sections)

~~**1805.8 Design for expansive soils.** Footings or foundations for buildings and structures founded on expansive soils shall be designed in accordance with Section 1805.8.1 or 1805.8.2. Footing or foundation design need not comply with Section 1805.8.1 or 1805.8.2 where the soil is removed in accordance with Section 1805.8.3, nor where the building official approves stabilization of the soil in accordance with Section 1805.8.4.~~

~~**1805.8.1 Foundations.** Footings or foundations placed on or within the active zone of expansive soils shall be designed to resist differential volume changes and to prevent structural damage to the supported structure. Deflection and racking of the supported structure shall be limited to that which will not interfere with the usability and serviceability of the structure. Foundations placed below where volume change occurs or below expansive soil shall comply with the following provisions:~~

1. ~~Foundations extending into or penetrating expansive soils shall be designed to prevent uplift of the supported structure.~~
2. ~~Foundations penetrating expansive soils shall be designed to resist forces exerted on the foundation due to soil volume changes or shall be isolated from the expansive soil.~~

~~**1805.8.2 Slab-on-ground foundations.** Moments, shears and deflections for use in designing slab-on-ground, mat or raft foundations on expansive soils shall be determined in accordance with *WRI/CRSI Design of Slab-on-Ground Foundations* or *PTI Standard Requirements for Analysis of Shallow Concrete Foundations on Expansive Soils.* Using the moments, shears and deflections determined above, nonprestressed slabs-on-ground, mat or raft foundations on expansive soils shall be designed in accordance with *WRI/CRSI Design of Slab-on-Ground Foundations* and post-tensioned slab-on-ground, mat or raft foundations on expansive soils shall be designed in accordance with *PTI Standard Requirements for Design of Shallow Post-Tensioned Concrete Foundations on Expansive Soils*. It shall be permitted to analyze and design such slabs by other methods that account for soil-structure interaction, the deformed shape of the soil support, the plate or stiffened plate action of the slab as well as both center lift and edge lift conditions. Such alternative methods shall be rational and the basis for all aspects and parameters of the method shall be available for peer review.~~

~~**1805.8.3 Removal of expansive soil.** Where expansive soil is removed in lieu of designing footings or foundations in accordance with Section 1805.8.1 or 1805.8.2, the soil shall be removed to a depth sufficient to ensure a constant moisture content in the remaining soil. Fill material shall not contain expansive soils and shall comply with Section 1803.5 or 1803.6.~~

> ~~**Exception:** Expansive soil need not be removed to the depth of constant moisture, provided the confining pressure in the expansive soil created by the fill and supported structure exceeds the swell pressure.~~

~~**1805.8.4 Stabilization.** Where the active zone of expansive soils is stabilized in lieu of designing footings or foundations in accordance with Section 1805.8.1 or 1805.8.2, the soil shall be stabilized by chemical, dewatering, presaturation or equivalent techniques.~~

~~**1805.9 Seismic requirements.** See Section 1908 for additional requirements for footings and foundations of structures assigned to Seismic Design Category C, D, E or F. For structures assigned to Seismic Design Category D, E or F, provisions of ACI 318, Sections 21.10.1 to 21.10.3, shall apply when not in conflict with the provisions of Section 1805. Concrete shall have a specified compressive strength of not less than 3,000 psi (20.68 MPa) at 28 days.~~

> ~~**Exceptions:**~~
>
> 1. ~~Group R or U occupancies of light-frame construction and two stories or less in height are permitted to use concrete with a specified compressive strength of not less than 2,500 psi (17.2 MPa) at 28 days.~~
> 2. ~~Detached one- and two-family dwellings of light-frame construction and two stories or less in height are not required to comply with the provisions of ACI 318, Sections 21.10.1 to 21.10.3.~~

3. Add new text as follows:

<u>**SECTION 1808**</u>
<u>**FOUNDATIONS**</u>

<u>**1808.1 General.** Foundations shall be designed and constructed in accordance with Sections 1808.2 through 1808.9. Shallow foundations shall also satisfy the requirements of Section 1809. Deep foundations shall also satisfy the requirements of Section 1810.</u>

<u>**1808.2 Design for capacity and settlement.** Foundations shall be so designed that the allowable bearing capacity of the soil is not exceeded, and that differential settlement is minimized. Foundations in areas with expansive soils shall be designed in accordance with the provisions of Section 1808.6</u>

<u>**1808.3 Design loads.** Foundations shall be designed for the most unfavorable effects due to the combinations of loads specified in Section 1605.2 or 1605.3. The dead load is permitted to include the weight of foundations and overlying fill. Reduced live loads, as specified in Sections 1607.9 and 1607.11, shall be permitted to be used in the design of foundations.</u>

<u>**1808.3.1 Seismic overturning.** Where foundations are proportioned using the load combinations of Section 1605.2, and the computation of seismic overturning effects is by Equivalent Lateral Force Analysis or Modal Analysis, the proportioning shall be in accordance with Section 12.13.4 of ASCE 7.</u>

<u>**1808.4 Vibratory loads.** Where machinery operations or other vibrations are transmitted through the foundation, consideration shall be given in the foundation design to prevent detrimental disturbances of the soil.</u>

1808.5 Shifting or moving soils. Where it is known that the shallow subsoils are of a shifting or moving character, foundations shall be carried to a sufficient depth to ensure stability.

1808.6 Design for expansive soils. Foundations for buildings and structures founded on expansive soils shall be designed in accordance with Section 1808.6.1 or 1808.6.2.

> **Exception:** Foundation design need not comply with Section 1808.6.1 or 1808.6.2 where one of the following conditions is satisfied:
>
> 1. The soil is removed in accordance with Section 1808.6.3; or
> 2. The building official approves stabilization of the soil in accordance with Section 1808.6.4.

1808.6.1 Foundations. Foundations placed on or within the active zone of expansive soils shall be designed to resist differential volume changes and to prevent structural damage to the supported structure. Deflection and racking of the supported structure shall be limited to that which will not interfere with the usability and serviceability of the structure.

Foundations placed below where volume change occurs or below expansive soil shall comply with the following provisions:

1. Foundations extending into or penetrating expansive soils shall be designed to prevent uplift of the supported structure.
2. Foundations penetrating expansive soils shall be designed to resist forces exerted on the foundation due to soil volume changes or shall be isolated from the expansive soil.

1808.6.2 Slab-on-ground foundations. Moments, shears and deflections for use in designing slab-on-ground, mat or raft foundations on expansive soils shall be determined in accordance with *WRI/CRSI Design of Slab-on-ground Foundations* or *PTI Standard Requirements for Analysis of Shallow Concrete Foundations on Expansive Soils*. Using the moments, shears and deflections determined above, nonprestressed slabs-on-ground, mat or raft foundations on expansive soils shall be designed in accordance with *WRI/CRSI Design of Slab-on-ground Foundations* and post-tensioned slab-on-ground, mat or raft foundations on expansive soils shall be designed in accordance with *PTI Standard Requirements for Design of Shallow Post-tensioned Concrete Foundations on Expansive Soils*. It shall be permitted to analyze and design such slabs by other methods that account for soil-structure interaction, the deformed shape of the soil support, the plate or stiffened plate action of the slab, as well as both center lift and edge lift conditions. Such alternative methods shall be rational and the basis for all aspects and parameters of the method shall be available for peer review.

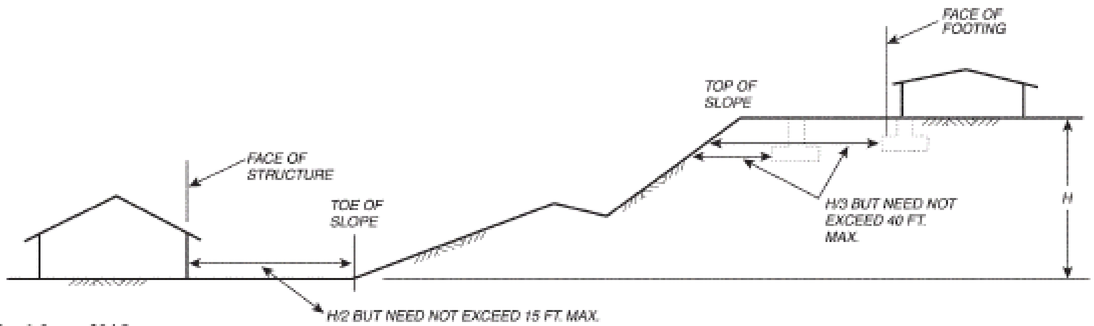
1808.6.3 Removal of expansive soil. Where expansive soil is removed in lieu of designing foundations in accordance with Section 1808.6.1 or 1808.6.2 the soil shall be removed to a depth sufficient to ensure a constant moisture content in the remaining soil. Fill material shall not contain expansive soils and shall comply with Section 1803.5 or 1803.6.

> **Exception:** Expansive soil need not be removed to the depth of constant moisture, provided the confining pressure in the expansive soil created by the fill and supported structure exceeds the swell pressure.

1808.6.4 Stabilization. Where the active zone of expansive soils is stabilized in lieu of designing foundations in accordance with Section 1808.6.1 or 1808.6.2, the soil shall be stabilized by chemical, dewatering, presaturation or equivalent techniques.

1808.7 Foundations on or adjacent to slopes. The placement of buildings and structures on or adjacent to slopes steeper than one unit vertical in three units horizontal (33.3-percent slope) shall comply with Sections 1808.7.1 through 1808.7.5.

1808.7.1 Building clearance from ascending slopes. In general, buildings below slopes shall be set a sufficient distance from the slope to provide protection from slope drainage, erosion and shallow failures. Except as provided in Section 1808.7.5 and Figure 1808.7.1 the following criteria will be assumed to provide this protection. Where the existing slope is steeper than one unit vertical in one unit horizontal (100-percent slope), the toe of the slope shall be assumed to be at the intersection of a horizontal plane drawn from the top of the foundation and a plane drawn tangent to the slope at an angle of 45 degrees (0.79 rad) to the horizontal. Where a retaining wall is constructed at the toe of the slope, the height of the slope shall be measured from the top of the wall to the top of the slope.



For SI: 1 foot = 304.8 mm.

FIGURE 1808.7.1
FOUNDATION CLEARANCES FROM SLOPES

1808.7.2 Foundation setback from descending slope surface. Foundations on or adjacent to slope surfaces shall be founded in firm material with an embedment and set back from the slope surface sufficient to provide vertical and lateral support for the foundation without detrimental settlement. Except as provided for in Section 1808.7.5 and Figure 1808.7.1 the following setback is deemed adequate to meet the criteria. Where the slope is steeper than 1 unit vertical in 1 unit horizontal (100-percent slope), the required setback shall be measured from an imaginary plane 45 degrees (0.79 rad) to the horizontal, projected upward from the toe of the slope.

1808.7.3 Pools. The setback between pools regulated by this code and slopes shall be equal to one-half the building footing setback distance required by this section. That portion of the pool wall within a horizontal distance of 7 feet (2134 mm) from the top of the slope shall be capable of supporting the water in the pool without soil support.

1808.7.4 Foundation elevation. On graded sites, the top of any exterior foundation shall extend above the elevation of the street gutter at point of discharge or the inlet of an approved drainage device a minimum of 12 inches (305 mm) plus 2 percent. Alternate elevations are permitted subject to the approval of the building official, provided it can be demonstrated that required drainage to the point of discharge and away from the structure is provided at all locations on the site.

1808.7.5 Alternate setback and clearance. Alternate setbacks and clearances are permitted, subject to the approval of the building official. The building official is permitted to require an investigation and recommendation of a registered design professional to demonstrate that the intent of this section has been satisfied. Such an investigation shall include consideration of material, height of slope, slope gradient, load intensity and erosion characteristics of slope material.

1808.8 Concrete foundations. The design, materials and construction of concrete foundations shall comply with Sections 1808.8.1 through 1808.8.6 and the provisions of Chapter 19.

> **Exception:** Where concrete footings supporting walls of light-frame construction are designed in accordance with Table 1809.7, a specific design in accordance with Chapter 19 is not required.

1808.8.1 Concrete or grout strength and mix proportioning. Concrete or grout in foundations shall have a specified compressive strength (*f ′c*) not less than the largest applicable value indicated in Table 1808.8.1. Where concrete is placed through a funnel hopper at the top of a deep foundation element the concrete mix shall be designed and proportioned so as to produce a cohesive workable mix having a slump of not less than 4 inches (102 mm) and not more than 6 inches (152 mm). Where concrete or grout is to be pumped, the mix design including slump shall be adjusted to produce a pumpable mixture.

TABLE 1808.8.1
MINIMUM SPECIFIED COMPRESSIVE STRENGTH, f'_c, OF CONCRETE OR GROUT

CONDITION	SPECIFIED COMPRESSIVE STRENGTH, $f'c$
1. Foundations for structures assigned to Seismic Design Category A, B, or C	2,500 psi (17.24 MPa)
2a. Foundations for Group R or U occupancies of light-framed construction, two stories or less in height, assigned to Seismic Design Category D, E, or F	2,500 psi (17.24 MPa)
2b. Foundations for other structures assigned to Seismic Design Category D, E, or F	3,000 psi (20.68 MPa)
3. Precast nonprestressed driven piles	3,000 psi (20.68 MPa)
5. Socketed drilled shafts	4,000 psi (27.58 MPa)
6. Micropiles	4,000 psi (27.58 MPa)
7. Precast prestressed driven piles	5,000 psi (34.48 MPa)

1808.8.2 Concrete cover. The concrete cover provided for prestressed and nonprestressed reinforcement in foundations shall be no less than that specified in Table 1808.8.2. Concrete cover shall be measured from the concrete surface to the outermost surface of the steel to which the cover requirement applies. Where concrete is placed in a temporary or permanent casing or a mandrel, the inside face of the casing or mandrel shall be considered the concrete surface.

TABLE 1808.8.2
MINIMUM CONCRETE COVER

FOUNDATION ELEMENT OR CONDITION	MINIMUM COVER
1. Shallow foundations	In accordance with Section 7.7 of ACI 318
2. Precast nonprestressed deep foundation elements [a] Exposed to seawater Not manufactured under plant conditions Manufactured under plant control conditions	 3 inches (76 mm) 2 inches (51 mm) In accordance with Section 7.7.3 of ACI 318
3. Precast prestressed deep foundation elements Exposed to seawater Other	 2.5 inches (64 mm) In accordance with Section 7.7.3 of ACI 318
4. Cast-in-place deep foundation elements not enclosed by a steel pipe, tube, or permanent casing	2.5 inches (64 mm)
5. Cast-in-place deep foundation elements enclosed by a steel pipe, tube, or permanent casing	1 inch (25 mm)
6. Structural steel core within a steel pipe, tube, or permanent casing	2 inches (51 mm)

a. Longitudinal bars spaced less than 1.5 inches (38 mm) clear distance apart shall be considered bundled bars for which the minimum concrete cover shall be equal to that for the equivalent diameter of the bundled bars.

1808.8.3 Placement of concrete. Concrete shall be placed in such a manner as to ensure the exclusion of any foreign matter and to secure a full-sized foundation. Concrete shall not be placed through water unless a tremie or other method approved by the building official is used. Where placed under or in the presence of water, the concrete shall be deposited by approved means to ensure minimum segregation of the mix and negligible turbulence of the water. Where depositing concrete from the top of a deep foundation element, the concrete shall be chuted directly into smooth-sided pipes or tubes or poured in a rapid and continuous operation through a funnel hopper centered at the top of the element.

1808.8.4 Protection of concrete. Concrete foundations shall be protected from freezing during depositing and for a period of not less than five days thereafter. Water shall not be allowed to flow through the deposited concrete.

1808.8.5 Forming of concrete. Concrete foundations are permitted to be cast against the earth where, in the opinion of the building official, soil conditions do not require form work. Where form work is required, it shall be in accordance with Chapter 6 of ACI 318.

1808.8.6 Seismic requirements. See Section 1908 for additional requirements for foundations of structures assigned to Seismic Design Category C, D, E or F.

For structures assigned to Seismic Design Category D, E or F, provisions of ACI 318, Sections 21.10.1 through 21.10.4, shall apply where not in conflict with the provisions of Sections 1808 through 1810

Exceptions:

1. Detached one- and two-family dwellings of light-frame construction and two stories or less above grade plane are not required to comply with the provisions of ACI 318, Sections 21.10.1 through ~~21.10.3~~ 21.10.4.
2. Section 21.10.4.4(a) of ACI 318 shall not apply.

1808.9 Vertical masonry foundation elements. Vertical masonry foundation elements that are not foundation piers as defined in Section 2102.1 shall be designed as piers, walls, or columns, as applicable, in accordance with ACI 530/ASCE 5/TMS 402.

1809
SHALLOW FOUNDATIONS

1809.1 General. Shallow foundations shall be designed and constructed in accordance with Sections 1809.2 through 1809.13.

1809.2 Supporting soils. Shallow foundations shall be built on undisturbed soil, compacted fill material or controlled low-strength material (CLSM). Compacted fill material shall be placed in accordance with Section 1803.5. CLSM shall be placed in accordance with Section 1803.6.

1809.3 Stepped footings. The top surface of footings shall be level. The bottom surface of footings shall be permitted to have a slope not exceeding one unit vertical in 10 units horizontal (10-percent slope). Footings shall be stepped where it is necessary to change the elevation of the top surface of the footing or where the surface of the ground slopes more than one unit vertical in 10 units horizontal (10-percent slope).

1809.4 Depth and width of footings. The minimum depth of footings below the undisturbed ground surface shall be 12 inches (305 mm). Where applicable, the requirements of Section 1809.5 shall also be satisfied. The minimum width of footings shall be 12 inches (305 mm).

1809.5 Frost protection. Except where otherwise protected from frost, foundation walls and other permanent supports of buildings and structures shall be protected from frost by one or more of the following methods:

1. Extending below the frost line of the locality;
2. Constructing in accordance with ASCE-32; or
3. Erecting on solid rock.

Exception: Free-standing buildings meeting all of the following conditions shall not be required to be protected:

1. Assigned to Occupancy Category I, in accordance with Section 1604.5;
2. Area of 600 square feet (56 m^2) or less for light-frame construction or 400 square feet (37 m^2) or less for other than light-frame construction; and
3. Eave height of 10 feet (3048 mm) or less.

Shallow foundations shall not bear on frozen soil unless such frozen condition is of a permanent character.

1809.6 Location of footings. Footings on granular soil shall be so located that the line drawn between the lower edges of adjoining footings shall not have a slope steeper than 30 degrees (0.52 rad) with the horizontal, unless the material supporting the higher footing is braced or retained or otherwise laterally supported in an approved manner or a greater slope has been properly established by engineering analysis.

1809.7 Prescriptive footings for light-frame construction. Where a specific design is not provided, concrete or masonry-unit footings supporting walls of light-frame construction shall be permitted to be designed in accordance with Table 1809.7.

TABLE 1809.7
PRESCRIPTIVE FOOTINGS SUPPORTING WALLS OF LIGHT-FRAME CONSTRUCTION [a, b, c, d, e]

NUMBER OF FLOORS SUPPORTED BY THE FOOTING [f]	WIDTH OF FOOTING (inches)	THICKNESS OF FOOTING (inches)
1	12	6
2	15	6
3	18	8 [g]

For SI: 1 inch = 25.4 mm, 1 foot = 304.8 mm.

a. Depth of footings shall be in accordance with Section 1809.4
b. The ground under the floor ~~is~~ shall be permitted to be excavated to the elevation of the top of the footing.
c. Interior-stud-bearing walls shall be permitted to be supported by isolated footings. The footing width and length shall be twice the width shown in this table, and footings shall be spaced not more than 6 feet on center.
d. See Section 1908 for additional requirements for concrete footings of structures assigned to Seismic Design Category C, D, E or F.
e. For thickness of foundation walls, see Section 1805.5.
f. Footings shall be permitted to support a roof in addition to the stipulated number of floors. Footings supporting roof only shall be as required for supporting one floor.
g. Plain concrete footings for Group R-3 occupancies shall be permitted to be 6 inches thick.

1809.8 Plain concrete footings. The edge thickness of plain concrete footings supporting walls of other than light-frame construction shall not be less than 8 inches (203 mm) where placed on soil.

> **Exception:** For plain concrete footings supporting Group R-3 occupancies, the edge thickness is permitted to be 6 inches (152 mm), provided that the footing does not extend beyond a distance greater than the thickness of the footing on either side of the supported wall.

1809.9 Masonry-unit footings. The design, materials and construction of masonry-unit footings shall comply with Sections 1809.9.1 and 1809.9.2 and the provisions of Chapter 21.

> **Exception:** Where a specific design is not provided, masonry-unit footings supporting walls of light-frame construction shall be permitted to be designed in accordance with Table 1809.7.

1809.9.1 Dimensions. Masonry-unit footings shall be laid in Type M or S mortar complying with Section 2103.8 and the depth shall not be less than twice the projection beyond the wall, pier or column. The width shall not be less than 8 inches (203 mm) wider than the wall supported thereon.

1809.9.2 Offsets. The maximum offset of each course in brick foundation walls stepped up from the footings shall be 1.5 inches (38 mm) where laid in single courses, and 3 inches (76 mm) where laid in double courses.

1809.10 Pier and curtain wall foundations. Except in Seismic Design Categories D, E and F, pier and curtain wall foundations shall be permitted to be used to support light-frame construction not more than two stories above grade plane, provided the following requirements are met:

1. All load-bearing walls shall be placed on continuous concrete footings bonded integrally with the exterior wall footings.
2. The minimum actual thickness of a load-bearing masonry wall shall not be less than 4 inches (102 mm) nominal or 3.625 inches (92 mm) actual thickness, and shall be bonded integrally with piers spaced 6 feet (1829 mm) on center (o.c.).
3. Piers shall be constructed in accordance with Chapter 21 and the following:
   3.1. The unsupported height of the masonry piers shall not exceed 10 times their least dimension.
   3.2. Where structural clay tile or hollow concrete masonry units are used for piers supporting beams and girders, the cellular spaces shall be filled solidly with concrete or Type M or S mortar.

   Exception: Unfilled hollow piers shall be permitted where the unsupported height of the pier is not more than four times its least dimension.

   3.3. Hollow piers shall be capped with 4 inches (102 mm) of solid masonry or concrete or the cavities of the top course shall be filled with concrete or grout.

4. The maximum height of a 4-inch (102mm)load-bearing masonry foundation wall supporting wood frame walls and floors shall not be more than 4 feet (1219 mm) in height.
5. The unbalanced fill for 4-inch (102 mm) foundation walls shall not exceed 24 inches (610 mm) for solid masonry, nor 12 inches (305mm)for hollow masonry.

1809.11 Steel grillage footings. Grillage footings of structural steel shapes shall be separated with approved steel spacers and be entirely encased in concrete with at least 6 inches (152 mm) on the bottom and at least 4 inches (102 mm) at all other points. The spaces between the shapes shall be completely filled with concrete or cement grout.

1809.12 Timber footings. Timber footings shall be permitted for buildings of Type V construction and as otherwise approved by the building official. Such footings shall be treated in accordance with AWPA U1 (Commodity Specification A, Use Category 4B). Treated timbers are not required where placed entirely below permanent water level, or where used as capping for wood piles that project above the water level over submerged or marsh lands. The compressive stresses perpendicular to grain in untreated timber footings supported upon treated piles shall not exceed 70 percent of the allowable stresses for the species and grade of timber as specified in the AF&PA NDS.

1809.13 Footing seismic ties. Where a structure is assigned to Seismic Design Category D, E or F in accordance with Section 1613, individual spread footings founded on soil defined in Section 1613.5.2 as Site Class E or F shall be interconnected by ties. Ties shall be capable of carrying, in tension or compression, a force equal to the product of the larger footing load times the seismic coefficient S_{DS} divided by 10 unless it is demonstrated that equivalent restraint is provided by reinforced concrete beams within slabs on grade or reinforced concrete slabs on grade.

4. Delete without substitution:

~~**1809.2.2.1 Materials.** Concrete shall have a 28-day specified compressive strength (f'_c) of not less than 3,000 psi (20.68 MPa).~~

~~**1809.2.2.5 Concrete cover.** Reinforcement for piles that are not manufactured under plant conditions shall have a concrete cover of not less than 2 inches (51 mm). Reinforcement for piles manufactured under plant control conditions shall have a concrete cover of not less than 1.25 inches (32 mm) for No. 5 bars and smaller, and not less than 1.5 inches (38 mm) for No. 6 through No. 11 bars except that longitudinal bars spaced less than 1.5 inches (38 mm) clear distance apart shall be considered bundled bars for which the minimum concrete cover shall be equal to that for the equivalent diameter of the bundled bars. Reinforcement for piles exposed to seawater shall have a concrete cover of not less than 3 inches (76 mm).~~

~~**1809.2.3.5 Concrete cover.** Prestressing steel and pile reinforcement shall have a concrete cover of not less than $1^{1}/_{4}$ inches (32 mm) for square piles of 12 inches (305 mm) or smaller size and $1^{1}/_{2}$ inches (38 mm) for larger piles, except that for piles exposed to seawater, the minimum protective concrete cover shall not be less than $2^{1}/_{2}$ inches (64 mm).~~

~~**1810.1.1 Materials.** Concrete shall have a 28-day specified compressive strength (f'_c) of not less than 2,500 psi (17.24 MPa). Where concrete is placed through a funnel hopper at the top of the pile, the concrete mix shall be designed and proportioned so as to produce a cohesive workable mix having a slump of not less than 4 inches (102 mm) and not more than 6 inches (152 mm). Where concrete is to be pumped, the mix design including slump shall be adjusted to produce a pumpable concrete.~~

~~**1810.1.3 Concrete placement.** Concrete shall be placed in such a manner as to ensure the exclusion of any foreign matter and to secure a full-sized shaft. Concrete shall not be placed through water except where a tremie or other Approved method is used. When depositing concrete from the top of the pile, the concrete shall not be chuted directly into the pile but shall be poured in a rapid and continuous operation through a funnel hopper centered at the top of the pile.~~

~~**1810.2.5 Concrete cover.** The minimum concrete cover shall be $2^{1}/_{2}$ inches (64 mm) for uncased shafts and 1 inch (25 mm) for cased shafts.~~

~~**1810.4.4 Concrete cover.** Pile reinforcement shall have a concrete cover of not less than 2.5 inches (64 mm), measured from the inside face of the drive casing or mandrel.~~

~~**1810.6.5 Placing concrete.** The placement of concrete shall conform to Section 1810.1.3, but is permitted to be chuted directly into smooth-sided pipes and tubes without a centering funnel hopper.~~

~~**1812.3 Materials.** Concrete shall have a 28-day specified compressive strength (f'_c) of not less than 2,500 psi (17.24 MPa). Where concrete is placed through a funnel hopper at the top of the pier, the concrete mix shall be designed and proportioned so as to produce a cohesive workable mix having a slump of not less than 4 inches (102 mm) and not more than 6 inches (152 mm). Where concrete is to be pumped, the mix design including slump shall be adjusted to produce a pumpable concrete.~~

~~**1812.5 Concrete placement.** Concrete shall be placed in such a manner as to ensure the exclusion of any foreign matter and to secure a full-sized shaft. Concrete shall not be placed through water except where a tremie or other approved method is used. When depositing concrete from the top of the pier, the concrete shall not be chuted directly into the pier but shall be poured in a rapid and continuous operation through a funnel hopper centered at the top of the pier.~~

~~**1812.10 Dewatering.** Where piers are carried to depths below water level, the piers shall be constructed by a method that will provide accurate preparation and inspection of the bottom, and the depositing or construction of sound concrete or other masonry in the dry.~~

~~**1812.7 Masonry.** Where the unsupported height of foundation piers exceeds six times the least dimension, the allowable working stress on piers of unit masonry shall be reduced in accordance with ACI 530/ASCE 5/TMS 402.~~

5. Revise as follows:

1808.2.23.2 (Supp) Seismic Design Category D, E or F. Where a structure is assigned to Seismic Design Category D, E or F in accordance with Section 1613, the requirements for Seismic Design Category C given in Section 1808.2.23.1 shall be met, in addition to the following. ~~Provisions of ACI 318, Section 21.10.4, shall apply when not in conflict with the provisions of Sections 1808 through 1812. Concrete shall have a specified compressive strength of not less than 3,000 psi (20.68 MPa) at 28 days.~~

~~**Exceptions:**~~

1. ~~Group R or U occupancies of light-frame construction and two stories or less above grade plane are permitted to use concrete with a specified compressive strength of not less than 2,500 psi (17.2 MPa) at 28 days.~~
2. ~~Detached one- and two-family dwellings of light-frame construction and two stories or less in height are not required to comply with the provisions of ACI 318, Section 21.10.4.~~
3. ~~Section 21.10.4.4(a) of ACI 318 need not apply to concrete piles.~~

1809.2.3.1 Materials. Prestressing steel shall conform to ASTM A 416. ~~Concrete shall have a 28-day specified compressive strength (f'_c) of not less than 5,000 psi (34.48 MPa).~~

1810.3.4 Reinforcement. For piles installed with a hollow-stem auger where full-length longitudinal steel reinforcement is placed without lateral ties, the reinforcement shall be placed through the hollow stem of the auger prior to filling the pile with concrete. ~~All pile reinforcement shall have a concrete cover of not less than 2.5 inches (64 mm).~~

Exception: Where physical constraints do not allow the placement of the longitudinal reinforcement prior to filling the pile with concrete or where partial-length longitudinal reinforcement is placed without lateral ties, the reinforcement is allowed to be placed after the piles are completely concreted but while concrete is still in a semifluid state.

1810.5.4 Reinforcement. ~~Reinforcement shall not be placed within 1 inch (25 mm) of the steel shell.~~ Reinforcing shall be required for unsupported pile lengths or where the pile is designed to resist uplift or unbalanced lateral loads.

1810.6.4 Reinforcement. Reinforcement steel shall conform to Section 1810.1.2. ~~Reinforcement shall not be placed within 1 inch (25 mm) of the steel casing.~~

1810.7.2 Materials. Pipe and steel cores shall conform to the material requirements in Section 1809.3. Pipes shall have a minimum wall thickness of $^3/_8$ inch (9.5 mm) and shall be fitted with a suitable steel-driving shoe welded to the bottom of the pipe. ~~Concrete shall have a 28-day specified compressive strength ($f'c$) of not less than 4,000 psi (27.58 MPa). The concrete mix shall be designed and proportioned so as to produce a cohesive workable mix with a slump of 4 inches to 6 inches (102 mm to 152 mm).~~

1810.7.4 Structural core. The gross cross-sectional area of the structural steel core shall not exceed 25 percent of the gross area of the caisson. ~~The minimum clearance between the structural core and the pipe shall be 2 inches (51 mm).~~ Where cores are to be spliced, the ends shall be milled or ground to provide full contact and shall be full-depth welded.

1810.7.6 Installation. The rock socket and pile shall be thoroughly cleaned of foreign materials before filling with concrete. Steel cores shall be bedded in cement grout at the base of the rock socket. ~~Concrete shall not be placed through water except where a tremie or other approved method is used.~~

1810.8.2 (Supp) Materials. ~~Grout shall have a specified compressive strength (f'_c) of not less than 4,000 psi (27.58 Mpa). The grout mix shall be designed and proportioned so as to produce a pumpable mixture.~~ Reinforcement shall consist of deformed reinforcing bars in accordance with ASTM A 615 Grade 60 or Grade 75 or ASTM A 722 Grade 150.

The steel pipe shall have a minimum wall thickness of 3/16 inch (4.8 mm). Splices shall comply with Section 1808.2.7. The steel pipe shall have a minimum yield strength exceeding 45,000 p.s.i. (310 MPa) and a minimum elongation of 15 percent as shown by mill certifications or two coupon test samples per 40, 000 pounds (18 160 kg) of pipe.

Reason: Clarifies the scope of requirements related to design of all foundations and design of shallow foundations. Collects and unifies general requirements (for instance, related to concrete strength, concrete cover, and concrete placement) to reduce unnecessary repetition.

The revisions in new Section 1808.9 fix a conflict in the existing code. The present text refers to masonry foundation piers with an unsupported height that exceeds six times the least dimension. However, Section 2102.1 and ACI 530 define a masonry foundation pier as having a height less than or equal to 4 times its thickness. The revised text directs the reader to the pertinent definition and design requirements. Depending on the dimensions of a vertical masonry element, it is designed as a foundation pier, a pier, a wall, or a column.

Section 1812.10 is deleted because it conflicts with other requirements and unnecessarily restates other requirements. Section 1812.5 permits placement of concrete in water where proper methods are employed, so placement "in the dry" is not required. Requirements for inspection are already set forth in Chapter 17.

Correlation notes: In Section 1704.4 exception item 2.2, change "Table 1805.4.2" to "Table 1809.7". In Section 1704.4 exception item 4, change "Table 1805.5(5) to "Table 1807.1.6.2".

Bibliography:
Composite of Chapter 18 reorganization assuming all of proponent's proposals are approved.

Cost Impact: The code change proposal will not increase the cost of construction.

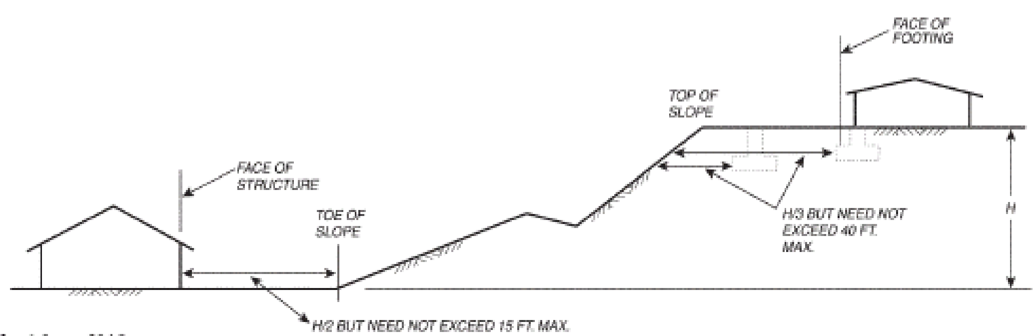
Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S151–07/08
Figure 1805.3.1

Proponent: Edwin T. Huston, Smith & Huston, Inc., representing the National Council of Structural Engineering Associations

Revise figure notation as follows:

AT LEAST THE SMALLER OF H/2 AND ~~BUT NEED NOT EXCEED~~ 15 FEET ~~FT. MAX~~
AT LEAST THE SMALLER OF H/3 AND ~~BUT NEED NOT EXCEED~~ 40 FEET ~~FT. MAX~~



For SI: 1 foot = 304.8 mm.

FIGURE 1805.3.1
FOUNDATION CLEARANCES FROM SLOPES

Reason: Code clarification. Editorial.

Cost Impact: The code change proposal will not increase the cost of construction.

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S152–07/08

1805.4.2.1, 1805.5.2.1, 1805.7.3, 1805.9, 1808.2.23.2, 1809.2.2.1, 1809.2.2.3, 1809.2.2.4, 1809.2.3.1, 1809.2.3.3, 1809.2.3.4, 1810.1.1, 1810.2.2, 1810.3.1, , 1810.4.1, 1810.5.2, 1810.5.2.3, 1810.6.2, 1810.7.2, 1812.3

Proponent: Philip Brazil, PE, SE, Reid Middleton, Inc., representing himself

Revise as follows:

1805.4.2.1 Concrete strength. Concrete in footings shall have a specified compressive strength (f'_c) of not less than 2,500 pounds per square inch (psi) (17 237 kPa)~~ at 28 days~~.

1805.5.2.1 Concrete foundation walls. Concrete foundation walls shall comply with the following:

1. The size and spacing of vertical reinforcement shown in Table 1805.5(5) is based on the use of reinforcement with a minimum yield strength of 60,000 psi (414 MPa). Vertical reinforcement with a minimum yield strength of 40,000 psi (276 MPa) or 50,000 psi (345 MPa) is permitted, provided the same size bar is used and the spacing shown in the table is reduced by multiplying the spacing by 0.67 or 0.83, respectively.
2. Vertical reinforcement, when required, shall be placed nearest the inside face of the wall a distance, *d,* from the outside face (soil side) of the wall. The distance, *d*, is equal to the wall thickness, *t*, minus 1.25 inches (32 mm) plus one-half the bar diameter, d_b [$d = t - (1.25 + d_b/2)$]. The reinforcement shall be placed within a tolerance of ± 3/8 inch (9.5 mm) where *d* is less than or equal to 8 inches (203 mm) or ± 1/2 inch (12.7 mm) where *d* is greater than 8 inches (203 mm).
3. In lieu of the reinforcement shown in Table 1805.5(5), smaller reinforcing bar sizes with closer spacings that provide an equivalent cross-sectional area of reinforcement per unit length of wall are permitted.
4. Concrete cover for reinforcement measured from the inside face of the wall shall not be less than 3/4 inch (19.1 mm). Concrete cover for reinforcement measured from the outside face of the wall shall not be less than 1.5 inches (38 mm) for No. 5 bars and smaller and not less than 2 inches (51 mm) for larger bars.
5. Concrete shall have a specified compressive strength, f'_c, of not less than 2,500 psi (17.2 MPa)~~ at 28 days~~.
6. The unfactored axial load per linear foot of wall shall not exceed 1.2 $t f'_c$, where *t* is the specified wall thickness in inches.

1805.7.3 Backfill. The backfill in the annular space around columns not embedded in poured footings shall be by one of the following methods:

1. Backfill shall be of concrete with ~~an ultimate~~ <u>a specified compressive</u> strength of 2,000 psi (13.8 MPa)~~ at 28 days~~. The hole shall not be less than 4 inches (102 mm) larger than the diameter of the column at its bottom or 4 inches (102 mm) larger than the diagonal dimension of a square or rectangular column.
2. Backfill shall be of clean sand. The sand shall be thoroughly compacted by tamping in layers not more than 8 inches (203 mm) in depth.
3. Backfill shall be of controlled low-strength material (CLSM).

1805.9 (Supp) Seismic requirements. See Section 1908 for additional requirements for footings and foundations of structures assigned to Seismic Design Category C, D, E or F.

For structures assigned to Seismic Design Category D, E or F, provisions of ACI 318, Sections 21.10.1 to 21.10.3, shall apply when not in conflict with the provisions of Section 1805. Concrete shall have a specified compressive strength of not less than 3,000 psi (20.68 MPa)~~ at 28 days~~.

Exceptions:

1. Group R or U occupancies of light-framed construction and two stories or less above grade plane are permitted to use concrete with a specified compressive strength of not less than 2,500 psi (17.2 MPa)~~ at 28 days~~.
2. Detached one- and two-family dwellings of light-frame construction and two stories or less above grade plane are not required to comply with the provisions of ACI 318, Sections 21.10.1 to 21.10.3.

1808.2.23.2 (Supp) Seismic Design Category D, E or F. Where a structure is assigned to Seismic Design Category D, E or F in accordance with Section 1613, the requirements for Seismic Design Category C given in Section 1808.2.23.1 shall be met, in addition to the following. Provisions of ACI 318, Section 21.10.4, shall apply when not in conflict with the provisions of Sections 1808 through 1812. Concrete shall have a specified compressive strength of not less than 3,000 psi (20.68 MPa)~~ at 28 days~~.

Exceptions:

1. Group R or U occupancies of light-frame construction and two stories or less above grade plane are permitted to use concrete with a specified compressive strength of not less than 2,500 psi (17.2 MPa)~~ at 28 days~~.
2. Detached one- and two-family dwellings of light-frame construction and two stories or less in height are not required to comply with the provisions of ACI 318, Section 21.10.4.
3. Section 21.10.4.4(a) of ACI 318 need not apply to concrete piles.

1809.2.2.1 Materials. Concrete shall have a ~~28-day~~ specified compressive strength (f'_c) of not less than 3,000 psi (20.68 MPa).

1809.2.2.3 Allowable stresses. The allowable compressive stress in the concrete shall not exceed 33 percent of the ~~28-day~~ specified compressive strength (f'_c) applied to the gross cross-sectional area of the pile. The allowable compressive stress in the reinforcing steel shall not exceed 40 percent of the yield strength of the steel (f_y) or a maximum of 30,000 psi (207 MPa). The allowable tensile stress in the reinforcing steel shall not exceed 50 percent of the yield strength of the steel (f_y) or a maximum of 24,000 psi (165 MPa).

1809.2.2.4 Installation. A precast concrete pile shall not be driven before the concrete has attained a compressive strength of at least 75 percent of the ~~28-day~~ specified compressive strength (f'_c), but not less than the strength sufficient to withstand handling and driving forces.

1809.2.3.1 Materials. Prestressing steel shall conform to ASTM A 416. Concrete shall have a ~~28-day~~ specified compressive strength (f'_c) of not less than 5,000 psi (34.48 MPa).

1809.2.3.3 (Supp) Allowable stresses. The allowable compressive stress, f_c, in concrete shall be determined as follows:

$f_c = 0.33\, f'_c - 0.27 f_{pc}$ **(Equation 18-10)**

where:

f'_c = The ~~28-day~~ specified compressive strength of the concrete.
f_{pc} = The effective prestress stress on the gross section.

1809.2.3.4 Installation. A prestressed pile shall not be driven before the concrete has attained a compressive strength of at least 75 percent of the ~~28-day~~ specified compressive strength (f'_c), but not less than the strength sufficient to withstand handling and driving forces.

1810.1.1 Materials. Concrete shall have a ~~28-day~~ specified compressive strength (f'_c) of not less than 2,500 psi (17.24 MPa). Where concrete is placed through a funnel hopper at the top of the pile, the concrete mix shall be designed and proportioned so as to produce a cohesive workable mix having a slump of not less than 4 inches (102 mm) and not more than 6 inches (152 mm). Where concrete is to be pumped, the mix design including slump shall be adjusted to produce a pumpable concrete.

1810.2.2 Allowable stresses. The maximum allowable design compressive stress for concrete not placed in a permanent steel casing shall be 25 percent of the ~~28-day~~ specified compressive strength (f'_c). Where the concrete is place in a permanent steel casing, the maximum allowable concrete stress shall be 33 percent of the ~~28-day~~ specified compressive strength (f'_c).

1810.3.1 Allowable stresses. The allowable design stress in the concrete of drilled or augered uncased piles shall not exceed 33 percent of the ~~28-day~~ specified compressive strength (f'_c). The allowable compressive stress of reinforcement shall not exceed 40 percent of the yield strength of the steel or 25,500 psi (175.8 MPa).

1810.4.1 Allowable stresses. The allowable design stress in the concrete shall not exceed 25 percent of the ~~28-day~~ specified compressive strength (f'_c) applied to a cross-sectional area not greater than the inside area of the drive casing or mandrel.

1810.5.2 Allowable stresses. The allowable design compressive stress in the concrete shall not exceed 33 percent of the ~~28-day~~ specified compressive strength (f'_c). The allowable concrete compressive stress shall be 0.40 (f'_c) for that portion of the pile meeting the conditions specified in Sections 1810.5.2.1 through 1810.5.2.4.

1810.5.2.3 Strength. The ratio of steel yield strength (f_y) to ~~28-day~~ specified compressive strength (f'_c) shall not be less than six.

1810.6.2 Allowable stresses. The allowable design compressive stress in the concrete shall not exceed 33 percent of the ~~28-day~~ specified compressive strength (f'_c). The allowable design compressive stress in the steel shall not exceed 35 percent of the minimum specified yield strength of the steel (F_y), provided F_y shall not be assumed greater than 36,000 psi (248 MPa) for computational purposes.

> **Exception:** Where justified in accordance with Section 1808.2.10, the allowable stresses are permitted to be increased to 0.50 F_y.

1810.7.2 Materials. Pipe and steel cores shall conform to the material requirements in Section 1809.3. Pipes shall have a minimum wall thickness of 3/8 inch (9.5 mm) and shall be fitted with a suitable steel-driving shoe welded to the bottom of the pipe. Concrete shall have a ~~28-day~~ specified compressive strength (f'_c) of not less than 4,000 psi (27.58 MPa). The concrete mix shall be designed and proportioned so as to produce a cohesive workable mix with a slump of 4 inches to 6 inches (102 mm to 152 mm).

1812.3 Materials. Concrete shall have a ~~28-day~~ specified compressive strength (f'_c) of not less than 2,500 psi (17.24 MPa). Where concrete is placed through a funnel hopper at the top of the pier, the concrete mix shall be designed and proportioned so as to produce a cohesive workable mix having a slump of not less than 4 inches (102 mm) and not more than 6 inches (152 mm). Where concrete is to be pumped, the mix design including slump shall be adjusted to produce a pumpable concrete.

Reason: The change is proposed for consistency with the provisions of ACI 318-05, which establishes the technical provisions for the determination of specified of compressive strength, including the time periods for strength tests of concrete samples. For example, Section 5.1.3 requires the specified compressive strength to be based on 28-day tests unless otherwise specified. Other test ages shall be specified in the design drawings or specifications. Section 5.6.2.4 requires a strength test to be the average of the strengths of two cylinder tests made from the same sample of concrete and tested at 28 days or at a test age designated for determination of the specified compressive strength. Requirements for setting the test age are already contained in ACI 318-05. Also specifying the test age in Chapter 18 of the IBC is redundant with respect to the commonly used test age of 28 days and in conflict with ACI 318 when a test age other than 28 days is preferred.

Cost Impact: The code change proposal will not increase the cost of construction.

Public Hearing:				
	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S153–07/08
1805.4.2.2, 1808.2.23.1

Proponent: Edwin T. Huston, Smith & Huston, Inc., National Council of Structural Engineering Associations

Revise as follows:

1805.4.2.2 Footing seismic ties. Where a structure is assigned to Seismic Design Category D, E or F in accordance with Section 1613, individual spread footings founded on soil defined in Section 1613.5.2 as Site Class E or F shall be interconnected by ties. Unless it is demonstrated that equivalent restraint is provided by reinforced concrete beams within slabs on grade or reinforced concrete slabs on grade, ~~T~~ties shall be capable of carrying, in tension or compression, a force equal to the lesser of the product of the larger footing design gravity load times the seismic coefficient, S_{DS}, divided by 10, and 25 percent of the smaller footing design gravity load. ~~unless it is demonstrated that equivalent restraint is provided by reinforced concrete beams within slabs on grade or reinforced concrete slabs on grade.~~

1808.2.23.1 (Supp) Seismic Design Category C. Where a structure is assigned to Seismic Design Category C in accordance with Section 1613, the following shall apply. Individual pile caps, piers or piles shall be interconnected by ties. Unless it can be demonstrated that equivalent restraint is provided by reinforced concrete beams within slabs on grade or reinforced concrete slabs on grade or confinement by competent rock, hard cohesive soils or very dense granular soils, ~~T~~ties shall be capable of carrying, in tension ~~and~~ or compression, a force equal to the lesser of the product of the larger pile cap or column design gravity load times the seismic coefficient, S_{DS}, divided by 10, and 25 percent of the smaller pile cap or column design gravity load ~~unless it can be demonstrated that equivalent restraint is provided by reinforced concrete beams within slabs on grade or reinforced concrete slabs on grade or confinement by competent rock, hard cohesive soils or very dense granular soils~~.

> **Exception:** In Groups R-3 and U occupancies of light-frame construction, pier foundations supporting foundation walls, isolated interior posts detailed so the pier is not subject to lateral loads, or exterior decks and patios, are not subject to interconnection if it can be shown the soils are of adequate stiffness, subject to the approval of the building official.

Reason: Code coordination, clarification, and relaxation of an overly restrictive code requirement. Changes "and" to "or" in Section 1808.2.23.1 for consistency with Section 1805.4.2.2 and with ASCE 7-05 Sections 12.13.5.2 and 12.13.6.2.

Clarifies the footing loads that are used to determine the tie force.

Where very large, heavily load foundations are adjacent to very small, lightly loads foundations basing the tie force on the larger foundation load can result in tie forces that are larger than the vertical loads being resisted by the smaller foundation. The added limit is to address this case.

Cost Impact: The code change proposal will not increase the cost of construction.

Public Hearing:				
	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S154–07/08

1805.4.6, 1807.2, 2304.9.5.2, Chapter 35; IRC R401.1, Chapter 43

Proponent: David P. Tyree, PE, CBO and Dennis Pitts, American Forest & Paper Association

THESE PROPOSALS ARE ON THE AGENDA OF THE IBC STRUCTURAL AND THE IRC BUILDING/ENERGY CODE DEVELOPMENT COMMITTEES AS 2 SEPARATE CODE CHANGES. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

PART I—IBC STRUCTURAL

1. Revise as follows:

1805.4.6 Wood foundations. Wood foundation systems shall be designed and installed in accordance with AF&PA ~~Technical Report No. 7~~ PWF. Lumber and plywood shall be treated in accordance with AWPA U1 (Commodity Specification A, Use Category 4B and Section 5.2) and shall be identified in accordance with Section 2303.1.8.1.

1807.2 Dampproofing required. Where hydrostatic pressure will not occur as determined by Section 1802.2.3, floors and walls for other than wood foundation systems shall be dampproofed in accordance with this section. Wood foundation systems shall be constructed in accordance with AF&PA ~~Technical Report No. 7~~ PWF.

2304.9.5.2 (Supp) Fastenings for wood foundations. Fastenings for wood foundations shall be as required in AF&PA ~~Technical Report No. 7~~ PWF.

2. Revise Chapter 35 standards as follows:

American Forest & Paper Association

> ~~T.R. No. 7—87 Technical Report—Basic Requirements for Permanent Wood Foundation System~~
> ANSI/AF&PA PWF-2007 Permanent Wood Foundation Design Specification.

PART II—IRC BUILDING/ENERGY

1. Revise as follows:

R401.1 Application. The provisions of this chapter shall control the design and construction of the foundation and foundation spaces for all buildings. In addition to the provisions of this chapter, the design and construction of foundations in areas prone to flooding as established by Table R301.2(1) shall meet the provisions of Section R324. Wood foundations shall be designed and installed in accordance with AF&PA ~~Report No. 7~~ PWF.

Exception: The provisions of this chapter shall be permitted to be used for wood foundations only in the following situations:

1. In buildings that have no more than two floors and a roof.
2. When interior basement and foundation walls are constructed at intervals not exceeding 50 feet (15 240 mm).

Wood foundations in Seismic Design Category D_0, D_1 or D_2 shall be designed in accordance with accepted engineering practice.

2. Revise standards in Chapter 43 as follows:

American Forest & Paper Association

~~T.R. No. 7—87 Basic Requirements for Permanent Wood Foundation System~~
ANSI/AF&PA PWF-2007 Permanent Wood Foundation Design Specification.

Reason: (IBC) This is an update to an existing AF&PA technical report which has been revised now to be a standard. The new standard is approved as an AF&PA standard and will further be approved as an ANSI consensus standard by August 2007. AF&PA's new standard for the design of Permanent Wood Foundations (PWF) entitled *PWF Design Specification* (*AF&PA PWF-2007*) was developed and balloted through AF&PA's Wood Design Standards Committee (WDSC), an ANSI-approved standards development group. The new PWF Design Specification was written as part of an effort to update design recommendations and procedures in the wood industry's documents design aides, such as *Technical Report 7: The Permanent Wood Foundation System* (1987) and the *The Permanent Wood Foundation System: Design, Fabrication and Installation Manual* (1987). Development of the new standard provides updated information and references to the latest wood design standards (NDS & SDPWS), the latest load standard (ASCE 7), and provides a consensus-based standard that can be referenced in the building code, replacing reference to *Technical Report 7*.

(IRC) The Permanent Wood Foundation Design Specification was developed as a consensus standard to replace reference to an AF&PA technical report that has been referenced in the codes for at least 20 years. The new specification refers the designer to current versions of the ASCE 7 standard for loading and to AF&PA's *National Design Specification for Wood Construction* and *Special Design Provisions for Wind and Seismic* for resistance. It also includes special requirements for below-ground design of treated wood structures. Copies of the Specification will be submitted separately.

Cost Impact: The code change proposal will not increase the cost of construction.

PART I — IBC STRUCTURAL

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

PART II — IRC BUILDING/ENERGY

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S155–07/08

202, 714.3, 1805.9, 1808.2.23.2, 2209.1, 2210, 2302.1; IRC R202, R403.1.4.1, R702.3.3, R702.3.6

Proponent: Philip Brazil, PE, SE, Reid Middleton, Inc., representing himself

THESE PROPOSALS ARE ON THE AGENDA OF THE IBC STRUCTURAL AND THE IRC BUILDING/ENERGY CODE DEVELOPMENT COMMITTEES AS 2 SEPARATE CODE CHANGES. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

PART I – IBC STRUCTURAL

Revise as follows:

SECTION 202
DEFINITIONS

LIGHT-FRAME CONSTRUCTION. A type of construction whose vertical and horizontal structural elements are primarily formed by a system of repetitive wood or ~~light gage~~ cold-formed steel framing members.

714.3 (Supp) Membrane protection. King studs and boundary elements that are integral elements in load-bearing walls of ~~light-framed~~ light-frame construction shall be permitted to have required fire-resistance ratings provided by the membrane protection provided for the load-bearing wall.

1805.9 (Supp) Seismic requirements. See Section 1908 for additional requirements for footings and foundations of structures assigned to Seismic Design Category C, D, E or F.

For structures assigned to Seismic Design Category D, E or F, provisions of ACI 318, Sections 21.10.1 to 21.10.3, shall apply when not in conflict with the provisions of Section 1805. Concrete shall have a specified compressive strength of not less than 3,000 psi (20.68 MPa) at 28 days.

Exceptions:

1. Group R or U occupancies of ~~light-framed~~ light-frame construction and two stories or less above grade plane are permitted to use concrete with a specified compressive strength of not less than 2,500 psi (17.2 MPa) at 28 days.
2. Detached one- and two-family dwellings of light-frame construction and two stories or less above grade plane are not required to comply with the provisions of ACI 318, Sections 21.10.1 to 21.10.3.

1808.2.23.2 (Supp) Seismic Design Category D, E or F. Where a structure is assigned to Seismic Design Category D, E or F in accordance with Section 1613, the requirements for Seismic Design Category C given in Section 1808.2.23.1 shall be met, in addition to the following. Provisions of ACI 318, Section 21.10.4, shall apply when not in conflict with the provisions of Sections 1808 through 1812. Concrete shall have a specified compressive strength of not less than 3,000 psi (20.68 MPa) at 28 days.

Exceptions:

1. Group R or U occupancies of ~~light-frame~~ light-frame construction and two stories or less above grade plane are permitted to use concrete with a specified compressive strength of not less than 2,500 psi (17.2 MPa) at 28 days.
2. Detached one- and two-family dwellings of light-frame construction and two stories or less in height are not required to comply with the provisions of ACI 318, Section 21.10.4.
3. Section 21.10.4.4(a) of ACI 318 need not apply to concrete piles.

2209.1 General. The design of cold-formed carbon and low-alloy steel structural members shall be in accordance with AISI-NAS. The design of cold-formed stainless-steel structural members shall be in accordance with ASCE 8. Cold-formed steel ~~light-framed~~ light-frame construction shall comply with Section 2210.

SECTION 2210
COLD-FORMED STEEL ~~LIGHT-FRAMED~~ LIGHT-FRAME CONSTRUCTION

2302.1 Definitions. The following words and terms shall, for the purposes of this chapter, have the meanings shown herein.

CONVENTIONAL LIGHT-FRAME ~~WOOD~~ CONSTRUCTION. A type of construction whose primary structural elements are formed by a system of repetitive wood-framing members. See Section 2308 for conventional light-frame ~~wood~~ construction provisions.

PART II – IRC BUILDING/ENERGY

Revise as follows:

SECTION R202
DEFINITIONS

~~LIGHT-FRAMED~~ LIGHT-FRAME CONSTRUCTION. A type of construction whose vertical and horizontal structural elements are primarily formed by a system of repetitive wood or ~~light gage~~ cold-formed steel framing members.

R403.1.4.1 Frost protection. Except where otherwise protected from frost, foundation walls, piers and other permanent supports of buildings and structures shall be protected from frost by one or more of the following methods:

1. Extended below the frost line specified in Table R301.2.(1);

2. Constructing in accordance with Section R403.3;
3. Constructing in accordance with ASCE 32; or
4. Erected on solid rock.

Exceptions:

1. Protection of freestanding accessory structures with an area of 600 square feet (56 m^2) or less, of ~~light-framed~~ light-frame construction, with an eave height of 10 feet (3048 mm) or less shall not be required.
2. Protection of freestanding accessory structures with an area of 400 square feet (37m^2) or less, of other than ~~light-framed~~ light-frame construction, with an eave height of 10 feet (3048 mm) or less shall not be required.
3. Decks not supported by a dwelling need not be provided with footings that extend below the frost line. Footings shall not bear on frozen soil unless the frozen condition is permanent.

R702.3.3 Steel framing. Steel framing supporting gypsum board shall not be less than 1.25 inches (32 mm) wide in the least dimension. ~~Light-gage~~ nonload-bearing cold-formed steel framing shall comply with ASTM C 645. Load-bearing steel framing and steel framing from 0.033 inch to 0.112 inch (1 mm to 3 mm) thick shall comply with ASTM C 955.

R702.3.6 Fastening. Screws for attaching gypsum board to wood framing shall be Type W or Type S in accordance with ASTM C 1002 and shall penetrate the wood not less than $^5/_8$ inch (16 mm). Screws for attaching gypsum board to ~~light-gage~~ cold-formed steel framing shall be Type S in accordance with ASTM C 1002 and shall penetrate the steel not less than $^3/_8$ inch (10 mm). Screws for attaching gypsum board to steel framing 0.033 inch to 0.112 inch (1mm to 3 mm) thick shall comply with ASTM C 954.

Reason: The purpose for the proposal is to harmonize the IBC, the IRC and the reference standards of both codes with respect to light-frame construction, cold-formed steel framing members and conventional light-frame construction. In IBC Section 2302.1, "conventional light-frame wood construction" is changed to "conventional light-frame construction" for consistency with IBC Section 2308 on conventional light-frame construction where the latter term is used consistently throughout the section. A thorough search of the IBC and IRC was performed during preparation of this proposal and the proposed revisions represent the only revisions necessary to complete the process of harmonization. Note that Section 11.2 of ASCE 7-05 defines "light-frame construction."

Cost Impact: The code change proposal will not increase the cost of construction.

PART I – IBC STRUCTURAL

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

PART II – IRC BUILDING/ENERGY

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S156–07/08
1806.1

Proponent: William Sherman, CH2M HILL, representing himself.

Revise as follows:

1806.1 General. Retaining walls shall be designed to ensure stability against overturning, sliding, excessive foundation pressure and water uplift. Retaining walls shall be designed for a safety factor of 1.5 against lateral sliding and overturning <u>excluding load combinations that include seismic forces</u>. <u>The dead load factor used in load combinations under Section 1605.3 shall be taken as 1.0 when used with the safety factors defined in this section. The safety factor against lateral sliding shall be taken as the available soil resistance at the base of the structure's foundation divided by the net lateral force applied to the structure.</u>

Reason: This proposal provides clarification and modification of existing code provisions.
An existing code clarification related to IBC 2000 states that the safety factor of 1.5 does not apply to load combinations that include seismic, but the published code remains unclear in this respect. Wording is added to clarify.

It is not practical to apply a reduction factor to dead loads, e.g. 0.6D, when an overall safety factor of 1.5 is also applied to stability requirements. Impractical and costly structures may result without the proposed revision.

The code does not define how the sliding safety factor is to be determined. Different safety factors can be obtained where passive pressures are included for retaining wall stability (depending upon whether the passive pressure force is included in the numerator or denominator). The proposed wording is based on recommended safety factor requirements in EM 1110-2-2502.

References:
EM 1110-2-2502, Retaining and Flood Walls, by the US Army Corps of Engineers (USACE).

Cost Impact: This code change proposal will reduce the cost of construction.

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S157–07/08
1806.1

Proponent: William Sherman, CH2M HILL, representing himself.

Revise as follows:

1806.1 General. Retaining walls shall be designed to ensure stability against overturning, sliding, excessive foundation pressure and water uplift. Retaining walls shall be designed for a safety factor of 1.5 against lateral sliding and overturning. <u>Where a keyway is extended below the wall base with the intent to engage passive pressure and enhance sliding stability, lateral soil pressures on both sides of the keyway shall be considered in the sliding analysis.</u>

Reason: Add a new requirement to existing provisions for retaining wall analyses, to ensure that a complete free-body diagram is used in design. Many software programs and design procedures for retaining walls with keyways ignore any driving pressures acting on the keyway, which is unconservative and ignores basic free-body diagram concepts. This provision clarifies that such forces must be considered but allows the engineer to determine the appropriate procedure.

References:
EM 1110-2-2502, Retaining and Flood Walls, by the US Army Corps of Engineers (USACE).

Cost Impact: This code change proposal will increase the cost of construction due to more conservative retaining wall design requirements than are commonly used.

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S158–07/08
1807, 1805

Proponent: Edwin T. Huston, Smith & Huston, Inc., representing the National Council of Structural Engineering Associations

Revise as follows:

SECTION ~~1807~~ <u>1805</u>
DAMPPROOFING AND WATERPROOFING

~~1807.1 Where required.~~ <u>1805.1 General.</u> Walls or portions thereof that retain earth and enclose interior spaces and floors below grade shall be waterproofed and dampproofed in accordance with this section, with the exception of those spaces containing groups other than residential and institutional where such omission is not detrimental to the building or occupancy.

Ventilation for crawl spaces shall comply with Section 1203.4.

~~1807.1.1~~ <u>1805.1.1</u> Story above grade plane. Where a basement is considered a story above grade plane and the finished ground level adjacent to the basement wall is below the basement floor elevation for 25 percent or more of the perimeter, the floor and walls shall be dampproofed in accordance with Section ~~1807.2~~ <u>1805.2</u> and a foundation drain shall be installed in accordance with Section ~~1807.4.2~~ <u>1805.4.2</u>. The foundation drain shall be installed around the portion of the perimeter where the basement floor is below ground level. The provisions of Sections 1802.2.3, <u>1805.3</u> ~~1807.3~~ and <u>1805.4.1</u> ~~1807.4.1~~ shall not apply in this case.

~~1807.1.2~~ <u>1805.1.2</u> **Under-floor space.** The finished ground level of an under-floor space such as a crawl space shall not be located below the bottom of the footings. Where there is evidence that the ground-water table rises to within 6 inches (152 mm) of the ground level at the outside building perimeter, or that the surface water does not readily drain from the building site, the ground level of the under-floor space shall be as high as the outside finished ground level, unless an approved drainage system is provided. The provisions of Sections 1802.2.3, ~~1807.2, 1807.3 and 1807.4~~ <u>1805.2, 1805.3 and 1805.4</u> shall not apply in this case.

~~1807.1.2.1~~ <u>1805.1.2.1</u> **Flood hazard areas.** For buildings and structures in flood hazard areas as established in Section 1612.3, the finished ground level of an under-floor space such as a crawl space shall be equal to or higher than the outside finished ground level.

Exception: Under-floor spaces of Group R-3 buildings that meet the requirements of FEMA/FIA-TB-11.

~~1807.1.3~~ <u>1805.1.3</u> **Ground-water control.** Where the ground-water table is lowered and maintained at an elevation not less than 6 inches (152 mm) below the bottom of the lowest floor, the floor and walls shall be dampproofed in accordance with Section ~~1807.2~~ <u>1805.2</u>. The design of the system to lower the ground-water table shall be based on accepted principles of engineering that shall consider, but not necessarily be limited to, permeability of the soil, rate at which water enters the drainage system, rated capacity of pumps, head against which pumps are to operate and the rated capacity of the disposal area of the system.

~~1807.2~~ <u>1805.2</u> **Dampproofing ~~required~~.** Where hydrostatic pressure will not occur as determined by Section 1802.2.3, floors and walls for other than wood foundation systems shall be dampproofed in accordance with this section. Wood foundation systems shall be constructed in accordance with AF&PA Technical Report No. 7.

~~1807.2.1~~ <u>1805.2.1</u> **Floors.** Dampproofing materials for floors shall be installed between the floor and the base course required by Section ~~1807.4.1~~ <u>1805.4.1</u>, except where a separate floor is provided above a concrete slab. Where installed beneath the slab, dampproofing shall consist of not less than 6-mil (0.006 inch; 0.152 mm) polyethylene with joints lapped not less than 6 inches (152 mm), or other approved methods or materials. Where permitted to be installed on top of the slab, dampproofing shall consist of mopped-on bitumen, not less than 4-mil (0.004 inch; 0.102 mm) polyethylene, or other approved methods or materials. Joints in the membrane shall be lapped and sealed in accordance with the manufacturer's installation instructions.

~~1807.2.2~~ <u>1805.2.2</u> **Walls.** Dampproofing materials for walls shall be installed on the exterior surface of the wall, and shall extend from the top of the footing to above ground level.

Dampproofing shall consist of a bituminous material, 3 pounds per square yard (16 N/m2) of acrylic modified cement, 0.125 inch (3.2 mm) coat of surface-bonding mortar complying with ASTM C 887, any of the materials permitted for waterproofing by Section ~~1807.3.2~~ <u>1805.3.2</u> or other approved methods or materials.

~~1807.2.2.1~~ <u>1805.2.2.1</u> **Surface preparation of walls.** Prior to application of dampproofing materials on concrete walls, holes and recesses resulting from the removal of form ties shall be sealed with a bituminous material or other approved methods or materials. Unit masonry walls shall be parged on the exterior surface belowground level with not less than 0.375 inch (9.5 mm) of portland cement mortar. The parging shall be coved at the footing.

Exception: Parging of unit masonry walls is not required where a material is approved for direct application to the masonry.

~~1807.3~~ <u>1805.3</u> **Waterproofing ~~required~~.** Where the ground-water investigation required by Section 1802.2.3 indicates that a hydrostatic pressure condition exists, and the design does not include a ground-water control system as described in Section ~~1807.1.3~~ <u>1805.1.3</u>, walls and floors shall be waterproofed in accordance with this section.

~~1807.3.1~~ <u>1805.3.1</u> **Floors.** Floors required to be waterproofed shall be of concrete and designed and constructed to withstand the hydrostatic pressures to which the floors will be subjected.

Waterproofing shall be accomplished by placing a membrane of rubberized asphalt, butyl rubber, fully adhered/fully bonded HDPE or polyolefin composite membrane or not less than 6-mil [0.006 inch (0.152 mm)] polyvinyl chloride with joints lapped not less than 6 inches (152 mm) or other approved materials under the slab. Joints in the membrane shall be lapped and sealed in accordance with the manufacturer's installation instructions.

~~1807.3.2~~ <u>1805.3.2</u> **Walls.** Walls required to be waterproofed shall be of concrete or masonry and shall be designed and constructed to withstand the hydrostatic pressures and other lateral loads to which the walls will be subjected.

Waterproofing shall be applied from the bottom of the wall to not less than 12 inches (305 mm) above the maximum elevation of the ground-water table. The remainder of the wall shall be dampproofed in accordance with Section ~~1807.2.2~~ <u>1805.2.2</u>. Waterproofing shall consist of two-ply hot-mopped felts, not less than 6-mil (0.006 inch; 0.152 mm) polyvinyl chloride, 40-mil (0.040 inch; 1.02 mm) polymer-modified asphalt, 6-mil (0.006 inch; 0.152 mm) polyethylene or other approved methods or materials capable of bridging nonstructural cracks. Joints in the membrane shall be lapped and sealed in accordance with the manufacturer's installation instructions.

~~1807.3.2.1~~ **<u>1805.3.2.1</u> Surface preparation of walls.** Prior to the application of waterproofing materials on concrete or masonry walls, the walls shall be prepared in accordance with Section ~~1807.2.2.1~~ <u>1805.2.2.1</u>.

~~1807.3.3~~ **<u>1805.3.3</u> Joints and penetrations.** Joints in walls and floors, joints between the wall and floor and penetrations of the wall and floor shall be made water-tight utilizing approved methods and materials.

~~1807.4~~ **<u>1805.4</u> Subsoil drainage system.** Where a hydrostatic pressure condition does not exist, dampproofing shall be provided and a base shall be installed under the floor and a drain installed around the foundation perimeter. A subsoil drainage system designed and constructed in accordance with Section ~~1807.1.3~~ <u>1805.1.3</u> shall be deemed adequate for lowering the ground-water table.

~~1807.4.1~~ **<u>1805.4.1</u> Floor base course.** Floors of basements, except as provided for in Section ~~1807.1.1~~ <u>1805.1.1</u>, shall be placed over a floor base course not less than 4 inches (102 mm) in thickness that consists of gravel or crushed stone containing not more than 10 percent of material that passes through a No. 4 (4.75 mm) sieve.

> **Exception:** Where a site is located in well-drained gravel or sand/gravel mixture soils, a floor base course is not required.

~~1807.4.2~~ **<u>1805.4.2</u> Foundation drain.** A drain shall be placed around the perimeter of a foundation that consists of gravel or crushed stone containing not more than 10-percent material that passes through a No. 4 (4.75 mm) sieve. The drain shall extend a minimum of 12 inches (305 mm) beyond the outside edge of the footing. The thickness shall be such that the bottom of the drain is not higher than the bottom of the base under the floor, and that the top of the drain is not less than 6 inches (152 mm) above the top of the footing. The top of the drain shall be covered with an approved filter membrane material. Where a drain tile or perforated pipe is used, the invert of the pipe or tile shall not be higher than the floor elevation. The top of joints or the top of perforations shall be protected with an approved filter membrane material. The pipe or tile shall be placed on not less than 2 inches (51 mm) of gravel or crushed stone complying with Section ~~1807.4.1~~ 1805.4.1, and shall be covered with not less than 6 inches (152 mm) of the same material.

~~1807.4.3~~ **<u>1805.4.3</u> Drainage discharge.** The floor base and foundation perimeter drain shall discharge by gravity or mechanical means into an approved drainage system that complies with the *International Plumbing Code*.

> **Exception:** Where a site is located in well-drained gravel or sand/gravel mixture soils, a dedicated drainage system is not required.

Reason: Relocates the section as part of an overall reorganization to consolidate structural design requirements that may later appear in reference standards. Makes minor editorial revisions of section titles (new Sections 1805.1, 1805.2, and 1805.3).

Bibliography:
Composite of Chapter 18 reorganization assuming all of proponent's proposals are approved.

Cost Impact: The code change proposal will not increase the cost of construction.

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S159–07/08
1807.1.2.1; IRC R408.7

Proponent: Rebecca C. Quinn, RC Quinn Consulting, Inc., representing US Department of Homeland Security, Federal Emergency Management Agency

THESE PROPOSALS ARE ON THE AGENDA OF THE IBC STRUCTURAL AND THE IRC BUILDING/ENERGY CODE DEVELOPMENT COMMITTEES AS 2 SEPARATE CODE CHANGES. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

PART I – IBC STRUCTURAL

Revise as follows:

1807.1.2.1 Flood hazard areas. For buildings and structures in flood hazard areas as established in Section 1612.3, the finished ground level of an under-floor space such as a crawl space shall be equal to or higher than the outside finished ground level<u> on at least one side</u>.

Exception: Under-floor spaces of Group R-3 buildings that meet the requirements of FEMA/ FIA-TB-11.

PART II – IRC BUILDING/ENERGY

Revise as follows:

R408.7 Flood resistance. For buildings located in areas prone to flooding as established in Table R301.2(1):

1. Walls enclosing the under-floor space shall be provided with flood openings in accordance with Section R324.2.2.
2. The finished ground level of the under-floor space shall be equal to or higher than the outside finished ground level<u> on at least one side</u>.

Exception: Under-floor spaces that meet the requirements of FEMA/FIA TB 11-1.

Reason: (IBC) The purpose of this code change is to clarify the existing requirement which is overly restrictive in that it requires the entire finished ground level of under-flood spaces to be equal to or higher than the outside finished ground level. In flood hazard areas, if the floor of an enclosed area, including a crawlspace, is below the exterior grade on all sides, then the building is considered to have a basement (as defined in 1612.2). Note that an under-floor space may be partially below grade; as long as the interior finished ground level is at or above the exterior finished grade on at least one side, then a basement as defined in 1612.2 is not created. Section 1807.1.2 specifies that the interior ground level of under-floor spaces may be below-grade if an approved drainage system is provided. This code change clarifies that that arrangement is not allowed in flood hazard areas because a basement (as defined in 1612.2) would be created.

(IRC) The purpose of this code change is to clarify the existing requirement which is overly restrictive in that it requires the entire finished ground level of under-flood spaces to be equal to or higher than the outside finished ground level. In flood hazard areas, if the floor of an enclosed area, including a crawlspace, is below the exterior grade on all sides, then the building is considered to have a basement (see R324.2.1(3)). Note that an under-floor space may be partially below grade; as long as the interior finished ground level is at or above the exterior finished grade on at least one side, then a basement is not created. Section R408.6 requires the grade in under-floor spaces to be as high as the outside finished grade unless an approved drainage system is provided. This code change clarifies that that arrangement is not allowed in flood hazard areas because a basement (below-grade on all sides) would be created.

Cost Impact: The code change proposal will not increase the cost of construction.

PART I – IBC STRUCTURAL

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

PART II – IRC BUILDING/ENERGY

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S160–07/08

1808, 1809, 1810 (New), 1811, 1812

Proponent: Edwin T. Huston, Smith & Huston, Inc., representing the National Council of Structural Engineering Associations

1. Revise as follows:

SECTION ~~1808~~ <u>1802</u>
~~PIER AND PILE FOUNDATIONS~~ <u>DEFINITIONS</u>

~~1808.1~~ <u>1802.1</u> Definitions. The following words and terms shall, for the purposes of this ~~section~~ <u>chapter</u>, have the meanings shown herein.

<u>**DEEP FOUNDATION.** A deep foundation is a foundation element that does not satisfy the definition of a shallow foundation.</u>

<u>**DRILLED SHAFT.** A drilled shaft is a cast-in-place deep foundation element constructed by drilling a hole (with or without permanent casing) into soil or rock and filling it with fluid concrete.</u>

Socketed drilled shaft A socketed drilled shaft is a drilled shaft with a permanent pipe or tube casing that extends down to bedrock and an uncased socket drilled into the bedrock [was caisson piles]

~~**FLEXURAL LENGTH.** Flexural length is the length of the pile from the first point of zero lateral deflection to the underside of the pile cap or grade beam.~~

MICROPILES. A micropile~~s~~ ~~are~~ is a ~~12-inch-diameter (305 mm) or less~~ bored, grouted-in-place deep foundation element ~~piles~~ that develops its load-carrying capacity by means of a bond zone in soil, bedrock, or a combination of soil and bedrock. ~~incorporating steel pipe (casing) and/or steel reinforcement.~~

SHALLOW FOUNDATION. A shallow foundation is an individual or strip footing, a mat foundation, a slab on grade foundation, or a similar foundation element.

~~**PIER FOUNDATIONS.** Pier foundations consist of isolated masonry or cast-in-place concrete structural elements extending into firm materials. Piers are relatively short in comparison to their width, with lengths less than or equal to 12 times the least horizontal dimension of the pier. Piers derive their load-carrying capacity through skin friction, through end bearing, or a combination of both.~~

~~**Belled piers.** Belled piers are cast-in-place concrete piers constructed with a base that is larger than the diameter of the remainder of the pier. The belled base is designed to increase the load-bearing area of the pier in end bearing.~~

~~**PILE FOUNDATIONS.** Pile foundations consist of concrete, wood or steel structural elements either driven into the ground or cast in place. Piles are relatively slender in comparison to their length, with lengths exceeding 12 times the least horizontal dimension. Piles derive their load-carrying capacity through skin friction, end bearing or a combination of both.~~

~~**Augered uncased piles.** Augered uncased piles are constructed by depositing concrete into an uncased augered hole, either during or after the withdrawal of the auger.~~

~~**Caisson piles.** Caisson piles are cast-in-place concrete piles extending into bedrock. The upper portion of a caisson pile consists of a cased pile that extends to the bedrock. The lower portion of the caisson pile consists of an uncased socket drilled into the bedrock.~~

~~**Concrete-filled steel pipe and tube piles.** Concrete-filled steel pipe and tube piles are constructed by driving a steel pipe or tube section into the soil and filling the pipe or tube section with concrete. The steel pipe or tube section is left in place during and after the deposition of the concrete.~~

~~**Driven uncased piles.** Driven uncased piles are constructed by driving a steel shell into the soil to shore an unexcavated hole that is later filled with concrete. The steel casing is lifted out of the hole during the deposition of the concrete.~~

~~**Enlarged base piles.** Enlarged base piles are cast-in-place concrete piles constructed with a base that is larger than the diameter of the remainder of the pile. The enlarged base is designed to increase the load-bearing area of the pile in end bearing.~~

~~**Steel-cased piles.** Steel-cased piles are constructed by driving a steel shell into the soil to shore an unexcavated hole. The steel casing is left permanently in place and filled with concrete.~~

~~**Timber piles.** Timber piles are round, tapered timbers with the small (tip) end embedded into the soil.~~

~~**1808.2 Piers and piles—general requirements.**~~

~~**1808.2.1 Design.** Piles are permitted to be designed in accordance with provisions for piers in Section 1808 and Sections 1812.3 through 1812.10 where either of the following conditions exists, subject to the approval of the building official:~~

1. ~~Group R-3 and U occupancies not exceeding two stories of light-frame construction, or~~
2. ~~Where the surrounding foundation materials furnish adequate lateral support for the pile.~~

~~**1808.2.2 General.** Pier and pile foundations shall be designed and installed on the basis of a foundation investigation as defined in Section 1802, unless sufficient data upon which to base the design and installation is available. The investigation and report provisions of Section 1802 shall be expanded to include, but not be limited to, the following:~~

1. ~~Recommended pier or pile types and installed capacities.~~
2. ~~Recommended center-to-center spacing of piers or piles.~~
3. ~~Driving criteria.~~
4. ~~Installation procedures.~~
5. ~~Field inspection and reporting procedures (to include procedures for verification of the installed bearing capacity where required).~~
6. ~~Pier or pile load test requirements.~~
7. ~~Durability of pier or pile materials~~
8. ~~Designation of bearing stratum or strata.~~
9. ~~Reductions for group action, where necessary~~

~~**1808.2.3 Special types of piles.** The use of types of piles not specifically mentioned herein is permitted, subject to the approval of the building official, upon the submission of acceptable test data, calculations and other information relating to the structural properties and load capacity of such piles. The allowable stresses shall not in any case exceed the limitations specified herein.~~

~~**1808.2.4 Pile caps.** Pile caps shall be of reinforced concrete, and shall include all elements to which piles are connected, including grade beams and mats. The soil immediately below the pile cap shall not be considered as carrying any vertical load. The tops of piles shall be embedded not less than 3 inches (76 mm) into pile caps and the caps shall extend at least 4 inches (102 mm) beyond the edges of piles. The tops of piles shall be cut back to sound material before capping.~~

~~**1808.2.5 (Supp) Stability.** Piers or piles shall be braced to provide lateral stability in all directions. Three or more piles connected by a rigid cap shall be considered braced, provided that the piles are located in radial directions from the centroid of the group not less than 60 degrees (1 rad) apart. A two-pile group in a rigid cap shall be considered to be braced along the axis connecting the two piles. Methods used to brace piers or piles shall be subject to the approval of the building official.~~

~~Piles supporting walls shall be driven alternately in lines spaced at least 1 foot (305 mm) apart and located symmetrically under the center of gravity of the wall load carried, unless effective measures are taken to provide for eccentricity and lateral forces, or the wall piles are adequately braced to provide for lateral stability. A single row of piles without lateral bracing is permitted for one- and two-family dwellings and lightweight construction not exceeding two stories above grade plane or 35 feet (10 668 mm) in building height, provided the centers of the piles are located within the width of the foundation wall.~~

~~**1808.2.6 Structural integrity.** Piers or piles shall be installed in such a manner and sequence as to prevent distortion or damage that may adversely affect the structural integrity of piles being installed or already in place.~~

~~**1808.2.7 Splices.** Splices shall be constructed so as to provide and maintain true alignment and position of the component parts of the pier or pile during installation and subsequent thereto and shall be of adequate strength to transmit the vertical and lateral loads and moments occurring at the location of the splice during driving and under service loading. Splices shall develop not less than 50 percent of the least capacity of the pier or pile in bending. In addition, splices occurring in the upper 10 feet (3048 mm) of the embedded portion of the pier or pile shall be capable of resisting at allowable working stresses the moment and shear that would result from an assumed eccentricity of the pier or pile load of 3 inches (76 mm), or the pier or pile shall be braced in accordance with Section 1808.2.5 to other piers or piles that do not have splices in the upper 10 feet (3048 mm) of embedment.~~

~~**1808.2.8 Allowable pier or pile loads.**~~

~~**1808.2.8.1 Determination of allowable loads.** The allowable axial and lateral loads on piers or piles shall be determined by an approved formula, load tests or method of analysis.~~

~~**1808.2.8.2 Driving criteria.** The allowable compressive load on any pile where determined by the application of an approved driving formula shall not exceed 40 tons (356 kN). For allowable loads above 40 tons (356 kN), the wave equation method of analysis shall be used to estimate pile driveability of both driving stresses and net displacement per blow at the ultimate load. Allowable loads shall be verified by load tests in accordance with Section 1808.2.8.3. The formula or wave equation load shall be determined for gravity-drop or power-actuated hammers and the hammer energy used shall be the maximum consistent with the size, strength and weight of the driven piles. The use of a follower is permitted only with the approval of the building official. The introduction of fresh hammer cushion or pile cushion material just prior to final penetration is not permitted.~~

~~**1808.2.8.3 Load tests.** Where design compressive loads per pier or pile are greater than those permitted by Section 1808.2.10 or where the design load for any pier or pile foundation is in doubt, control test piers or piles shall be tested in accordance with ASTM D 1143 or ASTM D 4945. At least one pier or pile shall be test loaded in each area of uniform subsoil conditions. Where required by the building official, additional piers or piles shall be load tested where necessary to establish the safe design capacity. The resulting allowable loads shall not be more than one-half of the ultimate axial load capacity of the test pier or pile as assessed by one of the published methods listed in Section 1808.2.8.3.1 with consideration for the test type, duration and subsoil. The ultimate axial load capacity shall be determined by a registered design professional with consideration given to tolerable total and differential settlements at design load in accordance with Section 1808.2.12. In subsequent installation of the balance of foundation piles, all piles shall be deemed to have a supporting capacity equal to the control pile where such piles are of the same type, size and relative length as the test pile; are installed using the same or comparable methods and equipment as the test pile; are installed in similar subsoil conditions as the test pile; and, for driven piles, where the rate of penetration (e.g., net displacement per blow) of such piles is equal to or less than that of the test pile driven with the same hammer through a comparable driving distance.~~

~~**1808.2.8.3.1 Load test evaluation.** It shall be permitted to evaluate pile load tests with any of the following methods:~~

1. ~~Davisson Offset Limit.~~
2. ~~Brinch-Hansen 90% Criterion.~~
3. ~~Butler-Hoy Criterion.~~
4. ~~Other methods approved by the building official.~~

~~**1808.2.8.4 Allowable frictional resistance.** The assumed frictional resistance developed by any pier or uncased cast-in-place pile shall not exceed one-sixth of the bearing value of the soil material at minimum depth as set forth in Table 1804.2, up to a maximum of 500 psf (24 kPa), unless a greater value is allowed by the building official after a soil investigation, as specified in Section 1802, is submitted or a greater value is substantiated by a load test in accordance with Section 1808.2.8.3. Frictional resistance and bearing resistance shall not be assumed to act simultaneously unless recommended by a soil investigation as specified in Section 1802.~~

~~**1808.2.8.5 Uplift capacity.** Where required by the design, the uplift capacity of a single pier or pile shall be determined by an approved method of analysis based on a minimum factor of safety of three or by load tests conducted in accordance with ASTM D 3689. The maximum allowable uplift load shall not exceed the ultimate load capacity as determined in Section 1808.2.8.3 divided by a factor of safety of two. For pile groups subjected to uplift, the allowable working uplift load for the group shall be the lesser of:~~

1. ~~The proposed individual pile uplift working load times the number of piles in the group.~~
2. ~~Two-thirds of the effective weight of the pile group and the soil contained within a block defined by the perimeter of the group and the length of the pile.~~

~~**1808.2.8.6 Load-bearing capacity.** Piers, individual piles and groups of piles shall develop ultimate load capacities of at least twice the design working loads in the designated load-bearing layers. Analysis shall show that no soil layer underlying the designated load-bearing layers causes the load-bearing capacity safety factor to be less than two.~~

~~**1808.2.8.7 Bent piers or piles.** The load-bearing capacity of piers or piles discovered to have a sharp or sweeping bend shall be determined by an approved method of analysis or by load testing a representative pier or pile.~~

~~**1808.2.8.8 Overloads on piers or piles.** The maximum compressive load on any pier or pile due to mislocation shall not exceed 110 percent of the allowable design load.~~

~~**1808.2.9 Lateral support.**~~

~~**1808.2.9.1 General.** Any soil other than fluid soil shall be deemed to afford sufficient lateral support to the pier or pile to prevent buckling and to permit the design of the pier or pile in accordance with accepted engineering practice and the applicable provisions of this code.~~

~~**1808.2.9.2 Unbraced piles.** Piles standing unbraced in air, water or in fluid soils shall be designed as columns in accordance with the provisions of this code. Such piles driven into firm ground can be considered fixed and laterally supported at 5 feet (1524 mm) below the ground surface and in soft material at 10 feet (3048 mm) below the ground surface unless otherwise prescribed by the building official after a foundation investigation by an approved agency.~~

~~**1808.2.9.3 Allowable lateral load.** Where required by the design, the lateral load capacity of a pier, a single pile or a pile group shall be determined by an approved method of analysis or by lateral load tests to at least twice the proposed design working load. The resulting allowable load shall not be more than one-half of that test load that produces a gross lateral movement of 1 inch (25 mm) at the ground surface.~~

~~**1808.2.10 Use of higher allowable pier or pile stresses.** Allowable stresses greater than those specified for piers or for each pile type in Sections 1809 and 1810 are permitted where supporting data justifying such higher stresses is filed with the building official. Such substantiating data shall include:~~

1. ~~A soils investigation in accordance with Section 1802.~~
2. ~~Pier or pile load tests in accordance with Section 1808.2.8.3, regardless of the load supported by the pier or pile.~~

~~The design and installation of the pier or pile foundation shall be under the direct supervision of a registered design professional knowledgeable in the field of soil mechanics and pier or pile foundations who shall certify to the building official that the piers or piles as installed satisfy the design criteria.~~

~~**1808.2.11 Piles in subsiding areas.** Where piles are installed through subsiding fills or other subsiding strata and derive support from underlying firmer materials, consideration shall be given to the downward frictional forces that may be imposed on the piles by the subsiding upper strata. Where the influence of subsiding fills is considered as imposing loads on the pile, the allowable stresses specified in this chapter are permitted to be increased where satisfactory substantiating data are submitted.~~

~~**1808.2.12 Settlement analysis.** The settlement of piers, individual piles or groups of piles shall be estimated based on approved methods of analysis. The predicted settlement shall cause neither harmful distortion of, nor instability in, the structure, nor cause any stresses to exceed allowable values.~~

~~**1808.2.13 Preexcavation.** The use of jetting, augering or other methods of preexcavation shall be subject to the approval of the building official. Where permitted, preexcavation shall be carried out in the same manner as used for piers or piles subject to load tests and in such a manner that will not impair the carrying capacity of the piers or piles already in place or damage adjacent structures. Pile tips shall be driven below the preexcavated depth until the required resistance or penetration is obtained.~~

~~**1808.2.14 Installation sequence.** Piles shall be installed in such sequence as to avoid compacting the surrounding soil to the extent that other piles cannot be installed properly, and to prevent ground movements that are capable of damaging adjacent structures.~~

~~**1808.2.15 Use of vibratory drivers.** Vibratory drivers shall only be used to install piles where the pile load capacity is verified by load tests in accordance with Section 1808.2.8.3. The installation of production piles shall be controlled according to power consumption, rate of penetration or other approved means that ensure pile capacities equal or exceed those of the test piles.~~

~~**1808.2.16 Pile driveability.** Pile cross sections shall be of sufficient size and strength to withstand driving stresses without damage to the pile, and to provide sufficient stiffness to transmit the required driving forces.~~

~~**1808.2.17 Protection of pile materials.** Where boring records or site conditions indicate possible deleterious action on pier or pile materials because of soil constituents, changing water levels or other factors, the pier or pile materials shall be adequately protected by materials, methods or processes approved by the building official. Protective materials shall be applied to the piles so as not to be rendered ineffective by driving. The effectiveness of such protective measures for the particular purpose shall have been thoroughly established by satisfactory service records or other evidence.~~

~~**1808.2.18 Use of existing piers or piles.** Piers or piles left in place where a structure has been demolished shall not be used for the support of new construction unless satisfactory evidence is submitted to the building official, which indicates that the piers or piles are sound and meet the requirements of this code. Such piers or piles shall be load tested or redriven to verify their capacities. The design load applied to such piers or piles shall be the lowest allowable load as determined by tests or redriving data.~~

~~**1808.2.19 Heaved piles.** Piles that have heaved during the driving of adjacent piles shall be redriven as necessary to develop the required capacity and penetration, or the capacity of the pile shall be verified by load tests in accordance with Section 1808.2.8.3.~~

~~**1808.2.20 Identification.** Pier or pile materials shall be identified for conformity to the specified grade with this identity maintained continuously from the point of manufacture to the point of installation or shall be tested by an approved agency to determine conformity to the specified grade. The approved agency shall furnish an affidavit of compliance to the building official.~~

~~**1808.2.21 Pier or pile location plan.** A plan showing the location and designation of piers or piles by an identification system shall be filed with the building official prior to installation of such piers or piles. Detailed records for piers or individual piles shall bear an identification corresponding to that shown on the plan.~~

~~**1808.2.22 Special inspection.** Special inspections in accordance with Sections 1704.8 and 1704.9 shall be provided for piles and piers, respectively.~~

~~**1808.2.23 Seismic design of piers or piles.**~~

~~**1808.2.23.1 (Supp) Seismic Design Category C.** Where a structure is assigned to Seismic Design Category C in accordance with Section 1613, the following shall apply. Individual pile caps, piers or piles shall be interconnected by ties. Ties shall be capable of carrying, in tension and compression, a force equal to the product of the larger pile cap or column load times the seismic coefficient, S_{DS}, divided by 10 unless it can be demonstrated that equivalent restraint is provided by reinforced concrete beams within slabs on grade or reinforced concrete slabs on grade or confinement by competent rock, hard cohesive soils or very dense granular soils.~~

> ~~**Exception:** In Groups R-3 and U occupancies of light-frame construction, pier foundations supporting foundation walls, isolated interior posts detailed so the pier is not subject to lateral loads, or exterior decks and patios, are not subject to interconnection if it can be shown the soils are of adequate stiffness, subject to the approval of the building official.~~

~~**1808.2.23.1.1 Connection to pile cap.** Concrete piles and concrete-filled steel pipe piles shall be connected to the pile cap by embedding the pile reinforcement or field-placed dowels anchored in the concrete pile in the pile cap for a distance equal to the development length. For deformed bars, the development length is the full development length for compression or tension, in the case of uplift, without reduction in length for excess area. Alternative measures for laterally confining concrete and maintaining toughness and ductile-like behavior at the top of the pile will be permitted provided the design is such that any hinging occurs in the confined region.~~

~~Ends of hoops, spirals and ties shall be terminated with seismic hooks, as defined in Section 21.1 of ACI 318, turned into the confined concrete core. The minimum transverse steel ratio for confinement shall not be less than one-half of that required for columns.~~

~~For resistance to uplift forces, anchorage of steel pipe (round HSS sections), concrete-filled steel pipe or H-piles to the pile cap shall be made by means other than concrete bond to the bare steel section.~~

> ~~**Exception:** Anchorage of concrete-filled steel pipe piles is permitted to be accomplished using deformed bars developed into the concrete portion of the pile.~~

~~Splices of pile segments shall develop the full strength of the pile, but the splice need not develop the nominal strength of the pile in tension, shear and bending when it has been designed to resist axial and shear forces and moments from the load combinations of Section 1605.4.~~

~~**1808.2.23.1.2 Design details.** Pier or pile moments, shears and lateral deflections used for design shall be established considering the nonlinear interaction of the shaft and soil, as recommended by a registered design professional. Where the ratio of the depth of embedment of the pile-to-pile diameter or width is less than or equal to six, the pile may be assumed to be rigid. Pile group effects from soil on lateral pile nominal strength shall be included where pile center-to-center spacing in the direction of lateral force is less than eight pile diameters. Pile group effects on vertical nominal strength shall be included where pile center- to-center spacing is less than three pile diameters. The pile uplift soil nominal strength shall be taken as the pile uplift strength as limited by the frictional force developed between the soil and the pile. Where a minimum length for reinforcement or the extent of closely spaced confinement reinforcement is specified at the top of the pier or pile, provisions shall be made so that those specified lengths or extents are maintained after pier or pile cutoff.~~

~~**1808.2.23.2 (Supp) Seismic Design Category D, E or F.** Where a structure is assigned to Seismic Design Category D, E or F in accordance with Section 1613, the requirements for Seismic Design Category C given in Section 1808.2.23.1 shall be met, in addition to the following. Provisions of ACI 318, Section 21.10.4, shall apply when not in conflict with the provisions of Sections 1808 through 1812. Concrete shall have a specified compressive strength of not less than 3,000 psi (20.68 MPa) at 28 days.~~

~~**Exceptions:**~~

1. ~~Group R or U occupancies of light-frame construction and two stories or less above grade plane are permitted to use concrete with a specified compressive strength of not less than 2,500 psi (17.2 MPa) at 28 days.~~
2. ~~Detached one- and two-family dwellings of light-frame construction and two stories or less in height are not required to comply with the provisions of ACI 318, Section 21.10.4.~~
3. ~~Section 21.10.4.4(a) of ACI 318 need not apply to concrete piles.~~

~~**1808.2.23.2.1 (Supp) Design details for piers, piles and grade beams.** Piers or piles on Site Class E or F sites, as determined in Section 1613.5.2, shall be designed and constructed to withstand maximum imposed curvatures from earthquake ground motions and structure response. Curvatures shall include free-field soil strains modified for soil pile-structure interaction coupled with pier or pile deformations induced by lateral pier or pile resistance to structure seismic forces.~~

~~**Exception:** Piers or piles that satisfy the following additional detailing requirements shall be deemed to comply with the curvature capacity requirements of this section.~~

1. ~~Precast prestressed concrete piles detailed in accordance with Section 1809.2.3.2.2.~~
2. ~~Cast-in-place concrete piles with a minimum longitudinal reinforcement ratio of 0.005 extending the full length of the pile and detailed in accordance with Sections 21.4.4.1, 21.4.4.2 and 21.4.4.3 of ACI 318 as required by this section.~~

~~Where constructed of nonprestressed concrete such piers or piles shall be designed and detailed in accordance with Sections 21.4.4.1, 21.4.4.2 and 21.4.4.3 of ACI 318 within seven pile diameters of the pile cap and within seven pile diameters of the interfaces of strata that are hard or stiff and strata that are liquefiable or are composed of soft to medium stiff clay.~~

~~Grade beams shall comply with the provisions in Section 21.10.3 of ACI 318 for grade beams, except where they have the capacity to resist the forces from the load combinations in Section 1605.4.~~

~~**1808.2.23.2.2 Connection to pile cap.** For piles required to resist uplift forces or provide rotational restraint, design of anchorage of piles into the pile cap shall be provided considering the combined effect of axial forces due to uplift and bending moments due to fixity to the pile cap. Anchorage shall develop a minimum of 25 percent of the strength of the pile in tension. Anchorage into the pile cap shall be capable of developing the following:~~

1. ~~In the case of uplift, the lesser of the nominal tensile strength of the longitudinal reinforcement in a concrete pile, or the nominal tensile strength of a steel pile, or the pile uplift soil nominal strength factored by 1.3 or the axial tension force resulting from the load combinations of Section 1605.4.~~
2. ~~In the case of rotational restraint, the lesser of the axial and shear forces, and moments resulting from the load combinations of Section 1605.4 or development of the full axial, bending and shear nominal strength of the pile.~~

~~**1808.2.23.2.3 Flexural strength.** Where the vertical lateral-force-resisting elements are columns, the grade beam or pile cap flexural strengths shall exceed the column flexural strength. The connection between batter piles and grade beams or pile caps shall be designed to resist the nominal strength of the pile acting as a short column. Batter piles and their connection shall be capable of resisting forces and moments from the load combinations of Section 1605.4.~~

~~SECTION 1809~~
~~DRIVEN PILE FOUNDATIONS~~

~~**1809.1 Timber piles.** Timber piles shall be designed in accordance with the AF&PA NDS.~~

~~**1809.1.1 Materials.** Round timber piles shall conform to ASTM D 25. Sawn timber piles shall conform to DOC PS-20~~

~~**1809.1.2 Preservative treatment.** Timber piles used to support permanent structures shall be treated in accordance with this section unless it is established that the tops of the untreated timber piles will be below the lowest ground-water level assumed to exist during the life of the structure. Preservative and minimum final retention shall be in accordance withAWPAU1 (Commodity Specification E, Use Category 4C) for round timber piles and AWPA U1 (Commodity Specification A, Use Category 4B) for sawn timber piles. Preservative-treated timber piles shall be subject to a quality control program administered by an approved agency. Pile cutoffs shall be treated in accordance with AWPA M4.~~

~~**1809.1.3 Defective piles.** Any substantial sudden increase in rate of penetration of a timber pile shall be investigated for possible damage. If the sudden increase in rate of penetration cannot be correlated to soil strata, the pile shall be removed for inspection or rejected.~~

~~**1809.1.4 Allowable stresses.** The allowable stresses shall be in accordance with the AF&PA NDS.~~

~~**1809.2 Precast concrete piles.**~~

~~**1809.2.1 General.** The materials, reinforcement and installation of precast concrete piles shall conform to Sections 1809.2.1.1 through 1809.2.1.4.~~

~~**1809.2.1.1 Design and manufacture.** Piles shall be designed and manufactured in accordance with accepted engineering practice to resist all stresses induced by handling, driving and service loads.~~

~~**1809.2.1.2 Minimum dimension.** The minimum lateral dimension shall be 8 inches (203 mm). Corners of square piles shall be chamfered.~~

~~**1809.2.1.3 Reinforcement.** Longitudinal steel shall be arranged in a symmetrical pattern and be laterally tied with steel ties or wire spiral spaced not more than 4 inches (102 mm) apart, center to center, for a distance of 2 feet (610 mm) from the ends of the pile; and not more than 6 inches (152 mm) elsewhere except that at the ends of each pile, the first five ties or spirals shall be spaced 1 inch (25 mm) center to center. The gage of ties and spirals shall be as follows:~~

~~For piles having a diameter of 16 inches (406 mm) or less, wire shall not be smaller than 0.22 inch (5.6 mm) (No. 5 gage).~~

~~For piles having a diameter of more than 16 inches (406 mm) and less than 20 inches (508 mm), wire shall not be smaller than 0.238 inch (6 mm) (No. 4 gage).~~

~~For piles having a diameter of 20 inches (508 mm) and larger, wire shall not be smaller than 0.25 inch (6.4 mm) round or 0.259 inch (6.6 mm) (No. 3 gage).~~

~~**1809.2.1.4 Installation.** Piles shall be handled and driven so as not to cause injury or overstressing, which affects durability or strength.~~

~~**1809.2.2 Precast nonprestressed piles.** Precast nonprestressed concrete piles shall conform to Sections 1809.2.2.1 through 1809.2.2.5.~~

~~**1809.2.2.1 Materials.** Concrete shall have a 28-day specified compressive strength (f'_c) of not less than 3,000 psi (20.68 MPa).~~

~~**1809.2.2.2 Minimum reinforcement.** The minimum amount of longitudinal reinforcement shall be 0.8 percent of the concrete section and shall consist of at least four bars.~~

~~**1809.2.2.2.1 Seismic reinforcement in Seismic Design Category C.** Where a structure is assigned to Seismic Design Category C in accordance with Section 1613, the following shall apply. Longitudinal reinforcement with a minimum steel ratio of 0.01 shall be provided throughout the length of precast concrete piles. Within three pile diameters of the bottom of the pile cap, the longitudinal reinforcement shall be confined with closed ties or spirals of a minimum 3/8 inch (9.5 mm) diameter. Ties or spirals shall be provided at a maximum spacing of eight times the diameter of the smallest longitudinal bar, not to exceed 6 inches (152 mm). Throughout the remainder of the pile, the closed ties or spirals shall have a maximum spacing of 16 times the smallest longitudinal bar diameter not to exceed 8 inches (203 mm).~~

~~**1809.2.2.2.2 Seismic reinforcement in Seismic Design Category D, E or F.** Where a structure is assigned to Seismic Design Category D, E or F in accordance with Section 1613, the requirements for Seismic Design Category C in Section 1809.2.2.2.1 shall apply except as modified by this section. Transverse confinement reinforcement consisting of closed ties or equivalent spirals shall be provided in accordance with Sections 21.4.4.1, 21.4.4.2 and 21.4.4.3 of ACI 318 within three pile diameters of the bottom of the pile cap. For other than Site Class E or F, or liquefiable sites and where spirals are used as the transverse reinforcement, a volumetric ratio of spiral reinforcement of not less than one-half that required by Section 21.4.4.1(a) of ACI 318 shall be permitted.~~

~~**1809.2.2.3 Allowable stresses.** The allowable compressive stress in the concrete shall not exceed 33 percent of the 28-day specified compressive strength (f'_c) applied to the gross cross-sectional area of the pile. The allowable compressive stress in the reinforcing steel shall not exceed 40 percent of the yield strength of the steel (f_y) or a maximum of 30,000 psi (207 MPa). The allowable tensile stress in the reinforcing steel shall not exceed 50 percent of the yield strength of the steel (f_y) or a maximum of 24,000 psi (165 MPa).~~

~~**1809.2.2.4 Installation.** A precast concrete pile shall not be driven before the concrete has attained a compressive strength of at least 75 percent of the 28-day specified compressive strength (f'_c), but not less than the strength sufficient to withstand handling and driving forces.~~

~~**1809.2.2.5 Concrete cover.** Reinforcement for piles that are not manufactured under plant conditions shall have a concrete cover of not less than 2 inches (51 mm).~~

~~Reinforcement for piles manufactured under plant control conditions shall have a concrete cover of not less than 1.25 inches (32 mm) for No. 5 bars and smaller, and not less than 1.5 inches (38 mm) for No. 6 through No. 11 bars except that longitudinal bars spaced less than 1.5 inches (38 mm) clear distance apart shall be considered bundled bars for which the minimum concrete cover shall be equal to that for the equivalent diameter of the bundled bars.~~

~~Reinforcement for piles exposed to seawater shall have a concrete cover of not less than 3 inches (76 mm).~~

~~**1809.2.3 Precast prestressed piles.** Precast prestressed concrete piles shall conform to the requirements of Sections 1809.2.3.1 through 1809.2.3.5.~~

~~**1809.2.3.1 Materials.** Prestressing steel shall conform to ASTM A 416. Concrete shall have a 28-day specified compressive strength (f'_c) of not less than 5,000 psi (34.48 MPa).~~

~~**1809.2.3.2 Design.** Precast prestressed piles shall be designed to resist stresses induced by handling and driving as well as by loads. The effective prestress in the pile shall not be less than 400 psi (2.76MPa) for piles up to 30 feet (9144 mm) in length, 550 psi (3.79 MPa) for piles up to 50 feet (15 240 mm) in length and 700 psi (4.83 MPa) for piles greater than 50 feet (15 240 mm) in length. Effective prestress shall be based on an assumed loss of 30,000 psi (207 MPa) in the prestressing steel. The tensile stress in the prestressing steel shall not exceed the values specified in ACI 318.~~

~~**1809.2.3.2.1 Design in Seismic Design Category C.** Where a structure is assigned to Seismic Design Category C in accordance with Section 1613, the following shall apply. The minimum volumetric ratio of spiral reinforcement shall not be less than 0.007 or the amount required by the following formula for the upper 20 feet (6096 mm) of the pile.~~

$$\cancel{\rho_s = 0.12 f'_c / f_{yh}} \qquad \textbf{(Equation 18-4)}$$

~~where:~~

~~f'_c = Specified compressive strength of concrete, psi (MPa).~~
~~f_{yh} = Yield strength of spiral reinforcement ≤ 85,000 psi (586 MPa).~~
~~ρ_s = Spiral reinforcement index (vol. spiral/vol. core).~~

~~At least one-half the volumetric ratio required by Equation 18-4 shall be provided below the upper 20 feet (6096 mm) of the pile. The pile cap connection by means of dowels as indicated in Section 1808.2.23.1 is permitted. Pile cap connection by means of developing pile reinforcing strand is permitted provided that the pile reinforcing strand results in a ductile connection.~~

~~**1809.2.3.2.2 Design in Seismic Design Category D, E or F.** Where a structure is assigned to Seismic Design Category D, E or F in accordance with Section 1613, the requirements for Seismic Design Category C in Section 1809.2.3.2.1 shall be met, in addition to the following:~~

1. ~~Requirements in ACI 318, Chapter 21, need not apply, unless specifically referenced.~~
2. ~~Where the total pile length in the soil is 35 feet (10 668 mm) or less, the lateral transverse reinforcement in the ductile region shall occur through the length of the pile. Where the pile length exceeds 35 feet (10 668 mm), the ductile pile region shall be taken as the greater of 35 feet (10 668 mm) or the distance from the underside of the pile cap to the point of zero curvature plus three times the least pile dimension.~~
3. ~~In the ductile region, the center-to-center spacing of the spirals or hoop reinforcement shall not exceed one-fifth of the least pile dimension, six times the diameter of the longtitudinal strand, or 8 inches (203 mm), whichever is smaller.~~
4. ~~Circular spiral reinforcement shall be spliced by lapping one full turn and bending the end of the spiral to a 90-degree hook or by use of a mechanical or welded splice complying with Sec. 12.14.3 of ACI 318.~~
5. ~~Where the transverse reinforcement consists of circular spirals, the volumetric ratio of spiral transverse reinforcement in the ductile region shall comply with the following:~~

$$\cancel{\rho_s = 0.25(f'_c/f_{yh})(A_g/A_{ch} - 1.0)[0.5 + 1.4P/(f'_c A_g)]} \qquad \textbf{(Equation 18-5)}$$

~~but not less than:~~

$$\cancel{\rho_s = 0.12(f'_c/f_{yh})[0.5 + 1.4P/(f'_c A_g)]}$$ ~~**(Equation 18-6)**~~

~~and need not exceed:~~

$$\cancel{\rho_s = 0.021}$$ ~~**(Equation 18-7)**~~

~~where:~~

~~A_g = Pile cross-sectional area, square inches (mm^2).~~
~~A_{ch} = Core area defined by spiral outside diameter, square inches (mm^2).~~
~~f'_c = Specified compressive strength of concrete, psi (MPa).~~
~~f_{yh} = Yield strength of spiral reinforcement ≤ 85,000 psi (586 MPa).~~
~~P = Axial load on pile, pounds (kN), as determined from Equations 16-5 and 16-6.~~
~~ρ_s = Volumetric ratio (vol. spiral/ vol. core).~~

~~This required amount of spiral reinforcement is permitted to be obtained by providing an inner and outer spiral.~~

6. ~~When transverse reinforcement consists of rectangular hoops and cross ties, the total cross-sectional area of lateral transverse reinforcement in the ductile region with spacings, and perpendicular to dimension, h_c, shall conform to:~~

$$\cancel{A_{sh} = 0.3sh_c(f'_c/f_{yh})(A_g/A_{ch} - 1.0)[0.5 + 1.4P/(f'_c A_g)]}$$ ~~**(Equation 18-8)**~~

~~but not less than:~~

$$\cancel{A_{sh} = 0.12sh_c(f'_c/f_{yh})[0.5 + 1.4P/(f'_c A_g)]}$$ ~~**(Equation 18-9)**~~

~~where:~~

~~f_{yh} = ≤ 70,000 psi (483 MPa).~~
~~h_c = Cross-sectional dimension of pile core measured center to center of hoop reinforcement, inch (mm).~~
~~s = Spacing of transverse reinforcement measured along length of pile, inch (mm).~~
~~A_{sh} = Cross-sectional area of tranverse reinforcement, square inches (mm^2).~~
~~f'_c = Specified compressive strength of concrete, psi (MPa).~~

~~The hoops and cross ties shall be equivalent to deformed bars not less than No. 3 in size. Rectangular hoop ends shall terminate at a corner with seismic hooks.~~

~~Outside of the length of the pile requiring transverse confinement reinforcing, the spiral or hoop reinforcing with a volumetric ratio not less than one-half of that required for transverse confinement reinforcing shall be provided.~~

~~**1809.2.3.3 (Supp) Allowable stresses.** The allowable compressive stress, f_c, in concrete shall be determined as follows:~~

$$\cancel{f'_c = 0.33 f'_c - 0.27 f_{pc}}$$ ~~**(Equation 18-10)**~~

~~where:~~

~~f'_c = The 28-day specified compressive strength of the concrete~~
~~f_{pc} = The effective prestress stress on the gross section.~~

~~**1809.2.3.4 Installation.** A prestressed pile shall not be driven before the concrete has attained a compressive strength of at least 75 percent of the 28-day specified compressive strength (f'_c), but not less than the strength sufficient to withstand handling and driving forces.~~

~~**1809.2.3.5 Concrete cover.** Prestressing steel and pile reinforcement shall have a concrete cover of not less than $1^1/_4$ inches (32 mm) for square piles of 12 inches (305 mm) or smaller size and $1^1/_2$ inches (38 mm) for larger piles, except that for piles exposed to seawater, the minimum protective concrete cover shall not be less than $2^1/_2$ inches (64 mm).~~

~~**1809.3 Structural steel piles.** Structural steel piles shall conform to the requirements of Sections 1809.3.1 through 1809.3.4.~~

~~**1809.3.1 Materials.** Structural steel piles, steel pipe and fully welded steel piles fabricated from plates shall conform to ASTMA36, ASTMA252, ASTMA283, ASTMA572, ASTM A 588, ASTM A 690, ASTM A 913 or ASTM A 992.~~

~~**1809.3.2 Allowable stresses.** The allowable axial stresses shall not exceed 35 percent of the minimum specified yield strength (F_y).~~

> ~~**Exception:** Where justified in accordance with Section 1808.2.10, the allowable axial stress is permitted to be increased above 0.35F_y, but shall not exceed 0.5F_y.~~

~~**1809.3.3 Dimensions of H-piles.** Sections of H-piles shall comply with the following:~~

1. ~~The flange projections shall not exceed 14 times the minimum thickness of metal in either the flange or the web and the flange widths shall not be less than 80 percent of the depth of the section.~~
2. ~~The nominal depth in the direction of the web shall not be less than 8 inches (203 mm).~~
3. ~~Flanges and web shall have a minimum nominal thickness of $^3/_8$ inch (9.5 mm).~~

~~**1809.3.4 Dimensions of steel pipe piles.** Steel pipe piles driven open ended shall have a nominal outside diameter of not less than 8 inches (203 mm). The pipe shall have a minimum cross section of 0.34 square inch (219 mm_2) to resist each 1,000 foot-pounds (1356 N-m) of pile hammer energy, or shall have the equivalent strength for steels having a yield strength greater than 35,000 psi (241 Mpa) or the wave equation analysis shall be permitted to be used to assess compression stresses induced by driving to evaluate if the pile section is appropriate for the selected hammer. Where pipe wall thickness less than 0.179 inch (4.6 mm) is driven open ended, a suitable cutting shoe shall be provided.~~

~~SECTION 1810~~
~~CAST-IN-PLACE CONCRETE PILE FOUNDATIONS~~

~~**1810.1 General.** The materials, reinforcement and installation of cast-in-place concrete piles shall conform to Sections 1810.1.1 through 1810.1.3.~~

~~**1810.1.1 Materials.** Concrete shall have a 28-day specified compressive strength (f'_c) of not less than 2,500 psi (17.24 MPa). Where concrete is placed through a funnel hopper at the top of the pile, the concrete mix shall be designed and proportioned so as to produce a cohesive workable mix having a slump of not less than 4 inches (102 mm) and not more than 6 inches (152 mm). Where concrete is to be pumped, the mix design including slump shall be adjusted to produce a pumpable concrete.~~

~~**1810.1.2 Reinforcement.** Except for steel dowels embedded 5 feet (1524 mm) or less in the pile and as provided in Section 1810.3.4, reinforcement where required shall be assembled and tied together and shall be placed in the pile as a unit before the reinforced portion of the pile is filled with concrete except in augered uncased cast-in-place piles. Tied reinforcement in augered uncased cast-in-place piles shall be placed after piles are concreted, while the concrete is still in a semifluid state.~~

~~**1810.1.2.1 Reinforcement in Seismic Design Category C.** Where a structure is assigned to Seismic Design Category C in accordance with Section 1613, the following shall apply. A minimum longitudinal reinforcement ratio of 0.0025 shall be provided for uncased cast-in-place concrete drilled or augered piles, piers or caissons in the top one-third of the pile length, a minimum length of 10 feet (3048 mm) below the ground or that required by analysis, whichever length is greatest. The minimum reinforcement ratio, but no less than that ratio required by rational analysis, shall be continued throughout the flexural length of the pile. There shall be a minimum of four longitudinal bars with closed ties (or equivalent spirals) of a minimum 3/8 inch (9 mm) diameter provided at 16-longitudinal-bar diameter maximum spacing. Transverse confinement reinforcement with a maximum spacing of 6 inches (152 mm) or 8-longitudinal-bar diameters, whichever is less, shall be provided within a distance equal to three times the least pile dimension of the bottom of the pile cap.~~

~~**1810.1.2.2 Reinforcement in Seismic Design Category D, E or F.** Where a structure is assigned to Seismic Design Category D, E or F in accordance with Section 1613, the requirements for Seismic Design Category C given above shall be met, in addition to the following. A minimum longitudinal reinforcement ratio of 0.005 shall be provided for uncased cast-in-place drilled or augered concrete piles, piers or caissons in the top one-half of the pile length a minimum length of 10 feet (3048 mm) below ground or throughout the flexural length of the pile, whichever length is greatest. The flexural length shall be taken as the length of the pile to a point where the concrete section cracking moment strength multiplied by 0.4 exceeds the required moment strength at that point. There shall be a minimum of four longitudinal bars with transverse confinement reinforcement provided in the pile in accordance with Sections~~

~~21.4.4.1, 21.4.4.2 and 21.4.4.3 of ACI 318 within three times the least pile dimension of the bottom of the pile cap. A transverse spiral reinforcement ratio of not less than one-half of that required in Section 21.4.4.1(a) of ACI 318 for other than Class E, F or liquefiable sites is permitted. Tie spacing throughout the remainder of the concrete section shall neither exceed 12-longitudinal-bar diameters, one-half the least dimension of the section, nor 12 inches (305 mm). Ties shall be a minimum of No. 3 bars for piles with a least dimension up to 20 inches (508 mm), and No. 4 bars for larger piles.~~

~~**1810.1.3 Concrete placement.** Concrete shall be placed in such a manner as to ensure the exclusion of any foreign matter and to secure a full-sized shaft. Concrete shall not be placed through water except where a tremie or other approved method is used. When depositing concrete from the top of the pile, the concrete shall not be chuted directly into the pile but shall be poured in a rapid and continuous operation through a funnel hopper centered at the top of the pile.~~

~~**1810.2 Enlarged base piles.** Enlarged base piles shall conform to the requirements of Sections 1810.2.1 through 1810.2.5.~~

~~**1810.2.1 Materials.** The maximum size for coarse aggregate for concrete shall be $^3/_4$ inch (19.1 mm). Concrete to be compacted shall have a zero slump.~~

~~**1810.2.2 Allowable stresses.** The maximum allowable design compressive stress for concrete not placed in a permanent steel casing shall be 25 percent of the 28-day specified compressive strength (f'_c). Where the concrete is place in a permanent steel casing, the maximum allowable concrete stress shall be 33 percent of the 28-day specified compressive strength (f'_c).~~

~~**1810.2.3 Installation.** Enlarged bases formed either by compacting concrete or driving a precast base shall be formed in or driven into granular soils. Piles shall be constructed in the same manner as successful prototype test piles driven for the project. Pile shafts extending through peat or other organic soil shall be encased in a permanent steel casing. Where a cased shaft is used, the shaft shall be adequately reinforced to resist column action or the annular space around the pile shaft shall be filled sufficiently to reestablish lateral support by the soil. Where pile heave occurs, the pile shall be replaced unless it is demonstrated that the pile is undamaged and capable of carrying twice its design load.~~

~~**1810.2.4 Load-bearing capacity.** Pile load-bearing capacity shall be verified by load tests in accordance with Section 1808.2.8.3.~~

~~**1810.2.5 Concrete cover.** The minimum concrete cover shall be $2^1/_2$ inches (64 mm) for uncased shafts and 1 inch (25 mm) for cased shafts.~~

~~**1810.3 Drilled or augered uncased piles.** Drilled or augered uncased piles shall conform to Sections 1810.3.1 through 1810.3.5.~~

~~**1810.3.1 Allowable stresses.** The allowable design stress in the concrete of drilled or augered uncased piles shall not exceed 33 percent of the 28-day specified compressive strength (f'_c). The allowable compressive stress of reinforcement shall not exceed 40 percent of the yield strength of the steel or 25,500 psi (175.8 MPa).~~

~~**1810.3.2 Dimensions.** The pile length shall not exceed 30 times the average diameter. The minimum diameter shall be 12 inches (305 mm).~~

> ~~**Exception:** The length of the pile is permitted to exceed 30 times the diameter, provided that the design and installation of the pile foundation are under the direct supervision of a registered design professional knowledgeable in the field of soil mechanics and pile foundations. The registered design professional shall certify to the building official that the piles were installed in compliance with the approved construction documents.~~

~~**1810.3.3 Installation.** Where pile shafts are formed through unstable soils and concrete is placed in an open-drilled hole, a steel liner shall be inserted in the hole prior to placing the concrete. Where the steel liner is withdrawn during concreting, the level of concrete shall be maintained above the bottom of the liner at a sufficient height to offset any hydrostatic or lateral soil pressure.~~

~~Where concrete is placed by pumping through a hollow- stem auger, the auger shall be permitted to rotate in a clockwise direction during withdrawal. The auger shall be withdrawn in continuous increments. Concreting pumping pressures shall be measured and maintained high enough at all times to offset hydrostatic and lateral earth pressures. Concrete volumes shall be measured to ensure that the volume of concrete placed in each pile is equal to or greater~~

~~than the theoretical volume of the hole created by the auger. Where the installation process of any pile is interrupted or a loss of concreting pressure occurs, the pile shall be redrilled to 5 feet (1524 mm) below the elevation of the tip of the auger when the installation was interrupted or concrete pressure was lost and reformed. Augered cast-in-place piles shall not be installed within six pile diameters center to center of a pile filled with concrete less than 12 hours old, unless approved by the building official. If the concrete level in any completed pile drops due to installation of an adjacent pile, the pile shall be replaced.~~

~~**1810.3.4 Reinforcement.** For piles installed with a hollow-stem auger where full-length longitudinal steel reinforcement is placed without lateral ties, the reinforcement shall be placed through the hollow stem of the auger prior to filling the pile with concrete. All pile reinforcement shall have a concrete cover of not less than 2.5 inches (64 mm).~~

> ~~**Exception:** Where physical constraints do not allow the placement of the longitudinal reinforcement prior to filling the pile with concrete or where partial-length longitudinal reinforcement is placed without lateral ties, the reinforcement is allowed to be placed after the piles are completely concreted but while concrete is still in a semifluid state.~~

~~**1810.3.5 Reinforcement in Seismic Design Category C, D, E or F.** Where a structure is assigned to Seismic Design Category C, D, E or F in accordance with Section 1613, the corresponding requirements of Sections 1810.1.2.1 and 1810.1.2.2 shall be met.~~

~~**1810.4 Driven uncased piles.** Driven uncased piles shall conform to Sections 1810.4.1 through 1810.4.4.~~

~~**1810.4.1 Allowable stresses.** The allowable design stress in the concrete shall not exceed 25 percent of the 28-day specified compressive strength (f_c) applied to a cross-sectional area not greater than the inside area of the drive casing or mandrel.~~

~~**1810.4.2 Dimensions.** The pile length shall not exceed 30 times the average diameter. The minimum diameter shall be 12 inches (305 mm).~~

> ~~**Exception:** The length of the pile is permitted to exceed 30 times the diameter, provided that the design and installation of the pile foundation is under the direct supervision of a registered design professional knowledgeable in the field of soil mechanics and pile foundations. The registered design professional shall certify to the building official that the piles were installed in compliance with the approved design.~~

~~**1810.4.3 Installation.** Piles shall not be driven within six pile diameters center to center in granular soils or within one-half the pile length in cohesive soils of a pile filled with concrete less than 48 hours old unless approved by the building official. If the concrete surface in any completed pile rises or drops, the pile shall be replaced. Piles shall not be installed in soils that could cause pile heave.~~

~~**1810.4.4 Concrete cover.** Pile reinforcement shall have a concrete cover of not less than 2.5 inches (64 mm), measured from the inside face of the drive casing or mandrel.~~

~~**1810.5 Steel-cased piles.** Steel-cased piles shall comply with the requirements of Sections 1810.5.1 through 1810.5.4.~~

~~**1810.5.1 Materials.** Pile shells or casings shall be of steel and shall be sufficiently strong to resist collapse and sufficiently water tight to exclude any foreign materials during the placing of concrete. Steel shells shall have a sealed tip with a diameter of not less than 8 inches (203 mm).~~

~~**1810.5.2 Allowable stresses.** The allowable design compressive stress in the concrete shall not exceed 33 percent of the 28-day specified compressive strength (f'_c). The allowable concrete compressive stress shall be 0.40 (f'_c) for that portion of the pile meeting the conditions specified in Sections 1810.5.2.1 through 1810.5.2.4.~~

~~**1810.5.2.1 Shell thickness.** The thickness of the steel shell shall not be less than manufacturer's standard gage No. 14 gage (0.068 inch) (1.75 mm) minimum.~~

~~**1810.5.2.2 Shell type.** The shell 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.~~

~~**1810.5.2.3 Strength.** The ratio of steel yield strength (f_y) to 28-day specified compressive strength (f'_c) shall not be less than six.~~

~~**1810.5.2.4 Diameter.** The nominal pile diameter shall not be greater than 16 inches (406 mm).~~

~~**1810.5.3 Installation.** Steel shells shall be mandrel driven their full length in contact with the surrounding soil. The steel shells shall be driven in such order and with such spacing as to ensure against distortion of or injury to piles already in place. A pile shall not be driven within four and one-half average pile diameters of a pile filled with concrete less than 24 hours old unless approved by the building official. Concrete shall not be placed in steel shells within heave range of driving.~~

~~**1810.5.4 Reinforcement.** Reinforcement shall not be placed within 1 inch (25 mm) of the steel shell. Reinforcing shall be required for unsupported pile lengths or where the pile is designed to resist uplift or unbalanced lateral loads.~~

~~**1810.5.4.1 Seismic reinforcement.** Where a structure is assigned to Seismic Design Category C, D, E or F in accordance with Section 1613, the reinforcement requirements for drilled or augered uncased piles in Section 1810.3.5 shall be met.~~

> ~~**Exception:** A spiral-welded metal casing of a thickness no less than the manufacturer's standard gage No. 14 gage [0.068 inch (1.7 mm)] is permitted to provide concrete confinement in lieu of the closed ties or equivalent spirals required in an uncased concrete pile. Where used as such, the metal casing shall be protected against possible deleterious action due to soil constituents, changing water levels or other factors indicated by boring records of site conditions.~~

~~**1810.6 Concrete-filled steel pipe and tube piles.** Concrete-filled steel pipe and tube piles shall conform to the requirements of Sections 1810.6.1 through 1810.6.5.~~

~~**1810.6.1 Materials.** Steel pipe and tube sections used for piles shall conform to ASTM A 252 or ASTM A 283. Concrete shall conform to Section 1810.1.1. The maximum coarse aggregate size shall be $^3/_4$ inch (19.1 mm).~~

~~**1810.6.2 Allowable stresses.** The allowable design compressive stress in the concrete shall not exceed 33 percent of the 28-day specified compressive strength (f'_c). The allowable design compressive stress in the steel shall not exceed 35 percent of the minimum specified yield strength of the steel (F_y), provided F_y shall not be assumed greater than 36,000 psi (248 MPa) for computational purposes.~~

> ~~**Exception:** Where justified in accordance with Section 1808.2.10, the allowable stresses are permitted to be increased to 0.50 F_y.~~

~~**1810.6.3 Minimum dimensions.** Piles shall have a nominal outside diameter of not less than 8 inches (203 mm) and a minimum wall thickness in accordance with Section 1809.3.4. For mandrel-driven pipe piles, the minimum wall thickness shall be $^1/_{10}$ inch (2.5 mm).~~

~~**1810.6.4 Reinforcement.** Reinforcement steel shall conform to Section 1810.1.2. Reinforcement shall not be placed within 1 inch (25 mm) of the steel casing.~~

~~**1810.6.4.1 Seismic reinforcement.** Where a structure is assigned to Seismic Design Category C, D, E or F in accordance with Section 1613, the following shall apply. Minimum reinforcement no less than 0.01 times the cross-sectional area of the pile concrete shall be provided in the top of the pile with a length equal to two times the required cap embedment anchorage into the pile cap, but not less than the tension development length of the reinforcement. The wall thickness of the steel pipe shall not be less than $^3/_{16}$ inch (5 mm).~~

~~**1810.6.5 Placing concrete.** The placement of concrete shall conform to Section 1810.1.3, but is permitted to be chuted directly into smooth-sided pipes and tubes without a centering funnel hopper.~~

~~**1810.7 Caisson piles.** Caisson piles shall conform to the requirements of Sections 1810.7.1 through 1810.7.6.~~

~~**1810.7.1 Construction.** Caisson piles shall consist of a shaft section of concrete-filled pipe extending to bedrock with an uncased socket drilled into the bedrock and filled with concrete. The caisson pile shall have a full-length structural steel core or a stub core installed in the rock socket and extending into the pipe portion a distance equal to the socket depth.~~

~~**1810.7.2 Materials.** Pipe and steel cores shall conform to the material requirements in Section 1809.3. Pipes shall have a minimum wall thickness of $^3/_8$ inch (9.5 mm) and shall be fitted with a suitable steel-driving shoe welded to the bottom of the pipe. Concrete shall have a 28-day specified compressive strength (f'_c) of not less than 4,000 psi (27.58 MPa). The concrete mix shall be designed and proportioned so as to produce a cohesive workable mix with a slump of 4 inches to 6 inches (102 mm to 152 mm).~~

~~**1810.7.3 Design.** The depth of the rock socket shall be sufficient to develop the full load-bearing capacity of the caisson pile with a minimum safety factor of two, but the depth shall not be less than the outside diameter of the pipe. The design of the rock socket is permitted to be predicated on the sum of the allowable load-bearing pressure on the bottom of the socket plus bond along the sides of the socket. The minimum outside diameter of the caisson pile shall be 18 inches (457 mm), and the diameter of the rock socket shall be approximately equal to the inside diameter of the pile.~~

~~**1810.7.4 Structural core.** The gross cross-sectional area of the structural steel core shall not exceed 25 percent of the gross area of the caisson. The minimum clearance between the structural core and the pipe shall be 2 inches (51 mm). Where cores are to be spliced, the ends shall be milled or ground to provide full contact and shall be full-depth welded.~~

~~**1810.7.5 Allowable stresses.** The allowable design compressive stresses shall not exceed the following: concrete, 0.33 f'_c; steel pipe, 0.35 F_y and structural steel core, 0.50 F_y.~~

~~**1810.7.6 Installation.** The rock socket and pile shall be thoroughly cleaned of foreign materials before filling with concrete. Steel cores shall be bedded in cement grout at the base of the rock socket. Concrete shall not be placed through water except where a tremie or other approved method is used.~~

~~**1810.8 (Supp) Micropiles.** Micropiles shall comply with the requirements of Sections 1810.8.1 through 1810.8.5.~~

~~**1810.8.1 (Supp)Construction.** Micropiles shall consist of a grouted section reinforced with steel pipe or steel reinforcement. Micropiles shall develop their load-carrying capacity through a bond zone in soil, bedrock or a combination of soil and bedrock. The steel pipe or steel reinforcement shall extend the full length of the micropile.~~

~~**1810.8.2 (Supp) Materials.** Grout shall have a specified compressive strength (f'_c) of not less than 4,000 psi (27.58 Mpa). The grout mix shall be designed and proportioned so as to produce a pumpable mixture. Reinforcement shall consist of deformed reinforcing bars in accordance with ASTM A 615 Grade 60 or Grade 75 or ASTM A 722 Grade 150. The steel pipe shall have a minimum wall thickness of 3/16 inch (4.8 mm). Splices shall comply with Section 1808.2.7. The steel pipe shall have a minimum yield strength exceeding 45,000 p.s.i. (310 MPa) and a minimum elongation of 15 percent as shown by mill certifications or two coupon test samples per 40, 000 pounds (18 160 kg) of pipe.~~

~~**1810.8.3 (Supp) Allowable stresses.** The allowable compressive stress in the grout shall not exceed 0.33 f'_c. The allowable compressive stress in the steel pipe and steel reinforcement shall not exceed the lesser of 0.4 F_y, and 32,000 psi (220 Mpa). The allowable tensile stress in the steel reinforcement shall not exceed 0.60 F_y. The allowable tensile stress in the cement grout shall be zero.~~

~~**1810.8.4 (Supp) Reinforcement.** For piles or portions of piles grouted inside a temporary or permanent casing or inside a hole drilled into bedrock or a hole drilled with grout, the steel pipe or steel reinforcement shall be designed to carry at least 40 percent of the design compression load. Piles or portions of piles grouted in an open hole in soil without temporary or permanent casing and without suitable means of verifying the hole diameter during grouting shall be designed to carry the entire compression load in the reinforcing steel. Where a steel pipe is used for reinforcement, the portion of the grout enclosed within the pipe is permitted to be included in the determination of the allowable stress in the grout.~~

~~**1810.8.4.1 (Supp) Seismic reinforcement.** Where a structure is assigned to Seismic Design Category C, a permanent steel casing shall be provided from the top of the pile down a minimum of 120 percent of the flexural length. Where a structure is assigned to Seismic Design D, E or F, the pile shall be considered as an alternative system in accordance with Section 104.11. The alternative pile system design, supporting documentation and test data shall be submitted to the building official for review and approval.~~

~~**1810.8.5 (Supp) Installation.** The pile shall be permitted to be formed in a hole advanced by rotary or percussive drilling methods, with or without casing. The pile shall be grouted with a fluid cement grout. The grout shall be pumped through a tremie pipe extending to the bottom of the pile until grout of suitable quality returns at the top of the pile. The following requirements apply to specific installation methods:~~

1. ~~For piles grouted inside a temporary casing, the reinforcing bars shall be inserted prior to withdrawal of the casing. The casing shall be withdrawn in a controlled manner with the grout level maintained at the top of the pile to ensure that the grout completely fills the drill hole. During withdrawal of the casing, the grout level inside the casing shall be monitored to check that the flow of grout inside the casing is not obstructed.~~

2. ~~For a pile or portion of a pile grouted in an open drill hole in soil without temporary casing, the minimum design diameter of the drill hole shall be verified by a suitable device during grouting.~~
3. ~~For piles designed for end bearing, a suitable means shall be employed to verify that the bearing surface is properly cleaned prior to grouting.~~
4. ~~Subsequent piles shall not be drilled near piles that have been grouted until the grout has had sufficient time to harden.~~
5. ~~Piles shall be grouted as soon as possible after drilling is completed.~~
6. ~~For piles designed with a full length casing, the casing shall be pulled back to the top of the bond zone and reinserted or some other suitable means shall be employed to ensure grout coverage outside the casing.~~

~~SECTION 1811~~
~~COMPOSITE PILES~~

~~**1811.1 General.** Composite piles shall conform to the requirements of Sections 1811.2 through 1811.5.~~

~~**1811.2 Design.** Composite piles consisting of two or more approved pile types~~

~~**1811.3 Limitation of load.** The maximum allowable load shall be limited by the capacity of the weakest section incorporated in the pile.~~

~~**1811.4 Splices.** Splices between concrete and steel or wood sections shall be designed to prevent separation both before and after the concrete portion has set, and to ensure the alignment and transmission of the total pile load. Splices shall be designed to resist uplift caused by upheaval during driving of adjacent piles, and shall develop the full compressive strength and not less than 50 percent of the tension and bending strength of the weaker section.~~

~~**1811.5 Seismic reinforcement.** Where a structure is assigned to Seismic Design Category C, D, E or F in accordance with Section 1613, the following shall apply. Where concrete and steel are used as part of the pile assembly, the concrete reinforcement shall comply with that given in Sections 1810.1.2.1 and 1810.1.2.2 or the steel section shall comply with Section 1810.6.4.1.~~

~~SECTION 1812~~
~~PIER FOUNDATIONS~~

~~**1812.1 General.** Isolated and multiple piers used as foundations shall conform to the requirements of Sections 1812.2 through 1812.10, as well as the applicable provisions of Section 1808.2.~~

~~**1812.2 Lateral dimensions and height.** The minimum dimension of isolated piers used as foundations shall be 2 feet (610 mm), and the height shall not exceed 12 times the least horizontal dimension.~~

~~**1812.3 Materials.** Concrete shall have a 28-day specified compressive strength~~ ($\cancel{f'_c}$) ~~of not less than 2,500 psi (17.24 MPa). Where concrete is placed through a funnel hopper at the top of the pier, the concrete mix shall be designed and proportioned so as to produce a cohesive workable mix having a slump of not less than 4 inches (102 mm) and not more than 6 inches (152 mm). Where concrete is to be pumped, the mix design including slump shall be adjusted to produce a pumpable concrete.~~

~~**1812.4 Reinforcement.** Except for steel dowels embedded 5 feet (1524 mm) or less in the pier, reinforcement where required shall be assembled and tied together and shall be placed in the pier hole as a unit before the reinforced portion of the pier is filled with concrete.~~

> ~~**Exception:** Reinforcement is permitted to be wet set and the~~ $\cancel{2^1/_2}$~~- inch (64 mm) concrete cover requirement be reduced to 2 inches (51 mm) for Group R-3 and U occupancies not exceeding two stories of light-frame construction, provided the construction method can be demonstrated to the satisfaction of the building official.~~

~~Reinforcement shall conform to the requirements of Sections 1810.1.2.1 and 1810.1.2.2.~~

~~Exceptions:~~

1. ~~Isolated piers supporting posts of Group R-3 and U occupancies not exceeding two stories of light-frame construction are permitted to be reinforced as required by rational analysis but not less than a minimum of one No. 4 bar, without ties or spirals, when detailed so the pier is not subject to lateral loads and the soil is determined to be of adequate stiffness.~~

2. ~~Isolated piers supporting posts and bracing from decks and patios appurtenant to Group R-3 and U occupancies not exceeding two stories of light-frame construction are permitted to be reinforced as required by rational analysis but not less than one No. 4 bar, without ties or spirals, when the lateral load, *E*, to the top of the pier does not exceed 200 pounds (890 N) and the soil is determined to be of adequate stiffness.~~
3. ~~Piers supporting the concrete foundation wall of Group R-3 and U occupancies not exceeding two stories of light-frame construction are permitted to be reinforced as required by rational analysis but not less than two No. 4 bars, without ties or spirals, when it can be shown the concrete pier will not rupture when designed for the maximum seismic load, E_m, and the soil is determined to be of adequate stiffness.~~
4. ~~Closed ties or spirals where required by Section 1810.1.2.2 are permitted to be limited to the top 3 feet (914 mm) of the piers 10 feet (3048 mm) or less in depth supporting Group R-3 and U occupancies of Seismic Design Category D, not exceeding two stories of light-frame construction.~~

~~**1812.5 Concrete placement.** Concrete shall be placed in such a manner as to ensure the exclusion of any foreign matter and to secure a full-sized shaft. Concrete shall not be placed through water except where a tremie or other approved method is used. When depositing concrete from the top of the pier, the concrete shall not be chuted directly into the pier but shall be poured in a rapid and continuous operation through a funnel hopper centered at the top of the pier.~~

~~**1812.6 Belled bottoms.** Where pier foundations are belled at the bottom, the edge thickness of the bell shall not be less than that required for the edge of footings. Where the sides of the bell slope at an angle less than 60 degrees (1 rad) from the horizontal, the effects of vertical shear shall be considered.~~

~~**1812.7 Masonry.** Where the unsupported height of foundation piers exceeds six times the least dimension, the allowable working stress on piers of unit masonry shall be reduced in accordance with ACI 530/ASCE 5/TMS 402.~~

~~**1812.8 Concrete.** Where adequate lateral support is not provided, and the unsupported height to least lateral dimension does not exceed three, piers of plain concrete shall be designed and constructed as pilasters in accordance with ACI 318. Where the unsupported height to least lateral dimension exceeds three, piers shall be constructed of reinforced concrete, and shall conform to the requirements for columns in ACI 318.~~

> ~~**Exception:** Where adequate lateral support is furnished by the surrounding materials as defined in Section 1808.2.9, piers are permitted to be constructed of plain or reinforced concrete. The requirements of ACI 318 for bearing on concrete shall apply.~~

~~**1812.9 Steel shell.** Where concrete piers are entirely encased with a circular steel shell, and the area of the shell steel is considered reinforcing steel, the steel shall be protected under the conditions specified in Section 1808.2.17. Horizontal joints in the shell shall be spliced to comply with Section 1808.2.7.~~

~~**1812.10 Dewatering.** Where piers are carried to depths below water level, the piers shall be constructed by a method that will provide accurate preparation and inspection of the bottom, and the depositing or construction of sound concrete or other masonry in the dry.~~

SECTION 1810
DEEP FOUNDATIONS

1810.1 General. Deep foundations shall be analyzed, designed, detailed, and installed in accordance with Sections 1810.1 through 1810.4.

1810.1.1 Geotechnical investigation. Deep foundations shall be designed and installed on the basis of a geotechnical investigation as set forth in Section 1803,

1810.1.2 Use of existing deep foundation elements. Deep foundation elements left in place where a structure has been demolished shall not be used for the support of new construction unless satisfactory evidence is submitted to the building official, which indicates that the elements are sound and meet the requirements of this code. Such elements shall be load tested or redriven to verify their capacities. The design load applied to such elements shall be the lowest allowable load as determined by tests or redriving data.

1810.1.3 Deep foundation elements classified as columns. Deep foundation elements standing unbraced in air, water, or fluid soils shall be classified as columns and designed as such in accordance with the provisions of this code from their top down to the point where adequate lateral support is provided in accordance with Section 1810.2.1.

Exception: Where the unsupported height to least horizontal dimension of a cast-in-place deep foundation element does not exceed three, it shall be permitted to design and construct such an element as a pedestal in accordance with ACI 318.

1810.1.4 Special types of deep foundations. The use of types of deep foundation elements not specifically mentioned herein is permitted, subject to the approval of the building official, upon the submission of acceptable test data, calculations and other information relating to the structural properties and load capacity of such elements. The allowable stresses for materials shall not in any case exceed the limitations specified herein.

1810.2 Analysis. The analysis of deep foundations for design shall be in accordance with Sections 1810.2.1 through 1810.2.5.

1810.2.1 Lateral support. Any soil other than fluid soil shall be deemed to afford sufficient lateral support to prevent buckling of deep foundation elements and to permit the design of the elements in accordance with accepted engineering practice and the applicable provisions of this code.

Where deep foundation elements stand unbraced in air, water, or fluid soils, it shall be permitted to consider them fixed and laterally supported at a point 5 feet (1524 mm) into stiff soil or 10 feet (3048 mm) into soft soil unless otherwise prescribed by the building official after a foundation investigation by an approved agency.

1810.2.2 Stability. Deep foundation elements shall be braced to provide lateral stability in all directions. Three or more elements connected by a rigid cap shall be considered braced, provided that the elements are located in radial directions from the centroid of the group not less than 60 degrees (1 rad) apart. A two-element group in a rigid cap shall be considered to be braced along the axis connecting the two elements. Methods used to brace deep foundation elements shall be subject to the approval of the building official.

Deep foundation elements supporting walls shall be placed alternately in lines spaced at least 1 foot (305 mm) apart and located symmetrically under the center of gravity of the wall load carried, unless effective measures are taken to provide for eccentricity and lateral forces, or the foundation elements are adequately braced to provide for lateral stability.

Exceptions:

1. Isolated cast-in-place deep foundation elements without lateral bracing shall be permitted where the least horizontal dimension is no less than 2 feet (610 mm), adequate lateral support in accordance with Section 1810.2.1 is provided for the entire height and the height does not exceed 12 times the least horizontal dimension.
2. A single row of deep foundation elements without lateral bracing is permitted for one- and two-family dwellings and lightweight construction not exceeding two stories above grade plane or 35 feet (10 668 mm) in building height, provided the centers of the elements are located within the width of the supported wall.

1810.2.3 Settlement. The settlement of a single deep foundation elements or group thereof shall be estimated based on approved methods of analysis. The predicted settlement shall cause neither harmful distortion of, nor instability in, the structure, nor cause any stresses to exceed allowable values.

1810.2.4 Lateral loads. The moments, shears and lateral deflections used for design of deep foundation elements shall be established considering the nonlinear interaction of the shaft and soil, as determined by a registered design professional. Where the ratio of the depth of embedment of the element to its least horizontal dimension is less than or equal to six, it shall be permitted to assume the element is rigid.

1810.2.4.1 Seismic Design Categories D through F. For structures assigned to Seismic Design Category D, E, or F, deep foundation elements on Site Class E or F sites, as determined in Section 1613.5.2, shall be designed and constructed to withstand maximum imposed curvatures from earthquake ground motions and structure 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.

Exception: Deep foundation elements that satisfy the following additional detailing requirements shall be deemed to comply with the curvature capacity requirements of this section.

1. Precast prestressed concrete piles detailed in accordance with Section 1810.3.8.3.3.
2. Cast-in-place deep foundation elements with a minimum longitudinal reinforcement ratio of 0.005 extending the full length of the element and detailed in accordance with Sections 21.4.4.1, 21.4.4.2 and 21.4.4.3 of ACI 318 as required by Section 1810.3.9.4.2.2.

1810.2.5 Group effects. The analysis shall include group effects on lateral behavior where the center-to-center spacing of deep foundation elements in the direction of lateral force is less than eight times the least horizontal dimension of an element. The analysis shall include group effects on axial behavior where the center-to-center spacing of deep foundation elements is less than three times the least horizontal dimension of an element.

1810.3 Design and detailing. Deep foundations shall be designed and detailed in accordance with Sections 1810.3.1 through 1810.3.12.

1810.3.1 Design conditions. Design of deep foundations shall include the design conditions specified in Sections 1810.3.1.1 through 1810.3.1.5, as applicable.

1810.3.1.1 Design methods for concrete elements. Where concrete deep foundations are laterally supported in accordance with Section 1810.2.1 for the entire height and applied forces cause bending moments no greater than those resulting from accidental eccentricities, structural design of the element using the load combinations of Section 1605.3 and the allowable stresses specified in this chapter shall be permitted. Otherwise, the structural design of concrete deep foundation elements shall use the load combinations of Section 1605.2 and approved strength design methods.

1810.3.1.2 Composite elements. Where a single deep foundation element comprises two or more sections of different materials or different types spliced together, each section of the composite assembly shall satisfy the applicable requirements of this code, and the maximum allowable load shall be limited by the capacity of the weakest section.

1810.3.1.3 Mislocation. To resist the effects of mislocation, compressive overload of deep foundation elements to 110 percent of the allowable design load shall be permitted.

1810.3.1.4 Driven piles. Driven piles shall be designed and manufactured in accordance with accepted engineering practice to resist all stresses induced by handling, driving and service loads.

1810.3.1.5 Casings. Temporary and permanent casings shall be of steel and shall be sufficiently strong to resist collapse and sufficiently water tight to exclude any foreign materials during the placing of concrete.
Where a permanent casing is considered reinforcing steel, the steel shall be protected under the conditions specified in Section 1810.3.2.5. Horizontal joints in the casing shall be spliced in accordance with Section 1810.3.6.

1810.3.2 Materials. The materials used in deep foundations elements shall satisfy the requirements of Sections 1810.3.2.1 through 1810.3.2.8, as applicable.

1810.3.2.1 Concrete. Where concrete is cast in a steel pipe or where an enlarged base is formed by compacting concrete, the maximum size for coarse aggregate shall be 3/4 inch (19.1 mm). Concrete to be compacted shall have a zero slump.

1810.3.2.1.1 Seismic hooks. For structures assigned to Seismic Design Category C, D, E, or F in accordance with Section 1613, the ends of hoops, spirals and ties used in concrete deep foundation elements shall be terminated with seismic hooks, as defined in Section 21.1 of ACI 318, and shall be turned into the confined concrete core.

1810.3.2.2 Prestressing steel. Prestressing steel shall conform to ASTM A 416.

1810.3.2.3 Structural steel. Structural steel piles, steel pipe and fully welded steel piles fabricated from plates shall conform to ASTM A 36, ASTM A 252, ASTM A 283, ASTM A 572, ASTM A 588, ASTM A 690, ASTM A 913 or ASTM A 992.

1810.3.2.4 Timber. Timber deep foundation elements shall be designed as piles or poles in accordance with the AF&PA NDS.
Round timber elements shall conform to ASTM D 25. Sawn timber elements shall conform to DOC PS-20.

1810.3.2.4.1 Preservative treatment. Timber deep foundation elements used to support permanent structures shall be treated in accordance with this section unless it is established that the tops of the untreated timber elements will be below the lowest ground-water level assumed to exist during the life of the structure. Preservative and minimum final retention shall be in accordance with AWPA U1 (Commodity Specification E, Use Category 4C) for round timber elements and AWPA U1 (Commodity Specification A, Use Category 4B) for sawn timber elements . Preservative-treated timber elements shall be subject to a quality control program administered by an approved agency. Element cutoffs shall be treated in accordance with AWPA M4.

1810.3.2.5 Protection of materials. Where boring records or site conditions indicate possible deleterious action on the materials used in deep foundation elements because of soil constituents, changing water levels or other factors, the elements shall be adequately protected by materials, methods or processes approved by the building official. Protective materials shall be applied to the elements so as not to be rendered ineffective by installation. The effectiveness of such protective measures for the particular purpose shall have been thoroughly established by satisfactory service records or other evidence.

1810.3.2.6 Allowable stresses. The allowable stresses for materials used in deep foundation elements shall not exceed those specified in Table 1810.3.2.6.

TABLE 1810.3.2.6
ALLOWABLE STRESSES FOR MATERIALS USED IN DEEP FOUNDATION ELEMENTS

MATERIAL TYPE AND CONDITION	MAXIMUM ALLOWABLE STRESS [a]
1. Concrete or grout in compression [b]	
Cast-in-place with a permanent casing in accordance with Section 1810.3.2.7	$0.4\ f'_c$
Cast-in-place in a pipe, tube, or other permanent casing	$0.33\ f'_c$
Cast-in-place without a permanent casing	$0.3\ f'_c$
Precast nonprestressed	$0.33\ f'_c$
Precast prestressed	$0.33\ f'_c - 0.27\ f_{pc}$
2. Nonprestressed reinforcement in compression	$0.4\ f_y \le 30{,}000$ psi
3. Structural steel in compression	
Cores within concrete-filled pipes or tubes	$0.5\ F_y \le 32{,}000$ psi
Pipes, tubes, or H-piles, where justified in accordance with Section 1810.3.2.8	$0.5\ F_y \le 32{,}000$ psi
Pipes or tubes for micropiles	$0.4\ F_y \le 32{,}000$ psi
Other pipes, tubes, or H-piles	$0.35\ F_y \le 16{,}000$ psi
5. Nonprestressed reinforcement in tension	
Within micropiles	$0.6\ f_y$
Other conditions	$0.5\ f_y \le 24{,}000$ psi
6. Structural steel in tension	
Pipes, tubes, or H-piles, where justified in accordance with Section 1810.3.2.8	$0.5\ F_y \le 32{,}000$ psi
Other pipes, tubes, or H-piles	$0.35\ F_y \le 16{,}000$ psi
7. Timber	In accordance with the AF&PA NDS

a. f'_c is the specified compressive strength of the concrete or grout. f_{pc} is the compressive stress on the gross concrete section due to effective prestress forces only. f_y is the specified yield strength of reinforcement. F_y is the specified minimum yield stress of structural steel.
b. The stresses specified apply to the gross cross-sectional area within the concrete surface. Where a temporary or permanent casing is used, the inside face of the casing shall be considered the concrete surface.

1810.3.2.7 Increased allowable compressive stress for cased cast-in-place elements. The allowable compressive stress in the concrete shall be permitted to be increased as specified in Table 1810.3.2.6 for those portions of permanently cased cast-in-place elements that satisfy the following conditions:

1. The design shall not use the casing to resist any portion of the axial load imposed.
2. The casing shall have a sealed tip and be mandrel driven.
3. The thickness of the casing shall not be less than manufacturer's standard gage No. 14 (0.068 inch) (1.75 mm).
4. The 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.
5. The ratio of steel yield strength (F_y) to specified compressive strength (f'_c) shall not be less than six.
6. The nominal diameter of the element shall not be greater than 16 inches (406 mm).

1810.3.2.8 Justification of higher allowable stresses. Use of allowable stresses greater than those specified in Section 1810.3.2.6 shall be permitted where supporting data justifying such higher stresses is filed with the building official. Such substantiating data shall include:

1. A soils investigation in accordance with Section 1802; and
2. Load tests in accordance with Section 1810.3.3.1.2, regardless of the load supported by the element.

The design and installation of the deep foundation elements shall be under the direct supervision of a registered design professional knowledgeable in the field of soil mechanics and deep foundations who shall certify to the building official that the elements as installed satisfy the design criteria.

1810.3.3 Determination of allowable loads. The allowable axial and lateral loads on deep foundation elements shall be determined by an approved formula, load tests or method of analysis.

1810.3.3.1 Allowable axial load. The allowable axial load on a deep foundation element shall be determined in accordance with Section 1810.3.3.1.

1810.3.3.1.1 Driving criteria. The allowable compressive load on any driven deep foundation element where determined by the application of an approved driving formula shall not exceed 40 tons (356 kN). For allowable loads above 40 tons (356 kN), the wave equation method of analysis shall be used to estimate driveability for both driving stresses and net displacement per blow at the ultimate load. Allowable loads shall be verified by load tests in accordance with Section 1810.3.3.1.2. The formula or wave equation load shall be determined for gravity-drop or power-actuated hammers and the hammer energy used shall be the maximum consistent with the size, strength and weight of the driven elements. The use of a follower is permitted only with the approval of the building official. The introduction of fresh hammer cushion or pile cushion material just prior to final penetration is not permitted.

1810.3.3.1.2 Load tests. Where design compressive loads are greater than those determined using the allowable stresses specified in Section 1810.3.2.6, where the design load for any deep foundation element is in doubt, or where cast-in-place deep foundation elements have an enlarged base formed either by compacting concrete or by driving a precast base, control test elements shall be tested in accordance with ASTM D 1143 or ASTM D 4945. At least one element shall be load tested in each area of uniform subsoil conditions. Where required by the building official, additional elements shall be load tested where necessary to establish the safe design capacity. The resulting allowable loads shall not be more than one-half of the ultimate axial load capacity of the test element as assessed by one of the published methods listed in Section 1810.3.3.1.3 with consideration for the test type, duration and subsoil. The ultimate axial load capacity shall be determined by a registered design professional with consideration given to tolerable total and differential settlements at design load in accordance with Section 1810.2.3. In subsequent installation of the balance of deep foundation elements, all elements shall be deemed to have a supporting capacity equal to that of the control element where such elements are of the same type, size and relative length as the test element; are installed using the same or comparable methods and equipment as the test element; are installed in similar subsoil conditions as the test element; and, for driven elements, where the rate of penetration (e.g., net displacement per blow) of such elements is equal to or less than that of the test element driven with the same hammer through a comparable driving distance.

1810.3.3.1.3 Load test evaluation methods. It shall be permitted to evaluate load tests of deep foundation elements using any of the following methods:

1. Davisson Offset Limit.
2. Brinch-Hansen 90% Criterion.
3. Butler-Hoy Criterion.
4. Other methods approved by the building official.

1810.3.3.1.4 Allowable frictional resistance. The assumed frictional resistance developed by any uncased cast-in-place deep foundation element shall not exceed one-sixth of the bearing value of the soil material at minimum depth as set forth in Table 1804.2, up to a maximum of 500 psf (24 kPa), unless a greater value is allowed by the building official on the basis of a soil investigation as specified in Section 1802 is submitted or a greater value is substantiated by a load test in accordance with Section 1810.3.3.1.2. Frictional resistance and bearing resistance shall not be assumed to act simultaneously unless recommended by a soil investigation as specified in Section 1802.

1810.3.3.1.5 Uplift capacity of a single deep foundation element. Where required by the design, the uplift capacity of a single deep foundation element shall be determined by an approved method of analysis based on a minimum factor of safety of three or by load tests conducted in accordance with ASTM D 3689. The maximum allowable uplift load shall not exceed the ultimate load capacity as determined in Section 1810.3.3.1.2 divided by a factor of safety of two.

1810.3.3.1.6 Uplift capacity of grouped deep foundation elements. For grouped deep foundation elements subjected to uplift, the allowable working uplift load for the group shall be the lesser of:

1. The proposed individual uplift working load times the number of elements in the group.
2. Two-thirds of the effective weight of the group and the soil contained within a block defined by the perimeter of the group and the length of the element.

1810.3.3.1.7 Load-bearing capacity. Deep foundation elements shall develop ultimate load capacities of at least twice the design working loads in the designated load-bearing layers. Analysis shall show that no soil layer underlying the designated load-bearing layers causes the load-bearing capacity safety factor to be less than two.

1810.3.3.1.8 Bent deep foundation elements. The load-bearing capacity of deep foundation elements discovered to have a sharp or sweeping bend shall be determined by an approved method of analysis or by load testing a representative element.

1810.3.3.2 Allowable lateral load. Where required by the design, the lateral load capacity of a single deep foundation element or a group thereof shall be determined by an approved method of analysis or by lateral load tests to at least twice the proposed design working load. The resulting allowable load shall not be more than one-half of that test load that produces a gross lateral movement of 1 inch (25 mm) at the ground surface.

1810.3.4 Subsiding soils. Where deep foundation elements are installed through subsiding fills or other subsiding strata and derive support from underlying firmer materials, consideration shall be given to the downward frictional forces that may be imposed on the elements by the subsiding upper strata.

Where the influence of subsiding fills is considered as imposing loads on the element, the allowable stresses specified in this chapter shall be permitted to be increased where satisfactory substantiating data are submitted.

1810.3.5 Dimensions of deep foundation elements. The dimensions of deep foundation elements shall be in accordance with Sections 1810.3.5.1 through 1810.3.5.3, as applicable.

1810.3.5.1 Precast. The minimum lateral dimension of precast concrete deep foundation elements shall be 8 inches (203 mm). Corners of square elements shall be chamfered.

1810.3.5.2 Cast-in-place or grouted-in-place. Cast-in-place and grouted-in-place deep foundation elements shall satisfy the requirements of this section.

1810.3.5.2.1 Cased. Cast-in-place deep foundation elements with a permanent casing shall have a nominal outside diameter of not less than 8 inches (203 mm).

1810.3.5.2.2 Uncased. Cast-in-place deep foundation elements without a permanent casing shall have a diameter of not less than 12 inches (305 mm). The element length shall not exceed 30 times the average diameter.

> **Exception:** The length of the element is permitted to exceed 30 times the diameter, provided the design and installation of the deep foundations are under the direct supervision of a registered design professional knowledgeable in the field of soil mechanics and deep foundations. The registered design professional shall certify to the building official that the elements were installed in compliance with the approved construction documents.

1810.3.5.2.3 Micropiles. Micropiles shall have an outside diameter of 12 inches (305 mm) or less. There is no minimum diameter for micropiles.

1810.3.5.3 Steel. Steel deep foundation elements shall satisfy the requirements of this section.

1810.3.5.3.1 H-piles. Sections of H-piles shall comply with the following:

1. The flange projections shall not exceed 14 times the minimum thickness of metal in either the flange or the web and the flange widths shall not be less than 80 percent of the depth of the section.
2. The nominal depth in the direction of the web shall not be less than 8 inches (203 mm).
3. Flanges and web shall have a minimum nominal thickness of $^3/_8$ inch (9.5 mm).

1810.3.5.3.2 Steel pipes and tubes. Steel pipes and tubes used as deep foundation elements shall have a nominal outside diameter of not less than 8 inches (203 mm). Where steel pipes or tubes are driven open-ended, they shall have a minimum of 0.34 square inch (219 mm^2) of steel in cross section to resist each 1,000 foot-pounds (1356 Nm) of pile hammer energy, or shall have the equivalent strength for steels having a yield strength greater than 35,000 psi (241 MPa) or the wave equation analysis shall be permitted to be used to assess compression stresses induced by driving to evaluate if the pile section is appropriate for the selected hammer. Where a pipe or tube with wall thickness less than 0.179 inch (4.6 mm) is driven open ended, a suitable cutting shoe shall be provided. Concrete filled steel pipes or tubes in structures assigned to Seismic Design Category C, D, E, or F shall have a wall thickness of not less than $^3/_{16}$ inch (5 mm). The pipe or tube casing for socketed drilled shafts shall have a nominal outside diameter of not less than 18 inches (457 mm), a wall thickness of not less than $^3/_8$ inch (9.5 mm), and a suitable steel driving shoe welded to the bottom; the diameter of the rock socket shall be approximately equal to the inside diameter of the casing.

Exceptions:

1. There is no minimum diameter for steel pipes or tubes used in micropiles.
2. For mandrel-driven pipes or tubes, the minimum wall thickness shall be $^1/_{10}$ inch (2.5 mm).

1810.3.6 Splices. Splices shall be constructed so as to provide and maintain true alignment and position of the component parts of the deep foundation element during installation and subsequent thereto and shall be of adequate strength to transmit the vertical and lateral loads and moments occurring at the location of the splice during driving and under service loading. Where deep foundation elements of the same type are being spliced, splices shall develop not less than 50 percent of the bending strength of the weaker section. Where deep foundation elements of different materials or different types are being spliced, splices shall develop the full compressive strength and not less than 50 percent of the tension and bending strength of the weaker section. Where structural steel cores are to be spliced, the ends shall be milled or ground to provide full contact and shall be full-depth welded.

Splices occurring in the upper 10 feet (3048 mm) of the embedded portion of an element shall be capable of resisting at allowable working stresses the moment and shear that would result from an assumed eccentricity of the axial load of 3 inches (76 mm), or the element shall be braced in accordance with Section 1810.2.2 to other deep foundation elements that do not have splices in the upper 10 feet (3048 mm) of embedment.

1810.3.6.1 Seismic Design Categories C through F. For structures assigned to Seismic Design Category C, D, E, or F, splices of deep foundation elements shall develop the lesser of the following:

1. The full strength of the deep foundation element; and
2. The axial and shear forces and moments from the load combinations with overstrength factor in Section 12.4.3.2 of ASCE 7.

1810.3.7 Top of pile detailing at cutoffs. Where a minimum length for reinforcement or the extent of closely spaced confinement reinforcement is specified at the top of a deep foundation element, provisions shall be made so that those specified lengths or extents are maintained after cutoff.

1810.3.8 Precast concrete piles. Precast concrete piles shall be designed and detailed in accordance with Sections 1810.3.8.1 through 1810.3.8.3.

1810.3.8.1 Reinforcement. Longitudinal steel shall be arranged in a symmetrical pattern and be laterally tied with steel ties or wire spiral spaced center to center as follows:

1. At not more than 1 inch (25 mm) for the first five ties or spirals at each end; then
2. At not more than 4 inches (102 mm) for the remainder of the first 2 feet (610 mm) from each end; and then
3. At not more than 6 inches (152 mm) elsewhere.

The size of ties and spirals shall be as follows:

1. For piles having a least horizontal dimension of 16 inches (406 mm) or less, wire shall not be smaller than 0.22 inch (5.6 mm) (No. 5 gage).
2. For piles having a least horizontal dimension of more than 16 inches (406 mm) and less than 20 inches (508 mm), wire shall not be smaller than 0.238 inch (6 mm) (No. 4 gage).
3. For piles having a least horizontal dimension of 20 inches (508 mm) and larger, wire shall not be smaller than 0.25 inch (6.4 mm) round or 0.259 inch (6.6 mm) (No. 3 gage).

1810.3.8.2 Precast nonprestressed piles. Precast nonprestressed concrete piles shall comply with the requirements of Sections 1810.3.8.2.1 through 1810.3.8.2.3.

1810.3.8.2.1 Minimum reinforcement. Longitudinal reinforcement shall consist of at least four bars with a minimum longitudinal reinforcement ratio of 0.008.

1810.3.8.2.2 Seismic reinforcement in Seismic Design Categories C through F. For structures assigned to Seismic Design Category C, D, E, or F in accordance with Section 1613, precast nonprestressed piles shall be reinforced as specified in this section. The minimum longitudinal reinforcement ratio shall be 0.01 throughout the length. Transverse reinforcement shall consist of closed ties or spirals with a minimum 3/8 inch (9.5 mm) diameter. Spacing of transverse reinforcement shall not exceed the smaller of eight times the diameter of the smallest longitudinal bar or 6 inches (152 mm) within a distance of three times the least pile dimension from the bottom of the pile cap. Spacing of transverse reinforcement shall not exceed 6 inches (152 mm) throughout the remainder of the pile.

1810.3.8.2.3 Additional seismic reinforcement in Seismic Design Categories D through F. For structures assigned to Seismic Design Category D, E or F in accordance with Section 1613, transverse reinforcement shall be in accordance with Section 1810.3.9.4.2.

1810.3.8.3 Precast prestressed piles. Precast prestressed concrete piles shall comply with the requirements of Sections 1810.3.8.3.1 through 1810.3.8.3.3.

1810.3.8.3.1 Effective prestress. The effective prestress in the pile shall not be less than 400 psi (2.76MPa) for piles up to 30 feet (9144 mm) in length, 550 psi (3.79 MPa) for piles up to 50 feet (15 240 mm) in length and 700 psi (4.83 MPa) for piles greater than 50 feet (15 240 mm) in length.

Effective prestress shall be based on an assumed loss of 30,000 psi (207 MPa) in the prestressing steel. The tensile stress in the prestressing steel shall not exceed the values specified in ACI 318.

1810.3.8.3.2 Seismic reinforcement in Seismic Design Category C. For structures assigned to Seismic Design Category C in accordance with Section 1613, precast prestressed piles shall have transverse reinforcement in accordance with this section. The volumetric ratio of spiral reinforcement shall not be less than the amount required by the following formula for the upper 20 feet (6096 mm) of the pile.

$$\rho_s = 0.12 f'_c / f_{yh} \qquad \textbf{(Equation 18-4)}$$

where:

f'_c = Specified compressive strength of concrete, psi (MPa)
f_{yh} = Yield strength of spiral reinforcement ≤ 85,000 psi (586 MPa).
ρ_s = Spiral reinforcement index (vol. spiral/vol. core).

At least one-half the volumetric ratio required by Equation 18-4 shall be provided below the upper 20 feet (6096 mm) of the pile.

1810.3.8.3.3 Seismic reinforcement in Seismic Design Categories D through F. For structures assigned to Seismic Design Category D, E or F in accordance with Section 1613, precast prestressed piles shall have transverse reinforcement in accordance with the following:

1. Requirements in ACI 318, Chapter 21, need not apply, unless specifically referenced.
2. Where the total pile length in the soil is 35 feet (10 668 mm) or less, the lateral transverse reinforcement in the ductile region shall occur through the length of the pile. Where the pile length exceeds 35 feet (10 668 mm), the ductile pile region shall be taken as the greater of 35 feet (10 668 mm) or the distance from the underside of the pile cap to the point of zero curvature plus three times the least pile dimension.
3. In the ductile region, the center-to-center spacing of the spirals or hoop reinforcement shall not exceed one-fifth of the least pile dimension, six times the diameter of the longtitudinal strand, or 8 inches (203 mm), whichever is smallest.
4. Circular spiral reinforcement shall be spliced by lapping one full turn and bending the end of each spiral to a 90-degree hook or by use of a mechanical or welded splice complying with Sec. 12.14.3 of ACI 318.
5. Where the transverse reinforcement consists of circular spirals, the volumetric ratio of spiral transverse reinforcement in the ductile region shall comply with the following:

$$\rho_s = 0.25(f'_c / f_{yh})(A_g / A_{ch} - 1.0)[0.5 + 1.4P/(f'_c A_g)] \qquad \textbf{(Equation 18-5)}$$

but not less than:

$$\rho_s = 0.12(f'_c / f_{yh})\,[0.5 + 1.4P/(f'_c A_g)] \geq 0.12 f'_c / f_{yh} \qquad \textbf{(Equation 18-6)}$$

and need not exceed:

$$\rho_s = 0.021 \qquad \textbf{(Equation 18-7)}$$

where:

A_g = Pile cross-sectional area, square inches (mm^2).
A_{ch} = Core area defined by spiral outside diameter, square inches (mm^2).
f'_c = Specified compressive strength of concrete, psi (MPa)
f_{yh} = Yield strength of spiral reinforcement ≤ 85,000 psi (586 MPa).
P = Axial load on pile, pounds (kN), as determined from Equations 16-5 and 16-7 .
ρ_s = Volumetric ratio (vol. spiral/ vol. core).

This required amount of spiral reinforcement is permitted to be obtained by providing an inner and outer spiral.

6. Where transverse reinforcement consists of rectangular hoops and cross ties, the total cross-sectional area of lateral transverse reinforcement in the ductile region with spacing, s, and perpendicular dimension, h_c, shall conform to:

$$A_{sh} = 0.3s\, h_c\, (f'_c / f_{yh})(A_g / A_{ch} - 1.0)[0.5 + 1.4P/(f'_c\, A_g)]$$ **(Equation 18-8)**

but not less than:

$$A_{sh} = 0.12s\, h_c\, (f'_c / f_{yh})\, [0.5 + 1.4P/(f'_c\, A_g)]$$ **(Equation 18-9)**

where:
f_{yh} = ≤ 70,000 psi (483 MPa).
h_c = Cross-sectional dimension of pile core measured center to center of hoop reinforcement, inch (mm).
s = Spacing of transverse reinforcement measured along length of pile, inch (mm).
A_{sh} = Cross-sectional area of tranverse reinforcement, square inches (mm^2)
f'_c = Specified compressive strength of concrete, psi (MPa)

The hoops and cross ties shall be equivalent to deformed bars not less than No. 3 in size. Rectangular hoop ends shall terminate at a corner with seismic hooks.

Outside of the length of the pile requiring transverse confinement reinforcing, the spiral or hoop reinforcing with a volumetric ratio not less than one-half of that required for transverse confinement reinforcing shall be provided.

1810.3.9 Cast-in-place deep foundations. Cast-in-place deep foundation elements shall be design and detailed in accordance with Sections 1810.3.9.1 through 1810.3.9.6.

1810.3.9.1 Design cracking moment. The design cracking moment (ΦM_n) for a cast-in-place deep foundation element not enclosed by a structural steel pipe or tube shall be determined using the following equation:

$$\phi M_n = 3\sqrt{f'_c}\, S_m$$ **(Equation 18-10)**

where:
f'_c = Specified compressive strength of concrete or grout, psi (MPa)
S_m = Elastic section modulus, neglecting reinforcement and casing, in^3 (mm^3)

1810.3.9.2 Required reinforcement. Where subject to uplift or where the required moment strength determined using the load combinations of Section 1605.2 exceeds the design cracking moment determined in accordance with Section 1810.3.9.1, cast-in-place deep foundations not enclosed by a structural steel pipe or tube shall be reinforced.

1810.3.9.3 Placement of reinforcement. Reinforcement where required shall be assembled and tied together and shall be placed in the deep foundation element as a unit before the reinforced portion of the element is filled with concrete.

Exceptions:

1. Steel dowels embedded 5 feet (1524 mm) or less shall be permitted to be placed after concreting, while the concrete is still in a semifluid state.

2. For deep foundation elements installed with a hollow-stem auger, tied reinforcement shall be placed after elements are concreted, while the concrete is still in a semifluid state. Longitudinal reinforcement without lateral ties shall be placed either through the hollow stem of the auger prior to concreting or after concreting, while the concrete is still in a semifluid state. All pile reinforcement shall have a concrete cover of not less than 2.5 inches (64 mm).
3. For Group R-3 and U occupancies not exceeding two stories of light-frame construction, reinforcement is permitted to be placed after concreting, while the concrete is still in a semifluid state, and the concrete cover requirement is permitted to be reduced to 2 inches (51 mm), provided the construction method can be demonstrated to the satisfaction of the building official.

1810.3.9.4 Seismic reinforcement. Where a structure is assigned to Seismic Design Category C reinforcement shall be provided in accordance with Section 1810.3.9.4.1. Where a structure is assigned to Seismic Design Category D, E, or F reinforcement shall be provided in accordance with Section 1810.3.9.4.2.

Exceptions:

1. Isolated deep foundation elements supporting posts of Group R-3 and U occupancies not exceeding two stories of light-frame construction shall be permitted to be reinforced as required by rational analysis but with not less than one No. 4 bar, without ties or spirals, where detailed so the element is not subject to lateral loads and the soil provides adequate lateral support in accordance with Section 1810.2.1.
2. Isolated deep foundation elements supporting posts and bracing from decks and patios appurtenant to Group R-3 and U occupancies not exceeding two stories of light-frame construction shall be permitted to be reinforced as required by rational analysis but with not less than one No. 4 bar, without ties or spirals, where the lateral load, *E*, to the top of the element does not exceed 200 pounds (890 N) and the soil provides adequate lateral support in accordance with Section 1810.2.1.
3. Deep foundation elements supporting the concrete foundation wall of Group R-3 and U occupancies not exceeding two stories of light-frame construction shall be permitted to be reinforced as required by rational analysis but with not less than two No. 4 bars, without ties or spirals, where the design cracking moment determined in accordance with Section 1810.3.9.1 exceeds the required moment strength determined using the load combinations with overstrength factor in Section 12.4.3.2 of ASCE 7 and the soil provides adequate lateral support in accordance with Section 1810.2.1.
4. Closed ties or spirals where required by Section 1810.3.9.4.2 shall be permitted to be limited to the top 3 feet (914 mm) of deep foundation elements 10 feet (3048 mm) or less in depth supporting Group R-3 and U occupancies of Seismic Design Category D, not exceeding two stories of light-frame construction.

1810.3.9.4.1 Seismic reinforcement in Seismic Design Category C. For structures assigned to Seismic Design Category C in accordance with Section 1613, cast-in-place deep foundation elements shall be reinforced as specified in this section. Reinforcement shall be provided where required by analysis.

A minimum of four longitudinal bars, with a minimum longitudinal reinforcement ratio of 0.0025, shall be provided throughout the minimum reinforced length of the element as defined below starting at the top of the element. The minimum reinforced length of the element shall be taken as the greatest of the following:

1. one-third of the element length;
2. a distance of 10 feet (3048 mm);
3. three times the least element dimension; and
4. the distance from the top of the element to the point where the design cracking moment determined in accordance with Section 1810.3.9.1 exceeds the required moment strength determined using the load combinations of Section 1605.2.

Transverse reinforcement shall consist of closed ties or spirals with a minimum 3/8 inch (9.5 mm) diameter. Spacing of transverse reinforcement shall not exceed the smaller of 6 inches (152 mm) or 8-longitudinal-bar diameters within a distance of three times the least element dimension from the bottom of the pile cap. Spacing of transverse reinforcement shall not exceed 16 longitudinal bar diameters throughout the remainder of the reinforced length.

Exceptions:

1. The requirements of this section shall not apply to concrete cast in structural steel pipes or tubes.
2. A spiral-welded metal casing of a thickness not less than manufacturer's standard gage No. 14 gage (0.068 inch) is permitted to provide concrete confinement in lieu of the closed ties or spirals. Where used as such, the metal casing shall be protected against possible deleterious action due to soil constituents, changing water levels or other factors indicated by boring records of site conditions.

1810.3.9.4.2 Seismic reinforcement in Seismic Design Categories D through F. For structures assigned to Seismic Design Category D, E or F in accordance with Section 1613, cast-in-place deep foundation elements shall be reinforced as specified in this section. Reinforcement shall be provided where required by analysis.

A minimum of four longitudinal bars, with a minimum longitudinal reinforcement ratio of 0.005 shall be provided throughout the minimum reinforced length of the element as defined below starting at the top of the element. The minimum reinforced length of the element shall be taken as the greatest of the following:

1. one-half of the element length;
2. a distance of 10 feet (3048 mm);
3. three times the least element dimension; and
4. the distance from the top of the element to the point where the design cracking moment determined in accordance with Section 1810.3.9.1 exceeds the required moment strength determined using the load combinations of Section 1605.2.

Transverse reinforcement shall consist of closed ties or spirals no smaller than No. 3 bars for elements with a least dimension up to 20 inches (508 mm), and No. 4 bars for larger elements. Throughout the remainder of the reinforced length outside the regions with transverse confinement reinforcement, as specified in Section 1810.3.9.4.2.1 or 1810.3.9.4.2.2, the spacing of transverse reinforcement shall not exceed the least of the following:

1. 12 longitudinal bar diameters;
2. one-half the least dimension of the element; and
3. 12 inches (305 mm).

Exceptions:

1. The requirements of this section shall not apply to concrete cast in structural steel pipes or tubes.
2. A spiral-welded metal casing of a thickness not less than manufacturer’s standard gage No. 14 gage (0.068 inch) is permitted to provide concrete confinement in lieu of the closed ties or spirals. Where used as such, the metal casing shall be protected against possible deleterious action due to soil constituents, changing water levels or other factors indicated by boring records of site conditions.

1810.3.9.4.2.1 Site Classes A through D. For Site Class A, B, C, or D sites, transverse confinement reinforcement shall be provided in the element in accordance with Sections 21.4.4.1, 21.4.4.2 and 21.4.4.3 of ACI 318 within three times the least element dimension of the bottom of the pile cap. A transverse spiral reinforcement ratio of not less than one-half of that required in Section 21.4.4.1(a) of ACI 318 shall be permitted.

1810.3.9.4.2.2 Site Classes E and F. For Site Class E or F sites, transverse confinement reinforcement shall be provided in the element in accordance with Sections 21.4.4.1, 21.4.4.2 and 21.4.4.3 of ACI 318 within seven times the least element dimension of the pile cap and within seven times the least element dimension of the interfaces of strata that are hard or stiff and strata that are liquefiable or are composed of soft to medium stiff clay.

1810.3.9.5 Belled drilled shafts. Where drilled shafts are belled at the bottom, the edge thickness of the bell shall not be less than that required for the edge of footings. Where the sides of the bell slope at an angle less than 60 degrees (1 rad) from the horizontal, the effects of vertical shear shall be considered.

1810.3.9.6 Socketed drilled shafts Socketed drilled shafts shall have a permanent pipe or tube casing that extends down to bedrock and an uncased socket drilled into the bedrock, both filled with concrete. Socketed drilled shafts shall have a full-length structural steel core or a stub core installed in the rock socket and extending into the pipe portion a distance equal to the socket depth. The depth of the rock socket shall be sufficient to develop the full load-bearing capacity of the element with a minimum safety factor of two, but the depth shall not be less than the outside diameter of the pipe or tube casing. The design of the rock socket is permitted to be predicated on the sum of the allowable load-bearing pressure on the bottom of the socket plus bond along the sides of the socket. The gross cross-sectional area of the structural steel core shall not exceed 25 percent of the gross area of the drilled shaft. The minimum clearance between the structural core and the pipe shall be 2 inches (51 mm).

1810.3.10 Micropiles. Micropiles shall be designed and detailed in accordance with Sections 1810.3.10.1 through 1810.3.10.4.

1810.3.10.1 Construction. Micropiles shall develop their load-carrying capacity by means of a bond zone in soil, bedrock or a combination of soil and bedrock. Micropiles shall be grouted and have either a steel pipe or tube or steel reinforcement at every section along the length. It shall be permitted to transition from deformed reinforcing bars to steel pipe or tube reinforcement by extending the bars into the pipe or tube section by at least their tension development length.

1810.3.10.2 Materials. Grout shall have a specified compressive strength (f'_c) of not less than 4,000 psi (27.58 Mpa). The grout mix shall be designed and proportioned so as to produce a pumpable mixture. Reinforcement shall consist of deformed reinforcing bars in accordance with ASTM A 615 Grade 60 or 75 or ASTM A 722 Grade 150.

The steel pipe or tube shall have a minimum wall thickness of 3/16 inch (4.8 mm). Splices shall comply with Section 1810.3.6. The steel pipe or tube shall have a minimum yield strength of 45,000 psi (310 MPa) and a minimum elongation of 15 percent as shown by mill certifications or two coupon test samples per 40,000 pounds (18 160 kg) of pipe or tube.

1810.3.10.3 Reinforcement. For micropiles or portions thereof grouted inside a temporary or permanent casing or inside a hole drilled into bedrock or a hole drilled with grout, the steel pipe or tube or steel reinforcement shall be designed to carry at least 40 percent of the design compression load. Micropiles or portions thereof grouted in an open hole in soil without temporary or permanent casing and without suitable means of verifying the hole diameter during grouting shall be designed to carry the entire compression load in the reinforcing steel. Where a steel pipe or tube is used for reinforcement, the portion of the grout enclosed within the pipe is permitted to be included in the determination of the allowable stress in the grout.

1810.3.10.4 Seismic reinforcement. For structures assigned to Seismic Design Category C, a permanent steel casing shall be provided from the top of the micropile down to the point of zero curvature. For structures assigned to Seismic Design D, E or F, the micropile shall be considered as an alternative system in accordance with Section 104.11. The alternative system design, supporting documentation and test data shall be submitted to the building official for review and approval.

1810.3.11 Pile caps. Pile caps shall be of reinforced concrete, and shall include all elements to which vertical deep foundation elements are connected, including grade beams and mats. The soil immediately below the pile cap shall not be considered as carrying any vertical load. The tops of vertical deep foundation elements shall be embedded not less than 3 inches (76 mm) into pile caps and the caps shall extend at least 4 inches (102 mm) beyond the edges of the elements. The tops of elements shall be cut or chipped back to sound material before capping.

1810.3.11.1 Seismic Design Categories C through F. For structures assigned to Seismic Design Category C, D, E, or F in accordance with Section 1613, concrete deep foundation elements shall be connected to the pile cap by embedding the element reinforcement or field-placed dowels anchored in the element into the pile cap for a distance equal to their development length in accordance with ACI 318. It shall be permitted to connect precast prestressed piles to the pile cap by developing the element prestressing strands into the pile cap provided the connection is ductile. For deformed bars, the development length is the full development length for compression, or tension~~,~~ in the case of uplift, without reduction for excess reinforcement in accordance with Section 12.2.5 of ACI 318. Alternative measures for laterally confining concrete and maintaining toughness and ductile-like behavior at the top of the element shall be permitted provided the design is such that any hinging occurs in the confined region.

The minimum transverse steel ratio for confinement shall not be less than one-half of that required for columns.

For resistance to uplift forces, anchorage of steel pipes, tubes, or H-piles to the pile cap shall be made by means other than concrete bond to the bare steel section. Concrete-filled steel pipes or tubes shall have reinforcement of not less than 0.01 times the cross-sectional area of the concrete fill developed into the cap and extending into the fill a length equal to two times the required cap embedment, but not less than the tension development length of the reinforcement.

1810.3.11.2 Seismic Design Categories D through F. For structures assigned to Seismic Design Category D, E, or F in accordance with Section 1613, deep foundation element resistance to uplift forces or rotational restraint shall be provided by anchorage into the pile cap, designed considering the combined effect of axial forces due to uplift and bending moments due to fixity to the pile cap. Anchorage shall develop a minimum of 25 percent of the strength of the element in tension. Anchorage into the pile cap shall be capable of developing the following:

1. In the case of uplift, the least of the following: the nominal tensile strength of the longitudinal reinforcement in a concrete element; the nominal tensile strength of a steel element; the frictional force developed between the element and the soil multiplied by 1.3; and the axial tension force resulting from the load combinations of Section 1605.4.
2. In the case of rotational restraint, the lesser of the following: the axial force, shear forces, and bending moments resulting from the load combinations of Section 1605.4; and development of the full axial, bending and shear nominal strength of the element.

Where the vertical lateral-force-resisting elements are columns, the pile cap flexural strengths shall exceed the column flexural strength. The connection between batter piles and pile caps shall be designed to resist the nominal strength of the pile acting as a short column. Batter piles and their connection shall be capable of resisting forces and moments from the load combinations of Section 1605.4.

1810.3.12 Grade beams. For structures assigned to Seismic Design Category D, E, or F in accordance with Section 1613, grade beams shall comply with the provisions in Section 21.10.3 of ACI 318 for grade beams, except where they have the capacity to resist the forces from the load combinations with overstrength factor in Section 12.4.3.2 of ASCE 7.

1810.3.13 Seismic ties. For structures assigned to Seismic Design Category C, D, E, or F in accordance with Section 1613 individual deep foundations shall be interconnected by ties. Ties shall be capable of carrying, in tension and compression, a force equal to the product of the larger pile cap or column load times the seismic coefficient, S_{DS}, divided by 10 unless it can be demonstrated that equivalent restraint is provided by reinforced concrete beams within slabs on grade or reinforced concrete slabs on grade or confinement by competent rock, hard cohesive soils or very dense granular soils.

> **Exception:** In Group R-3 and U occupancies of light-frame construction, deep foundation elements supporting foundation walls, isolated interior posts detailed so the element is not subject to lateral loads, or exterior decks and patios are not subject to interconnection where the soils are of adequate stiffness, subject to the approval of the building official.

1810.4 Installation. Deep foundations shall be installed in accordance with Section 1810.4. Where a single deep foundation element comprises two or more sections of different materials or different types spliced together, each section shall satisfy the applicable conditions of installation.

1810.4.1 Structural integrity. Deep foundation elements shall be installed in such a manner and sequence as to prevent distortion or damage that may adversely affect the structural integrity of adjacent structures or of foundation elements being installed or already in place and as to avoid compacting the surrounding soil to the extent that other foundation elements cannot be installed properly.

1810.4.1.1 Compressive strength of precast concrete piles. A precast concrete pile shall not be driven before the concrete has attained a compressive strength of at least 75 percent of the specified compressive strength (f'_c), but not less than the strength sufficient to withstand handling and driving forces.

1810.4.1.2 Casing Where cast-in-place deep foundation elements are formed through unstable soils and concrete is placed in an open-drilled hole, a casing shall be inserted in the hole prior to placing the concrete. Where the casing is withdrawn during concreting, the level of concrete shall be maintained above the bottom of the casing at a sufficient height to offset any hydrostatic or lateral soil pressure. Driven casings shall be mandrel driven their full length in contact with the surrounding soil.

1810.4.1.3 Driving near uncased concrete. Deep foundation elements shall not be driven within six element diameters center to center in granular soils or within one-half the element length in cohesive soils of an uncased element filled with concrete less than 48 hours old unless approved by the building official. If the concrete surface in any completed element rises or drops, the element shall be replaced. Driven uncased deep foundation elements shall not be installed in soils that could cause heave.

1810.4.1.4 Driving near cased concrete. Deep foundation elements shall not be driven within four and one-half average diameters of a cased element filled with concrete less than 24 hours old unless approved by the building official. Concrete shall not be placed in casings within heave range of driving.

1810.4.1.5 Defective timber piles. Any substantial sudden increase in rate of penetration of a timber pile shall be investigated for possible damage. If the sudden increase in rate of penetration cannot be correlated to soil strata, the pile shall be removed for inspection or rejected.

1810.4.2 Identification. Deep foundation materials shall be identified for conformity to the specified grade with this identity maintained continuously from the point of manufacture to the point of installation or shall be tested by an approved agency to determine conformity to the specified grade. The approved agency shall furnish an affidavit of compliance to the building official.

1810.4.3 Location plan. A plan showing the location and designation of deep foundation elements by an identification system shall be filed with the building official prior to installation of such elements. Detailed records for elements shall bear an identification corresponding to that shown on the plan.

1810.4.4 Preexcavation. The use of jetting, augering or other methods of preexcavation shall be subject to the approval of the building official. Where permitted, preexcavation shall be carried out in the same manner as used for deep foundation elements subject to load tests and in such a manner that will not impair the carrying capacity of the elements already in place or damage adjacent structures. Element tips shall be driven below the preexcavated depth until the required resistance or penetration is obtained.

1810.4.5 Vibratory driving. Vibratory drivers shall only be used to install deep foundation elements where the element load capacity is verified by load tests in accordance with Section 1810.3.3.1.2. The installation of production elements shall be controlled according to power consumption, rate of penetration or other approved means that ensure element capacities equal or exceed those of the test elements.

1810.4.6 Heaved elements. Deep foundation elements that have heaved during the driving of adjacent elements shall be redriven as necessary to develop the required capacity and penetration, or the capacity of the element shall be verified by load tests in accordance with Section 1810.3.3.1.2.

1810.4.7 Enlarged base cast-in-place elements. Enlarged bases for cast-in-place deep foundation elements formed by compacting concrete or by driving a precast base shall be formed in or driven into granular soils. Such elements shall be constructed in the same manner as successful prototype test elements driven for the project. Shafts extending through peat or other organic soil shall be encased in a permanent steel casing. Where a cased shaft is used, the shaft shall be adequately reinforced to resist column action or the annular space around the shaft shall be filled sufficiently to reestablish lateral support by the soil. Where heave occurs, the element shall be replaced unless it is demonstrated that the element is undamaged and capable of carrying twice its design load.

1810.4.8 Hollow-stem augered, cast-in-place elements. Where concrete is placed by pumping through a hollow-stem auger, the auger shall be permitted to rotate in a clockwise direction during withdrawal. The auger shall be withdrawn in continuous increments. Concreting pumping pressures shall be measured and maintained high enough at all times to offset hydrostatic and lateral earth pressures. Concrete volumes shall be measured to ensure that the volume of concrete placed in each element is equal to or greater than the theoretical volume of the hole created by the auger. Where the installation process of any element is interrupted or a loss of concreting pressure occurs, the element shall be redrilled to 5 feet (1524 mm) below the elevation of the tip of the auger when the installation was interrupted or concrete pressure was lost and reformed. Augered cast-in-place elements shall not be installed within six diameters center to center of an element filled with concrete less than 12 hours old, unless approved by the building official. If the concrete level in any completed element drops due to installation of an adjacent element, the element shall be replaced.

1810.4.9 Socketed drilled shafts. The rock socket and pipe or tube casing of socketed drilled shafts shall be thoroughly cleaned of foreign materials before filling with concrete. Steel cores shall be bedded in cement grout at the base of the rock socket. Concrete shall not be placed through water except where a tremie or other approved method is used.

1810.4.10 Micropiles Micropile deep foundation elements shall be permitted to be formed in holes advanced by rotary or percussive drilling methods, with or without casing. The elements shall be grouted with a fluid cement grout. The grout shall be pumped through a tremie pipe extending to the bottom of the element until grout of suitable quality returns at the top of the element. The following requirements apply to specific installation methods:

1. For micropiles grouted inside a temporary casing, the reinforcing bars shall be inserted prior to withdrawal of the casing. The casing shall be withdrawn in a controlled manner with the grout level maintained at the top of the element to ensure that the grout completely fills the drill hole. During withdrawal of the casing, the grout level inside the casing shall be monitored to verify that the flow of grout inside the casing is not obstructed.
2. For a micropile or portion thereof grouted in an open drill hole in soil without temporary casing, the minimum design diameter of the drill hole shall be verified by a suitable device during grouting.
3. For micropiles designed for end bearing, a suitable means shall be employed to verify that the bearing surface is properly cleaned prior to grouting.
4. Subsequent micropiles shall not be drilled near elements that have been grouted until the grout has had sufficient time to harden.
5. Micropiles shall be grouted as soon as possible after drilling is completed.
6. For micropiles designed with a full length casing, the casing shall be pulled back to the top of the bond zone and reinserted or some other suitable means employed to assure grout coverage outside the casing.

1810.4.11 Special inspection. Special inspections in accordance with Sections 1704.8 and 1704.9 shall be provided for driven and cast-in-place deep foundation elements, respectively.

Reason: Significant clarification, update, generalization, and simplification of the code requirements for deep foundations.

Reorganizes deep foundation requirements to eliminate repetition, fix conflicting definitions, and generalize and simplify requirements where possible. Most of the changes proposed are either purely editorial or nearly editorial. The substantive and nearly editorial changes are described herewith.

Definitions: The current definitions cause confusion and conflict. For instance, consider the definitions of "pier" and "pile". Some of the requirements for piers differ from those for piles, but the definitions only confuse matters. By the current definitions, piers must 1) be isolated, 2) be constructed of masonry or cast-in-place concrete, and 3) have a length of no more than 12 times the least horizontal dimension; piles must 1) be of concrete, wood, or steel either driven into the ground or cast in place, and 2) have a length exceeding 12 times the least horizontal dimension. As a result, foundation elements that have length less than 12 times the least horizontal dimension are **neither piers nor piles** if grouped or if constructed of wood, steel, or precast concrete. Several sections (such as 1808.2.5 and 1808.2.9.3) assume that piers are isolated, as the definition requires; Section 1812.1 addresses "isolated and multiple piers", which conflicts with the definition. The solution is, not to revise or add definitions but, to generalize and unify the requirements to the extent possible and then describe specific conditions of concern while specifying the related requirements. In order to unify, generalize, and simplify the requirements, some minor substantive changes are produced. Where the substantive change is small but the improvement in clarity and consistency of application is great, such revisions aid the registered design professional, the building official, and the public. This change proposal groups all deep foundation systems together (by defining shallow foundations) and sets forth general rules for the analysis, design, detailing, and installation of deep foundations. Specific deep foundation types are defined only where the rules for that type are so many and so peculiar that providing verbal descriptions for scoping would become unmanageable.

The exceptions that appeared in Section 1808.2.1 are embodied in the overall revisions to the requirements for deep foundations.

In a related proposal the numbered list in Section 1810.1.1 is moved to the section for geotechnical investigations.

New Section 1810.1.3 is based on current requirements. Concrete elements with height no greater than three times the least horizontal dimension are pedestals (not pilasters) per ACI 318. The exception in current Section 1812.8 is not really an exception, as it describes a different case than that addressed in the text (laterally supported versus unsupported). The intent of that exception is carried forward in the proposed text since it permits use of unreinforced sections where lateral support is provided, moment demands are less than the design cracking moment, and seismic concerns do not govern.

Since one of the conditions of concern in Section 1810.2.1 is "fluid soils", the revised text makes clear that the embedment required is the distance into either stiff soil or soft soil (not the distance below the ground surface). Although the terms "stiff soil" and "soft soil" are, strictly speaking, not (and never have been) defined in this code as related to this provision, they are in general agreement with the terms used in site classification (Section 1613.5.2, Site Classes D and E, respectively).

In new Section 1810.2.2 the first exception addresses a prime condition of what were previously termed "piers"—that is, permission to use isolated elements without additional lateral bracing.

New Sections 1810.2.4 and 1810.2.5 generalize requirements that were previously required for deep foundations of structures assigned to Seismic Design Category C, D, E, or F. This generalization is consistent with current practice and the recommendations of every published standard for deep foundations. Compliance is possible using traditional tables, formulas, or charts or using analysis methods that have been commonly employed for several decades.

New Section 1810.3.1.1 formalizes current practice as implied by the present text of Chapter 18 and as explicitly stated in other documents. The concrete design methods commonly employed no longer recognize allowable stress or working stress design. For many decades structural concrete design has employed the strength design method. However, there is a long tradition of using simple allowable stress design approaches for the proportioning of deep foundation elements (for both soil-foundation behavior and structural design). The proposed text is consistent with the design approaches specified in ACI 543 (Design, Manufacture, and Installation of Concrete Piles). Section 2.3 of that document reads (in part) as follows:

> "Whereas axial compression may often be the primary mode of loading, concrete piles are also frequently subjected to axial tension, bending, and shear loadings as well as various combinations of loading, as noted in Section 2.2. Concrete piles must have adequate structural capacity for all modes and combinations of loading that they will experience. For combined flexure and thrust loadings, the structural adequacy can be evaluated more readily through the use of moment-thrust interaction diagrams and strength-design methods.
>
> :... Because of the historical use of allowable capacities and stresses in piling design, however, recommendations are also provided for allowable axial service capacities for concentrically loaded, laterally supported piles. The allowable service capacities P_a recommended in Section 2.3.3 are intended specifically for cases in which the soil provides full lateral support to the pile and where the applied forces cause no more than minor bending moments resulting from accidental eccentricities. Piles subjected to larger bending moments or with unsupported lengths must be treated as columns in accordance with ACI 318-95 and the provisions given in Sections 2.3.2, 2.3.4, and 2.3.5 of this report."

Rather than treating composite elements in a different section, this proposal generalizes the requirement that each component of the composite element must comply with the applicable provisions of the code.

The proposed text uses the term "casing" in a manner consistent with current practice and use of the terms as defined in ACI 336.1 (Specification for the Construction of Drilled Piers). The term "casing" is appropriate where the element in question resists earth and water pressures. The term "liner" (which does not appear in this proposed text) applies where the element in question resists internal concrete pressures, but is "not designed for external earth and water pressures." Chapter 18 of the IBC does not venture so far into construction methods as to address liners. The only prior occurrences of "liner" (Section 1810.3.3) are related to "hydrostatic and lateral soil pressure," for which the term "casing" is more appropriate. Where the current text of Chapter 18 uses "shell" interchangeably with "casing", this proposal uses "casing" consistently.

The proposed treatment of timber deep foundation elements is more consistent both internally and with respect to the reference codes and standards. The present definition of "timber pile," which requires that the element be round and be placed tip first conflicts with the text that addresses sawn timber piles, which are square, and the reference standard (AF&PA NDS), which permits use of piles (tip first) or poles (butt first). New Section 1810.3.2.4 acknowledges use of both piles and poles.

Allowable stresses: The treatment of allowable stresses in this proposal is simple, clear, consistent, and even-handed as applied to deep foundation elements of different types. In generalizing and treating consistently, minor substantive changes result. The table below compares the allowable stresses specified in ACI 543, the 2006 IBC, and this proposal. For most types this proposal represents no change from the 2006 IBC. However, a few cases have changes of up to about 10 percent (which, practically speaking, is negligible). In the 2006 IBC, driven uncased piles and drilled or augered uncased piles have considerably different allowable stresses. This proposal splits the difference and is generally consistent with ACI 543. The real strength of the proposed approach is that it can be applied to other types of deep foundations without conflict, confusion, or question. Where the present text of Section 1808.2.3 is applied to "special types of piles" it is unclear which of the ten sets of allowable stresses should not be exceeded. Using the proposed text, which generalizes the treatment of allowable stresses, such questions have a ready, defensible answer.

Element type	ACI 543	2006 IBC	This proposal
Precast prestressed concrete piles	$0.33 f'_c - 0.27 f_{pc}$	$0.33 f'_c - 0.27 f_{pc}$	$0.33 f'_c - 0.27 f_{pc}$
Precast nonprestressed concrete piles	$0.33 f'_c$ $0.39 f_y$	$0.33 f'_c$ $0.4 f_y \le 30{,}000$ psi	$0.33 f'_c$ $0.4 f_y \le 30{,}000$ psi
Cast-in-place, uncased, plain	$0.29 f'_c$	$0.25 f'_c$	$0.3 f'_c$
Cast-in-place, uncased, reinforced DRIVEN	$0.28 f'_c$ $0.33 f_y$	$0.25 f'_c$	$0.3 f'_c$ $0.4 f_y \le 30{,}000$ psi
Cast-in-place, uncased, reinforced DRILLED or AUGERED	$0.28 f'_c$ $0.33 f_y$	$0.33 f'_c$ $0.4 f_y \le 25{,}500$ psi	$0.3 f'_c$ $0.4 f_y \le 30{,}000$ psi
Cast-in-place, cased	$0.32 f'_c$	$0.33 f'_c$	$0.33 f'_c$
Cast-in-place, special casing	$0.26(f'_c + 8.2 t_s f_{ys}/D)$ $\le 0.4 f'_c$	$0.4 f'_c$	$0.4 f'_c$
Cast-in-place, structural steel pipe or tube	$0.37 f'_c$ $0.43 f_{yp}$	$0.33 f'_c$ $0.35 F_y \le 12{,}600$ psi up to $0.5 F_y$	$0.33 f'_c$ $0.35 F_y \le 16{,}000$ psi up to $0.5 F_y \le 32{,}000$ psi
Caisson (socketed drilled shaft)	$0.37 f'_c$ $0.43 f_{yp}$	$0.33 f'_c$ $0.35 F_y$ (pipe or tube) $0.5 F_y$ (core)	$0.33 f'_c$ $0.35 F_y \le 16{,}000$ psi up to $0.5 F_y \le 32{,}000$ psi $0.5 F_y$ (core)
Structural steel elements	---	$0.35 F_y$ up to $0.5 F_y$	$0.35 F_y \le 16{,}000$ psi up to $0.5 F_y \le 32{,}000$ psi
Micropiles	---	$0.33 f'_c$ $0.4 f_y \le 32{,}000$ psi $0.4 F_y \le 32{,}000$ psi $0.6 f_y$ Tension	$0.33 f'_c$ $0.4 f_y \le 30{,}000$ psi $0.4 F_y \le 32{,}000$ psi $0.6 f_y$ Tension
Timber	---	In accordance with the AF&PA NDS	In accordance with the AF&PA NDS

The first requirement of new Section 1810.3.2.7 is consistent with current practice and the requirements of ACI 543 Table 2.2. The second requirement is moved from Section 1810.5.1; it is the sealed tip that produces a displacement pile with increased capacity.

In new Section 1810.3.3.1.2 the requirement for load testing of cast-in-place deep foundation elements with an enlarged base previously appeared in Section 1810.2.4.

For improved clarity of application and consistency with ASCE 7-05, seismic requirements are rewritten to avoid "cascading," which often led to confusion concerning scope. For instance, new Section 1810.3.8.2.2 applies to Seismic Design Categories C through F and new Section 1810.3.8.2.3 provides "additional" requirements for Seismic Design Categories D through F; in both cases the scope is clearly defined. In another instance it was possible to separate the requirements; Section 1810.3.8.3.2 applies to Seismic Design Category C and Section 1810.3.8.3.3 (with revised Equation 18-6) applies to Seismic Design Categories D through F.

The change at the end of new Section 1810.3.8.2.2 is editorial although it may not appear so. First, the 8 inch maximum spacing is changed to 6 inches since new Section 1810.3.8.1 specifies a maximum spacing of 6 inches for non-seismic cases. Then, the spacing of 16 longitudinal bar diameters can be eliminated since 16 times the smallest bar diameter (3/8") is no more stringent.

The change in new Section 1810.3.8.3.2 is editorial since Equation 18-4 produces a value greater than 0.007 where the minimum value of f'_c (5 ksi) is used with the maximum value of f_{yh} (85 ksi).

The revision to Equation 18-6 eliminates cascading requirements from the section above.

New Sections 1810.3.9.1 and 1810.3.9.1 clarify the present requirements, agree with the requirements of ACI 318-08, and allow elimination of the definition for flexural length. For both uncased and cased cast-in-place deep foundation elements (but not concrete filled pipes and tubes) reinforcement must be provided where moments exceed a reasonable lower bound for the capacity of the plain concrete section. In several sections of the 2006 IBC (and other related documents) that design cracking moment is taken as 0.4 times the "concrete section cracking moment strength." Section 9.5.2.3 of ACI 318 defines the cracking moment strength as 7.5 times the square root of f'_c times the elastic section modulus of the gross section (0.4 × 7.5 = 3). Using Chapter 22 of ACI 318-08, one would take □ = 0.6 times 5.0 times the square root of f'_c times the elastic section modulus of the gross section (0.6 × 5.0 = 3). The proposed text is consistent with the current requirement and paves the way for use of a reference standard in the future. The proposed sections are also used in place of the less clear phrase "not rupture" in Exception 3 of new Section 1810.3.9.4.

Editorial note: Where metric units are used, Equation 18-10 should be shown as $\phi M_n = 0.25\sqrt{f'_c}S_m$.

The proposed revisions in new Section 1810.3.10.1 clarifies the intent to permit the pipe or tube casing to terminate above the bond zone, with deformed bar reinforcement continuing below. It also specifies a splice condition for that transition.

In new Section 1810.3.10.4 "120 percent of the flexural length" is changed to "the point of zero curvature" for two reasons. First, with the revisions related to design cracking moment, this is the only section that uses the current definition of flexural length (first point of zero lateral deflection). Second, the distance to the point of zero curvature, which is also used in new Section 1810.3.8.3.3, is approximately equal to 120 percent of the distance to the point of zero deflection.

Bibliography:
Composite of Chapter 18 reorganization assuming all of proponent's proposals are approved.

Cost Impact: The code change proposal will not increase the cost of construction.

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S161–07/08
1808.2.8.5

Proponent: Edwin T. Huston, Smith & Huston, Inc., representing National Council of Structural Engineering Associations

Revise as follows:

1808.2.8.5 Uplift capacity. Where required by the design, the uplift capacity of a single pier or pile shall be determined by an approved method of analysis based on a minimum factor of safety of three or by load tests conducted in accordance with ASTM D 3689. The maximum allowable uplift load shall not exceed the ultimate load capacity as determined in Section 1808.2.8.3 divided by a factor of safety of two.

> <u>**Exception:** Where uplift is due to wind or seismic loading, the minimum factor of safety shall be 2 where capacity is determined by analysis and 1.5 where capacity is determined by load tests.</u>

For pile groups subjected to uplift, the allowable working uplift load for the group shall be the lesser of:

1. The proposed individual pile uplift working load times the number of piles in the group.
2. Two-thirds of the effective weight of the pile group and the soil contained within a block defined by the perimeter of the group and the length of the pile.

Reason: Code update and consistency. Changes the required factor of safety for uplift where uplift is due to wind or seismic loading. The revision is generally consistent with the long-standing practice of permitting wind or seismic loads that are four-thirds of those for sustained loads.

Cost Impact: The code change proposal will not increase the cost of construction.

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S162–07/08
1808.2.8.5

Proponent: Edwin T. Huston, Smith & Huston, Inc., representing National Council of Structural Engineering Associations

Revise as follows:

1808.2.8.5 Uplift capacity. Where required by the design, the uplift capacity of a single pier or pile shall be determined by an approved method of analysis based on a minimum factor of safety of three or by load tests conducted in accordance with ASTM D 3689. The maximum allowable uplift load shall not exceed the ultimate load capacity as determined in Section 1808.2.8.3<u>, using the results of load tests conducted in accordance with ASTM D 3689,</u> divided by a factor of safety of two. For pile groups subjected to uplift, the allowable working uplift load for the group shall be <u>calculated by an approved method of analysis. Where the deep foundation elements in the group are placed at a spacing of at least 2.5 times the least horizontal dimension of the largest single element, the allowable working uplift load for the group is permitted to be calculated as</u> the lesser of:

1. The proposed individual pile uplift working load times the number of piles in the group.
2. Two-thirds of the effective weight of the pile group and the soil contained within a block defined by the perimeter of the group and the length of the pile.

Reason: Code clarification and update. Clarifies the safety factor applied to results of load tests in accordance with ASTM D 3689. Allows more precise calculation of uplift capacity.

Cost Impact: The code change proposal will not increase the cost of construction.

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S163–07/08
1808.2.8.8

Proponent: Edwin T. Huston, Smith & Huston, Inc., representing National Council of Structural Engineering Associations

Revise as follows:

1808.2.8.8 Overloads on piers or piles. <u>The foundation or superstructure shall be designed to resist the effects of the mislocation of any deep foundation element by no less than 3 inches (76 mm).</u> The maximum compressive load on any pier or pile due to mislocation shall not exceed 110 percent of the allowable design load.

Reason: Code update and consistency. Proper design of deep foundations must recognize the significance of permitted construction tolerances. The effects of mislocation are recognized in the current text of Chapter 18 in Section 1808.2.8.8 (which permits slight overload due to mislocation) and Section 1808.2.7 (which requires that splices be designed for accidental eccentricity). This change formalizes the requirement to consider mislocation. The effects of mislocation can be resisted by flexure and axial load in vertical deep foundation elements, flexure in pile caps, or resistance in the superstructure above. This proposed change permits any of these approaches.

Cost Impact: The code change proposal will not increase the cost of construction.

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S164–07/08
1808.2.9.2

Proponent: Edwin T. Huston, Smith & Huston, Inc., representing National Council of Structural Engineering Associations

Revise as follows:

1808.2.9.2 Unbraced piles. Piles standing unbraced in air, water or in fluid soils shall be designed as columns in accordance with the provisions of this code. Such piles driven into firm ground can be considered fixed and laterally supported at 5 feet (1524 mm) below the ground surface and in soft material at 10 feet (3048 mm) below the ground surface unless otherwise <u>approved</u> ~~prescribed~~ by the building official <u>on the basis of</u> ~~after~~ a ~~foundation~~ <u>geotechnical</u> investigation by <u>a registered design professional</u> ~~an approved agency~~.

Reason: Code update. Revises the process to be consistent with practice. Building officials do not prescribe the depth of embedment needed for lateral support; they approve such depths on the basis of geotechnical investigations.

Cost Impact: The code change proposal will not increase the cost of construction.

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S165–07/08
1808.2.9.2

Proponent: Edwin T. Huston, Smith & Huston, Inc., representing National Council of Structural Engineering Associations

Revise as follows:

1808.2.9.2 Unbraced piles. Piles standing unbraced in air, water or in fluid soils shall be designed as columns in accordance with the provisions of this code. Such piles driven into firm ground can be considered ~~fixed and~~ laterally supported at 5 feet (1524 mm) below the ground surface and in soft material at 10 feet (3048 mm) below the ground surface unless otherwise prescribed by the building official after a foundation investigation by an approved agency.

Reason: Code update. Fixity should be considered explicitly as is common in practice. Explicit consideration of fixity may be based on formulas and charts from the literature (such as in NAVFAC design manuals) or on p-y analysis (using LPILE or similar programs).

While it is reasonable to assume that piles are laterally supported where embedded in firm or soft soil, it is not reasonable to assume them to have flexural fixity with the distances of embedment indicated. Firm and soft soil conditions vary considerably. Also, the stiffness of the deep foundation element plays a critical role in assessing fixity. For typical deep foundation elements with diameters of 14 to 48 inches in soil profiles associated with Site Classes A, B, C, D, and E, the depth to fixity may vary from a low of about 3 feet to a high of about 20 feet.

Cost Impact: The code change proposal will not increase the cost of construction.

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S166–07/08
1808.2.9.3, 1808.2.12

Proponent: Edwin T. Huston, Smith & Huston, Inc., representing National Council of Structural Engineering Associations

Revise as follows:

1808.2.9.3 Allowable lateral load. Where required by the design, the lateral load capacity of a pier, a single pile or a pile group shall be determined by an approved method of analysis or by lateral load tests to at least twice the proposed design working load. The resulting allowable load shall not be more than one-half of <u>the</u> ~~that test~~ load that produces a gross lateral movement of 1 inch (25 mm) at the <u>lower of the top of foundation element and the</u> ground surface<u>, unless it can be shown that the predicted lateral movement shall cause neither harmful distortion of, nor instability in, the structure, nor cause any element to be loaded beyond its capacity</u>.

1808.2.12 Settlement analysis. The settlement of piers, individual piles or groups of piles shall be estimated based on approved methods of analysis. The predicted settlement shall cause neither harmful distortion of, nor instability in, the structure, nor cause any <u>element to be loaded beyond its capacity</u> ~~stresses to exceed allowable values~~.

Reason: Code clarification and update. Clarifies that the lateral drift criterion applies to both analysis and testing. Allows demonstration of different lateral drift criteria in a manner similar to that for settlement. Revises the text related to settlement for better consistency with modern design techniques (which are not limited to allowable stress design).

Cost Impact: The code change proposal will not increase the cost of construction.

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S167–07/08
1808.2.10, 1810.3.2, 1810.4.2

Proponent: Edwin T. Huston, Smith & Huston, Inc., representing National Council of Structural Engineering Associations

Revise as follows:

1808.2.10 Use of higher allowable pier or pile stresses. Allowable stresses greater than those specified for piers or for each pile type in Sections 1809 and 1810 are permitted where supporting data justifying such higher stresses is filed with the building official. Such substantiating data shall include:

1. A soils investigation in accordance with Section 1802.
2. Pier or pile load tests in accordance with Section 1808.2.8.3, regardless of the load supported by the pier or pile.

The design and installation of the pier or pile foundation shall be under the direct supervision of a registered design professional knowledgeable in the field of soil mechanics and pier or pile foundations who shall ~~certify~~ <u>submit a report</u> to the building official <u>stating</u> that the piers or piles as installed satisfy the design criteria.

1810.3.2 Dimensions. The pile length shall not exceed 30 times the average diameter. The minimum diameter shall be 12 inches (305 mm).

Exception: The length of the pile is permitted to exceed 30 times the diameter, provided that the design and installation of the pile foundation are under the direct supervision of a registered design professional knowledgeable in the field of soil mechanics and pile foundations. The registered design professional shall ~~certify~~ submit a report to the building official stating that the piles were installed in compliance with the approved construction documents.

1810.4.2 Dimensions. The pile length shall not exceed 30 times the average diameter. The minimum diameter shall be 12 inches (305 mm).

Exception: The length of the pile is permitted to exceed 30 times the diameter, provided that the design and installation of the pile foundation is under the direct supervision of a registered design professional knowledgeable in the field of soil mechanics and pile foundations. The registered design professional shall ~~certify~~ submit a report to the building official stating that the piles were installed in compliance with the approved design.

Reason: For legal and professional practice reasons, changes requirements for registered design professional to "certify" to "report".

Cost Impact: The code change proposal will not increase the cost of construction.

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S168–07/08
1808.2.23.1.1

Proponent: Edwin T. Huston, Smith & Huston, Inc., representing National Council of Structural Engineering Associations

Revise as follows:

1808.2.23.1.1 Connection to pile cap. Concrete piles and concrete-filled steel pipe piles shall be connected to the pile cap by embedding the pile reinforcement or field-placed dowels anchored in the concrete pile in the pile cap for a distance equal to the development length. For deformed bars, the development length is the full development length for compression or tension, in the case of uplift, without reduction in length for excess area. Alternative measures for laterally confining concrete and maintaining toughness and ductile-like behavior at the top of the pile will be permitted provided the design is such that any hinging occurs in the confined region.

Ends of hoops, spirals and ties shall be terminated with seismic hooks, as defined in Section 21.1 of ACI 318, turned into the confined concrete core. The minimum transverse steel ratio for confinement shall not be less than one-half of that required for columns.

For resistance to uplift forces, anchorage of steel pipe (round HSS sections), concrete-filled steel pipe or H-piles to the pile cap shall be made by means other than concrete bond to the bare steel section.

Exception: Anchorage of concrete-filled steel pipe piles is permitted to be accomplished using deformed bars developed into the concrete portion of the pile.

Splices of pile segments shall develop the full strength of the pile, but the splice need not develop the nominal strength of the pile in tension, shear and bending when it has been designed to resist axial and shear forces and moments from the load combinations of Section 1605.4.

Where this chapter requires detailing of concrete deep foundation elements in accordance with Section 21.4.4.1 of ACI 318, compliance with Equation (10-5) of ACI 318 shall not be required.

Reason: Relaxes an overly restrictive code requirement. Proper design for deep foundation elements differs from that for columns in several respects.

First, axial compression is severely limited by the capacity of the soil-foundation interface (as reflected by code limits to a small fraction of the concrete compressive strength). The purpose of ACI 318 Equation (10-5) is to provide significant residual compressive strength for concentrically loaded spiral columns subjected to very large axial compression.

"ACI 318 Section R10.9.3 — The effect of spiral reinforcement in increasing the load-carrying strength of the concrete within the core is not realized until the column has been subjected to a load and deformation sufficient to cause the concrete shell outside the core to spall off. The amount of spiral reinforcement required by Eq. (10-5) is intended to provide additional load-carrying strength for concentrically loaded columns equal to or slightly greater than the strength lost when the shell spalls off."

Second, the concern driving transverse confinement reinforcement for deep foundations is flexural ductility. The commentary to ACI 318 explains both the reason to provide confinement reinforcement for deep foundation elements and the result achieved by compliance with Equation (21-2).

"R21.10.4.4 — During earthquakes, piles can be subjected to extremely **high flexural demands** at points of discontinuity, especially just below the pile cap and near the base of a soft or loose soil deposit. The 1999 code requirement for confinement reinforcement at the top of the pile is based on numerous failures observed at this location in recent earthquakes. Transverse reinforcement is required in this region to provide **ductile performance**. …"

"ACI 318 R21.4.4 — … Eq. (21-2) and (21-4) govern for large-diameter columns, and are intended to **ensure adequate flexural curvature capacity in yielding regions**."

Third, the concrete cover required for uncased deep foundation elements is much greater than that for columns. As a result, application of Eq. (10-5) to deep foundation elements results in amounts of transverse reinforcement that are unwarranted and often un-placable as shown in the comparison below. The calculations use typical material properties, bar sizes, and minimum cover and also illustrate the effect of the permitted reduction in transverse reinforcement ratio for Site Classes A, B, C, and D. It can be seen that application of Eq. (10-5) produces transverse reinforcement spacing that is difficult or even impossible to implement (even where the reduction applies). The problem is especially pronounced for small diameter elements as the ratio of gross area to core area results in Eq. (10-5) controlling by a factor of 5 to 8. Where those elements are augercast (as is often the case for small diameter uncased cast-in-place deep foundations) it is nearly impossible to place properly a reinforcement cage with #5 spiral at 2.5" o.c.

f'_c =	**5**	ksi
f_y =	**60**	ksi
cover =	**2.5**	inch
Site Class =	**E**	(no reduction)

Diameter	**1.0 x** min ρ_s			max	$A_{s,reqd}$ at max spcg		Spcg for #3		Spcg for #4		Spcg for #5	
D	(21-2)	(10-5)	ratio	spcg*	(21-2)	(10-5)	(21-2)	(10-5)	(21-2)	(10-5)	(21-2)	(10-5)
12"	0.010	0.073	**7.27**	3	0.05	0.38	[3.0"]	0.9"	[3.0"]	1.6"	[3.0"]	2.4"
14"	0.010	0.053	**5.32**	3.5	0.08	0.42	[3.5"]	0.9"	[3.5"]	1.7"	[3.5"]	2.6"
16"	0.010	0.042	**4.18**	4	0.11	0.46	[4.0"]	1.0"	[4.0"]	1.7"	[4.0"]	2.7"
18"	0.010	0.034	**3.44**	4.5	0.15	0.50	3.4"	1.0"	[4.5"]	1.8"	[4.5"]	2.8"
20"	0.010	0.029	**2.92**	4.5	0.17	0.49	2.9"	1.0"	[4.5"]	1.8"	[4.5"]	2.8"
22"	0.010	0.025	**2.53**	5.5	0.23	0.59	2.6"	1.0"	4.7"	1.9"	[5.5"]	2.9"
24"	0.010	0.022	**2.23**	6	0.29	0.64	2.3"	1.0"	4.2"	1.9"	[6.0"]	2.9"
26"	0.010	0.020	**2.00**	6	0.32	0.63	2.1"	1.0"	3.8"	1.9"	5.9"	3.0"
28"	0.010	0.018	**1.81**	6	0.35	0.62	1.9"	1.1"	3.5"	1.9"	5.4"	3.0"
30"	0.010	0.017	1.65	6	0.38	0.62	1.8"	1.1"	3.2"	1.9"	5.0"	3.0"
32"	0.010	0.015	1.52	6	0.41	0.61	1.6"	1.1"	3.0"	2.0"	4.6"	3.0"
34"	0.010	0.014	1.40	6	0.44	0.61	1.5"	1.1"	2.8"	2.0"	4.3"	3.0"
36"	0.010	0.013	1.31	6	0.47	0.61	1.4"	1.1"	2.6"	2.0"	4.0"	3.1"

*#6 bars; #8 for D > 20"   maximum spacing controls bracketed values

f'_c =	**4**	ksi
f_y =	**60**	ksi
cover =	**2.5**	inch
Site Class =	**D**	(reduction permitted)

Diameter	**0.5 x** min ρ_s			max	$A_{s,reqd}$ at max spcg		Spcg for #3		Spcg for #4		Spcg for #5	
D	(21-2)	(10-5)	ratio	spcg*	(21-2)	(10-5)	(21-2)	(10-5)	(21-2)	(10-5)	(21-2)	(10-5)
12"	0.004	0.029	**7.27**	3	0.02	0.15	[3.0"]	2.2"	[3.0"]	[3.0"]	[3.0"]	[3.0"]
14"	0.004	0.021	**5.32**	3.5	0.03	0.17	[3.5"]	2.3"	[3.5"]	[3.5"]	[3.5"]	[3.5"]
16"	0.004	0.017	**4.18**	4	0.04	0.18	[4.0"]	2.4"	[4.0"]	[4.0"]	[4.0"]	[4.0"]
18"	0.004	0.014	**3.44**	4.5	0.06	0.20	[4.5"]	2.5"	[4.5"]	4.5"	[4.5"]	[4.5"]
20"	0.004	0.012	**2.92**	4.5	0.07	0.20	[4.5"]	2.5"	[4.5"]	[4.5"]	[4.5"]	[4.5"]
22"	0.004	0.010	**2.53**	5.5	0.09	0.24	[5.5"]	2.6"	[5.5"]	4.6"	[5.5"]	[5.5"]
24"	0.004	0.009	**2.23**	6	0.11	0.25	5.8"	2.6"	[6.0"]	4.7"	[6.0"]	[6.0"]
26"	0.004	0.008	**2.00**	6	0.13	0.25	5.2"	2.6"	[6.0"]	4.8"	[6.0"]	[6.0"]
28"	0.004	0.007	**1.81**	6	0.14	0.25	4.8"	2.6"	[6.0"]	4.8"	[6.0"]	[6.0"]
30"	0.004	0.007	1.65	6	0.15	0.25	4.4"	2.7"	[6.0"]	4.8"	[6.0"]	[6.0"]
32"	0.004	0.006	1.52	6	0.16	0.25	4.1"	2.7"	[6.0"]	4.9"	[6.0"]	[6.0"]
34"	0.004	0.006	1.40	6	0.17	0.24	3.8"	2.7"	[6.0"]	4.9"	[6.0"]	[6.0"]
36"	0.004	0.005	1.31	6	0.19	0.24	3.5"	2.7"	[6.0"]	4.9"	[6.0"]	[6.0"]

*#6 bars; #8 for D > 20"   maximum spacing controls bracketed values

In a related proposal Section 1808.2.12.1.1 is split up and reorganized. Should both proposals be accepted this new text is intended to appear as Section 1810.3.2.1.2 with the title "**ACI 318 Equation (10-5).**"

Cost Impact: The code change proposal will not increase the cost of construction.

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S169–07/08
1810.1.1

Proponent: Edwin T. Huston, Smith & Huston, Inc., representing National Council of Structural Engineering Associations

Revise as follows:

1810.1.1 Materials. Concrete shall have a 28-day specified compressive strength (f'_c) of not less than 2,500 psi (17.24 MPa). Where concrete is placed through a funnel hopper at the top of the pile, the concrete mix shall be designed and proportioned so as to produce a cohesive workable mix having a slump of not less than 4 inches (102 mm) and not more than ~~6~~ 8 inches (~~152~~ 204 mm). Where concrete is to be pumped, the mix design including slump shall be adjusted to produce a pumpable concrete.

Reason: Code update. Section 2.4.3 of ACI 336.1-01 (Specification for the Construction of Drilled Piers) specifies a slump of 4 to 6 inches for dry method placement uncased or with permanent casing and 6 to 8 inches for dry method placement with temporary casing. Since both methods of placement are permitted by the IBC, the range of permitted slumps should be stated accordingly.

Cost Impact: The code change proposal will not increase the cost of construction.

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S170–07/08
1810.7.1, 1810.7.4

Proponent: Edwin T. Huston, Smith & Huston, Inc., representing Council of Structural Engineering Associations

Revise as follows:

1810.7.1 Construction. Caisson piles shall consist of a shaft section of concrete-filled pipe extending to bedrock with an uncased socket drilled into the bedrock and filled with concrete. The caisson pile shall have reinforcement or a ~~full-length~~ structural steel core for the length as indicated by an approved method of analysis.~~or a stub core installed in the rock socket and extending into the pipe portion a distance equal to the socket depth~~.

1810.7.4 Structural core. Where a structural steel core is used, t~~T~~he gross cross-sectional area of the ~~structural steel~~ core shall not exceed 25 percent of the gross area of the caisson. The minimum clearance between the structural core and the pipe shall be 2 inches (51 mm). Where cores are to be spliced, the ends shall be milled or ground to provide full contact and shall be full-depth welded.

Reason: Code update and consistency. Socketed drilled shafts often employ reinforcement instead of structural core sections. This common construction is widely acknowledged. For instance, ACI 336.3 (Design and Construction of Drilled Piers) reads as follows:
"**3.6-- Piers socketed in rock**
This type of pier is socketed into rock to a depth of one to six times the diameter of the pier for the purpose of developing high service loading capacity. The drilled pier consists of a heavy wall permanent casing fitted with a cutting shoe and seated into the top of rock. A steel core or heavy reinforcing cage is encased in the concrete which extends into the rock socket. A column cap is designed to transfer loads from the superstructure to one or more rock socketed piers. The pier is designed to support all load in the rock."

Analysis is required to determine the appropriate distance to extend reinforcement or a structural steel core from the socket into the pipe or tube casing.

Cost Impact: The code change proposal will not increase the cost of construction.

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S171–07/08
1808.1, 1813 (New)

Proponent: Robert M. Hoyt, Hoyt Engineering, PC, Representing Deep Foundations Institute Committee on Helical Foundations and Tiebacks, Deep Foundations Institute

1. Add new test as follows:

1808.1 Definitions. The following words and terms shall, for the purposes of this section, have the meanings shown herein.

PILE FOUNDATIONS. Pile foundations consist of concrete, wood or steel structural elements either driven into the ground or cast in place. Piles are relatively slender in comparison to their length, with lengths exceeding 12 times the least horizontal dimension. Piles derive their load-carrying capacity through skin friction, end bearing or a combination of both.

HELICAL PILE. Manufactured steel foundation pile consisting of a central shaft and one or more helical bearing plates. A helical pile is installed by rotating into the ground. Each helical bearing plate is formed into a screw thread with a uniform defined pitch.

SECTION 1813
HELICAL PILE FOUNDATIONS

1813.1 General. Helical pile foundations shall conform to the requirements of Sections 1813.2 through 1813.7.

1813.2 Dimensions. Dimensions of the central shaft and the number, size and thicknesses of helical bearing plates shall be sufficient to support the service loads, as listed in an ICC-ES evaluation report or as determined by a licensed design professional experienced in geotechnical engineering and the design of foundations utilizing helical piles.

1813.3 Design and manufacture. Helical piles shall be designed and manufactured in accordance with accepted engineering practice to resist all stresses induced by installation into the ground and service loads. When compliance of helical piles with Section 1813.2 is based on an ICC-ES evaluation, a copy of the evaluation report shall be provided to the building department.

1813.4 Allowable stresses. The allowable design stress, F_a, in the steel components of the pile shall not exceed the least value of the following:

$F_a = 0.6\ F_y$ or $0.5\ F_u$ **(Equation 18-11)**

where:
F_y = yield strength of the steel
F_u = ultimate strength of the steel

1813.5 Allowable loads

1813.5.1 Allowable axial load. The allowable axial design load, Q_a, of helical piles shall be limited to the least value associated with the interaction of the pile with the soil, the strength of the pile shaft, the strength of the shaft couplings, the strength of the helical bearing plates, or the strength of the joint between the helical bearing plates and the shaft, as listed in an ICC-ES evaluation report or as determined by a licensed design professional experienced in such evaluations.

1813.5.2 Allowable lateral load. The allowable lateral design load, Pa, of helical piles shall be limited to the least value associated with the interaction of the pile with the soil, the strength of the pile shaft, or the strength of the shaft couplings, as listed in an ICC-ES evaluation report or as determined by a licensed design professional experienced in such evaluations of helical piles.

1813.6 Special inspection. Special inspections in accordance with Section 1704.8 are required, as prescribed in Section 1808.2.22. The records submitted to the building official shall include, in addition to the records specified in 1704.8, installation equipment used; pile shaft dimensions, helix configuration and material grades; torsional resistance vs. embedment length; and such other installation data as the special inspector may deem appropriate.

1813.7 Installation. Helical piles shall be installed to specified embedment depth and torsional resistance criteria as stated in an applicable ICC-ES evaluation report or as determined by a licensed design professional experienced in geotechnical engineering and the design of foundations utilizing helical piles. The torque applied during installation shall not exceed the maximum allowable installation torque of the helical pile as listed in an applicable ICC-ES evaluation report or, in the absence of such a report, as recommended by the helical pile manufacturer.

Reason: The purpose of this proposal is to add provisions addressing the design and installation of helical pile foundations. Helical piles are not currently listed and their design and installation are not currently addressed in the Code. Helical pile foundations are becoming more and more common in civil construction. Proposed new Section 1813 contains specifications for the design and installation of helical pile foundations. The section will extend the coverage of the code to an increasingly popular but currently un-regulated type of deep foundation. The proposed definition, design and installation provisions conform to newly adopted ICC-ES AC358.

Cost Impact: The code change proposal will not increase the cost of construction.

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S172–07/08

1904.2 (New), 1904.2.1, 1904.2.2, 1904.2.3, 1904.3, 1904.4, 1904.5 (New), 1907.7, 1907.7.1, 1907.7.2, 1907.7.3, 1907.7.4, 1907.7.5 (New), 1907.7.6, 1907.7.7, 1908, 1909.6.1, 1909.6.3, 1912.1, Table 1704.3, 1708.3

Proponent: Joseph J. Messersmith, Jr. PE, Portland Cement Association; Daniel Falconer, PE, American Concrete Institute

1. Add new text as follows:

1904.2 Exposure categories and classes. Concrete shall be assigned to exposure classes based on:

1. Exposure to freezing and thawing in a moist condition or deicer chemicals;
2. Exposure to sulfates in water or soil;
3. Exposure to water where the concrete is intended to have low permeability; and
4. Exposure to chlorides from deicing chemicals, salt, salt water, brackish water, seawater or spray from these sources, where the concrete has steel reinforcement.

2. Delete without substitution:

~~**1904.2 Freezing and thawing exposures.** Concrete that will be exposed to freezing and thawing, deicing chemicals or other exposure conditions as defined below shall comply with Sections 1904.2.1 through 1904.2.3.~~

~~**1904.2.1 Air entrainment.** Concrete exposed to freezing and thawing or deicing chemicals shall be air entrained in accordance with ACI 318, Section 4.2.1.~~

3. Revise as follows:

~~**1904.2.2**~~ **1904.3 (Supp) Concrete properties.** Concrete ~~that will be subject to the following exposures~~ mixtures shall conform to the ~~corresponding~~ most restrictive maximum water-cementitious materials ratios and minimum specified concrete compressive strength requirements of ACI 318, Section ~~4.2.2.~~ 4.3: based on the exposure classes assigned in Section 1904.2.

~~1. Concrete intended to have low permeability where exposed to water;~~
~~2. Concrete exposed to freezing and thawing in a moist condition or deicer chemicals; or~~
~~3 Corrosion protection of reinforcement in concrete exposed to chlorides from deicing chemicals, salt, salt water, brackish water, seawater, or spray from these sources.~~

> **Exception:** For occupancies and appurtenances thereto in Group R occupancies that are in buildings less than four stories above grade plane, normal-weight aggregate concrete ~~shall~~ is permitted to comply with the requirements of Table 1904.2.2(2) based on the weathering classification (freezing and thawing) determined from Figure 1904.2.2 in lieu of the requirements of ACI 318, Table 4.3.1.a.

~~In addition, concrete that will be exposed to deicing chemicals shall conform to the limitation of Section 1904.2.3.~~

~~1904.2~~ 1904.4.1 Freezing and thawing exposures. Concrete that will be exposed to freezing and thawing, in the presence of moisture, with or without deicing chemicals being present, ~~or other exposure conditions as defined below~~ shall comply with Sections ~~1904.2.1 through 1904.2.3~~ 1904.4.1 and 1904.4.2.

~~1904.2.1~~ 1904.4.2 Air entrainment. Concrete exposed to freezing and thawing ~~or deicing chemicals~~ while moist shall be air entrained in accordance with ACI 318, Section ~~4.2.1~~ 4.4.1:

~~1904.2.3~~ 1904.4.2 Deicing chemicals. For concrete exposed to freezing and thawing in the presence of moisture and deicing chemicals, the maximum weight of fly ash, other pozzolans, silica fume or slag that is included in the concrete shall not exceed the percentages of the total weight of cementitious materials permitted by ACI 318, Section ~~4.2.3~~ 4.4.2.

4. Delete without substitution:

~~**1904.3 Sulfate exposures.** Concrete that will be exposed to sulfate-containing solutions or soils shall comply with the maximum water-cementitious materials ratios and/or minimum specified compressive strength and be made with the appropriate type of cement in accordance with the provisions of ACI 318, Section 4.3.~~

~~**1904.4 Corrosion protection of reinforcement.** Reinforcement in concrete shall be protected from corrosion and exposure to chlorides in accordance with ACI 318, Section 4.4.~~

5. Add new text as follows:

1904.5 Alternative cementitious materials for sulfate exposure. Alternative combinations of cementitious materials for use in sulfate-resistant concrete to those listed in ACI 318, Table 4.3.1.b shall be permitted in accordance with ACI 318, Section 4.5.1.

6. Revise as follows:

1907.7 Concrete protection for reinforcement. The minimum specified concrete cover for reinforcement shall comply with Sections 1907.7.1 through 1907.7.7.

1907.7.1 Cast-in-place concrete (nonprestressed). Minimum specified concrete cover shall be provided for reinforcement in nonprestressed, cast-in-place concrete construction in accordance with ACI 318, Section 7.7.1.

1907.7.2 Cast-in-place concrete (prestressed). The minimum specified concrete cover for prestressed and nonprestressed reinforcement, ducts and end fittings in cast-in-place prestressed concrete shall comply with ACI 318, Section 7.7.2.

1907.7.3 Precast concrete (manufactured under plant control conditions). The minimum specified concrete cover for prestressed and nonprestressed reinforcement, ducts and end fittings in precast concrete manufactured under plant control conditions shall comply with ACI 318, Section 7.7.3.

1907.7.4 Bundled bars. The minimum specified concrete cover for bundled bars shall comply with ACI 318, Section 7.7.4.

1907.7.5 Headed shear stud reinforcement. For headed shear stud reinforcement, the minimum specified concrete cover shall comply with ACI 318, Section 7.7.5.

~~1907.7.5~~ 1907.7.6 Corrosive environments. In corrosive environments or other severe exposure conditions, prestressed and nonprestressed reinforcement shall be provided with additional protection in accordance with ACI 318, Section ~~7.7.5~~ 7.7.6.

~~1907.7.6~~ 1907.7.7 Future extensions. Exposed reinforcement, inserts and plates intended for bonding with future extensions shall be protected from corrosion.

~~1907.7.7~~ 1907.7.8 Fire protection. When this code requires a thickness of cover for fire protection greater than the minimum concrete cover ~~specified~~ in Section 1907.7, such greater thickness shall be ~~used~~ specified.

1908.1 General. The text of ACI 318 shall be modified as indicated in Sections 1908.1.1 through ~~1908.1.16~~ 1908.1.9.

7**. Delete without substitution:**

~~**1908.1.1 ACI 318, Section 10.5.** Modify ACI 318, Section 10.5, by adding new Section 10.5.5 to read as follows:~~

~~*10.5.5 – In structures assigned to Seismic Design Category B, beams in ordinary moment frames forming part of the seismic-force-resisting system shall have at least two main flexural reinforcing bars continuously top and bottom throughout the beam and continuous through or developed within exterior columns or boundary elements.*~~

~~**1908.1.2ACI 318, Section 11.11.** Modify ACI 318, Section 11.11, by changing its title to read as shown below and by adding new Section 11.11.3 to read as follows:~~

~~*11.11– Special provisions for columns.*~~
~~*11.11.3 – In structures assigned to Seismic Design Category B, columns of ordinary moment frames having a clear height-to-maximum-plan-dimension ratio of five or less shall be designed for shear in accordance with 21.12.3.*~~

8. Revise as follows:

~~1908.1.3~~ 1908.1.1 ACI 318, Section ~~21.1~~ 2.2. Modify existing definitions and add the following definitions to ACI 318, Section ~~21.1~~ 2.2.

9. Add new definition as follows:

SPECIAL STRUCTURAL WALL. A cast-in-place or precast wall complying with the requirements of 21.2.3 through 21.2.7, 21.10, and 21.11, as applicable, in addition to the requirements for ordinary reinforced concrete structural walls *or ordinary precast structural walls, as applicable*. *Where ASCE 7 refers to a "special reinforced concrete structural wall," it shall be deemed to mean a "special structural wall."*

~~1908.1.4~~ 1908.1.2 ACI 318, Section ~~21.2.1~~ 21.1.1. Modify ACI 318 Sections ~~21.2.1.2, 21.2.1.3 and 21.2.1.4~~ 21.1.13 through 21.1.1.5, to read as follows:

10. Delete and substitute as follows:

~~*21.2.1.2 – For structures assigned to Seismic Design Category A or B*, provisions of Chapters 1 through 18 and 22 shall apply except as modified by the provisions of this chapter. Where the design seismic loads are computed using provisions for intermediate or special concrete systems, the requirements of Chapter 21 for intermediate or special systems, as applicable, shall be satisfied.~~

~~*21.2.1.3 – For structures assigned to Seismic Design Category C*, intermediate or special moment frames, intermediate precast structural walls *or ordinary or special reinforced concrete structural walls* shall be used to resist *seismic* forces induced by earthquake motions. Where the design seismic loads are computed using provisions for special concrete systems, the requirements of Chapter 21 for special systems, as applicable, shall be satisfied.~~

~~*21.2.1.4 – For structures assigned to Seismic Design Category D, E or F*, special moment frames, *special reinforced concrete structural walls*, diaphragms and trusses *and foundations* complying with 21.2 through 21.10 *or intermediate precast structural walls complying with* 21.13 shall be used to resist forces induced by earthquake motions. Members not proportioned to resist earthquake forces shall comply with 21.11.~~

21.1.1.3 – Structures assigned to SDC B shall comply with Chapters 1 through 19 and 22. For a structure assigned to SDC B using ordinary moment frames as part of the seismic-force resisting system, the provisions of 21.1.2 and 21.2 shall apply. For a structure assigned to SDC B and using intermediate or special systems, the applicable provisions of 21.1.1.3 through 21.1.7, and 21.3 through 21.10 shall also apply.

21.1.1.4 – Structures assigned to SDC C shall comply with Chapters 1 through 19, and the seismic-force-resisting system shall be intermediate or special moment frames, intermediate precise structural walls, or ordinary reinforced concrete or special structural walls. For a structure assigned to SDC C and using intermediate moment frames as part of the seismic-force-resisting system the provisions of 21.1.2 and 21.3 shall apply. For a structure assigned to SDC C and using special moment frames, or intermediate precast or special structural walls, the applicable provisions of 21.1.3 through 21.1.7, and 21.4 through 21.10 shall also apply. Any structure assigned to SDC C shall satisfy 21.1.8. *Except for footings, pedestals and basement walls in accordance with 22.10 or as permitted by the International Building Code, structural elements of plain concrete are prohibited.*

21.1.1.5 – Structures assigned to SDC D, E or F shall comply with Chapters 1 through 19, and the seismic-force-resisting system shall be special moment frames, intermediate precast structural walls, or special structural walls. For a structure assigned to SDC, D, E, or F, the provisions of 21.1.2 through 21.1.8 and 21.4 through 21.13 shall apply. *Except for footings, pedestals and basement walls in accordance with 22.10 or as permitted by the International Building Code, structural elements of plain concrete are prohibited.*

11. Delete without substitution:

~~**1908.1.5 ACI 318, Section 21.2.5.** Modify ACI 318, Section 21.2.5, by renumbering as Section 21.2.5.1 and adding new Section 21.2.5.2 to read as follows:~~

~~21.2.5 – Reinforcement in members resisting earthquake- induced forces.~~

~~*21.2.5.1 – Except as permitted in 21.2.5.2, reinforcement resisting earthquake-induced flexural and axial forces in frame members and in structural wall boundary elements shall comply with ASTM A 706. ASTM A 615, Grades 40 and 60 reinforcement, shall be permitted in these members if (a) the actual yield strength based on mill tests does not exceed the specified yield, fy, strength by more than 18,000 psi (124 MPa) [retests shall not exceed this value by more than an additional 3,000 psi (21 MPa)], and (b) the ratio of the actual tensile strength to the actual yield strength is not less than 1.25. For computing shear strength,* the value of *fyt* for transverse reinforcement, including spiral reinforcement, shall not exceed 60,000 psi (414 MPa).~~

~~*21.2.5.2 – Prestressing steel shall be permitted in flexural members of frames, provided the average prestress, fpc, calculated for an area equal to the member's shortes cross-sectional dimension multiplied by the perpendicular dimension shall be the lesser of 700 psi (4.83MPa) or fc /6 at locations of nonlinear action where prestressing steel is used in members of frames.*~~

~~**1908.1.6 ACI 318, Section 21.2.** Modify ACI 318, Section 21.2, by adding new Section 21.2.9 to read as follows:~~

~~*21.2.9 – Anchorages for unbonded post-tensioning tendons resisting earthquake induced forces in structures assigned to Seismic Design Category C, D, E or F shall withstand, without failure, 50 cycles of loading ranging between 40 and 85 percent of the specified tensile strength of the prestressing steel.*~~

~~**1908.1.7 ACI 318, Section 21.3.** Modify ACI 318, Section 21.3, by adding new Section 21.3.2.5 to read as follows:~~

~~*21.3.2.5 – Unless the special moment frame is qualified for use through structural testing as required by 21.6.3, for flexural members prestressing steel shall not provide more than one-quarter of the strength for either positive or negative moment at the critical section in a plastic hinge location and shall be anchored at or beyond the exterior face of a joint*~~*.*

12. Revise as follows:

~~**1908.1.13**~~ **1908.1.3 ACI 318, Section** ~~**21.13**~~ **21.4.** Modify ACI 318, ~~Section 21.13, by renumbering Section 21.13.3 to become 21.13.4 and adding new Sections 21.13.3, 21.13.5 and 21.13.6~~ Section 21.4, by renumbering Section 21.4.3 to become 21.4.4 and adding new Sections 21.4.3, 21.4.5 and 21.4.6 to read as follows:

~~21.13.3~~ 21.4.3~~–~~ ~~Except for Type 2 mechanical splices, connection elements~~ Connections that are designed to yield shall be capable of maintaining 80 percent of their design strength at the deformation induced by the design displacement or shall use Type 2 mechanical splices.

~~21.13.4~~ 21.4.4– Elements of the connection that are not designed to yield shall develop at least 1.5 *Sy*.

~~21.13.5~~ 21.4.5 – Wall piers not designed as part of a moment frame shall have transverse reinforcement designed to resist the shear forces determined from ~~21.12.3~~ 21.3.3. Spacing of transverse reinforcement shall not exceed 8 inches (203 mm). Transverse reinforcement shall be extended beyond the pier clear height for at least 12 inches (305 mm).

Exceptions:

1. Wall piers that satisfy ~~21.11~~ 21.13.

2. Wall piers along a wall line within a story where other shear wall segments provide lateral support to the wall piers and such segments have a total stiffness of at least six times the sum of the stiffnesses of all the wall piers.

~~21.13.6~~ 21.4.6~~–~~ Wall segments with a horizontal length-to-thickness ratio less than 2.5 shall be designed as columns.

~~1908.1.8~~ 1908.1.4 ACI 318, Section ~~21.7~~ 21.9 Modify ACI 318, Section ~~21.7~~ 21.9, by adding new Section ~~21.7.10~~ 21.9.10 to read as follows:

~~21.7.10~~ 21.9.10~~–~~ Wall piers and wall segments.

~~21.7.10.1~~ 21.9.10.1 –Wall piers not designed as a part of a special moment frame shall have transverse reinforcement designed to satisfy the requirements in ~~21.7.10.2~~ 21.9.10.2.

> ***Exceptions:***
> *1. Wall piers that satisfy ~~21.11~~ 21.13.*
> *2. Wall piers along a wall line within a story where other shear wall segments provide lateral support to the wall piers and such segments have a total stiffness of at least six times the sum of the stiffness of all the wall piers.*

~~21.7.10.2~~ 21.9.10.2 – Transverse reinforcement with seismic hooks at both ends shall be designed to resist the shear forces determined from ~~21.4.5.1~~ 21.6.5.1. Spacing of transverse reinforcement shall not exceed 6 inches (152 mm). Transverse reinforcement shall be extended beyond the pier clear height for at least 12 inches (305 mm).

~~21.7.10.3~~ 21.9.10.3 – Wall segments with a horizontal length-to-thickness ratio less than 2.5 shall be designed as columns.

~~1908.1.9~~ 1908.1.5 ACI 318, Section ~~21.8~~ 21.10. Modify ACI 318, Section ~~21.8.1~~ 21.10.2 , to read as follows:

~~21.8.1~~ 21.10.2 – Special structural walls constructed using precast concrete shall satisfy all the requirements of ~~21.7~~ 21.9 for cast-in-place special structural walls in addition to Sections ~~21.13.2 through 21.13.4~~ 21.4.2 through 21.4.4.

~~1908.1.10~~ 1908.1.6 ACI 318, Section ~~21.10.1.1~~ 21.12.1.1. Modify ACI 318, Section ~~21.10.1.1~~ 21.12.1.1, to read as follows:

~~21.10.1.1~~ 21.12.1.1– Foundations resisting earthquake-induced forces or transferring earthquake-induced forces between a structure and the ground shall comply with the requirements of Section ~~21.10~~ 21.12 and other applicable provisions of ACI 318 *unless modified by Chapter 18 of the International Building Code*.

13. Delete without substitution:

~~**1908.1.11 ACI 318, Section 21.11.** Modify ACI 318, Section 21.11.2.2 to read as follows:~~

~~21.11.2.2 – Members with factored gravity axial forces exceeding (Agf _c/10) shall satisfy 21.4.3, 21.4.4.1(c), 21.4.4.3 and 21.4.5. The maximum longitudinal spacing of ties shall be so for the full column height. Spacing, so, shall not exceed the smaller of six diameters of the smallest longitudinal bar enclosed and 6 inches (152 mm). Lap splices of longitudinal reinforcement in such members need not satisfy 21.4.3.2 in structures where the seismic- force-resisting system does not include special moment frames.~~

~~**Section 1908.1.12ACI 318, Section 21.12.5**. Modify ACI 318, Section 21.12.5, by adding new Section 21.12.5.6 to read as follows:~~

~~21.12.5.6 – Columns supporting reactions from discontinuous stiff members, such as walls, shall be designed for the special load combinations in Section 1605.4 of the International Building Code and shall be provided with transverse reinforcement at the spacing, so, as defined in 21.12.5.2 over their full height beneath the level at which the discontinuity occurs. This transverse reinforcement shall be extended above and below the column as required in 21.4.4.5.~~

14. Revise as follows:

~~1908.1.14~~ 1908.1.7 ACI 318, Section 22.6. Modify ACI 318, Section 22.6, by adding new Section 22.6.7 to read:

22.6.7 – Detailed plain concrete structural walls.

22.6.7.1 – Detailed plain concrete structural walls are walls conforming to the requirements of ordinary structural plain concrete walls and 22.6.7.2.

22.6.7.2 - Reinforcement shall be provided as follows:

(a) Vertical reinforcement of at least 0.20 square inch (129 mm2) in cross-sectional area shall be provided continuously from support to support at each corner, at each side of each opening and at the ends of walls. The continuous vertical bar required beside an opening is permitted to substitute for one of the two No. 5 bars required by 22.6.6.5.

(b) Horizontal reinforcement at least 0.20 square inch (129 mm2) in cross-sectional area shall be provided:

1. *Continuously at structurally connected roof and floor levels and at the top of walls;*
2. *At the bottom of load-bearing walls or in the top of foundations where doweled to the wall; and*
3. *At a maximum spacing of 120 inches (3048mm).*

Reinforcement at the top and bottom of openings, where used in determining the maximum spacing specified in Item 3 above, shall be continuous in the wall.

~~1908.1.15~~ <u>1908.1.8</u> ACI 318, Section 22.10. Delete ACI 318, Section 22.10, and replace with the following:

22.10 – Plain concrete in structures assigned to Seismic Design Category C, D, E or F.

22.10.1 – Structures assigned to Seismic Design Category C, D, E or F shall not have elements of structural plain concrete, except as follows:

(a) Structural plain concrete basement, foundation or other walls below the base are permitted in detached one- and two-family dwellings three stories or less in height constructed with stud-bearing walls. In dwellings assigned to Seismic Design Category D or E, the height of the wall shall not exceed 8 feet (2438 mm), the thickness shall not be less than 71/2 inches (190 mm), and the wall shall retain no more than 4 feet (1219 mm) of unbalanced fill. Walls shall have reinforcement in accordance with 22.6.6.5.

(b) Isolated footings of plain concrete supporting pedestals or columns are permitted, provided the projection of the footing beyond the face of the supported member does not exceed the footing thickness.

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

(c) Plain concrete footings supporting walls are permitted, provided the footings have at least two continuous longitudinal reinforcing bars. Bars shall not be smaller than No. 4 and shall have a total area of not less than 0.002 times the gross cross-sectional area of the footing. For footings that exceed 8 inches (203 mm) in thickness, a minimum of one bar shall be provided at the top and bottom of the footing. Continuity of reinforcement shall be provided at corners and intersections.

Exceptions:

1. *In detached one- and two-family dwellings three stories or less in height and constructed with stud-bearing walls, plain concrete footings without longitudinal reinforcement supporting walls are permitted.*
2. *For foundation systems consisting of a plain concrete footing and a plain concrete stem wall, a minimum of one bar shall be provided at the top of the stem wall and at the bottom of the footing.*
3. *Where a slab on ground is cast monolithically with the footing, one No. 5 bar is permitted to be located at either the top of the slab or bottom of the footing.*

15. Delete without substitution:

~~**1908.1.16 (Supp) ACI 318, Section D.3.3.** Modify ACI 318, Sections D.3.3.2 through D.3.3.5 to read as follows:~~

~~D.3.3.2 B *In structures assigned to Seismic Design Category C, D, E or F,* post-installed anchors for use under D.2.3 shall have passed the Simulated Seismic Tests of ACI 355.2.~~

~~D.3.3.3 B *In structures assigned to Seismic Design Category C, D, E or F,* the design strength of anchors shall be taken as 0.75nN_n and 0.75nV_n, where n is given in D.4.4 or D.4.5, and N_n and V_n are determined in accordance with D.4.1. D.3.3.4 B *In structures assigned to Seismic Design Category C, D, E or F,* anchors shall be designed to be governed by tensile or shear strength of a ductile steel element, unless D.3.3.5 is satisfied.~~

~~***Exception:** Anchors in concrete designed to support nonstructural components in accordance with ASCE 7 Section 13.4.2 need not satisfy Section D.3.3.4.*~~

~~D.3.3.5 B Instead of D.3.3.4, the attachment that the anchor is connecting to the structure shall be designed so that the attachment will undergo ductile yielding at a load level corresponding to anchor forces not greater that the design strength of anchor specified in D.3.3.3, *or the minimum design strength of the anchor shall be at least 2.5 times the factored forces transmitted by the attachment*.~~

~~***Exception:** Anchors in concrete designed to support nonstructural components in accordance with ASCE 7 Section 13.4.2 need not satisfy Section D.3.3.5.*~~

16. Add new text as follows:

1908.1.9 ACI 318, Section D.4.2.2. Modify ACI 318, Sections D.4.2.2 to read as follows:

D.4.2.2 – The concrete breakout strength requirements for anchors shall be considered satisfied by the design procedure of D.5.2 and D.6.2.

17. Revise as follows:

1909.6.1 Basement walls. The thickness of exterior basement walls and foundation walls shall be not less than 71/2 inches (191 mm). ~~Structural plain concrete exterior basement walls shall be exempt from the requirements for special exposure conditions of Section 1904.2.2.~~

1909.6.3 Openings in walls. Not less than ~~two~~ one No. 5 ~~bars~~ bar shall be provided around window ~~and~~ door and similar sized openings. ~~Such bars~~ The bar shall ~~extend at least 24 inches (610 mm) beyond~~ be anchored to develop f_y in tension at the corners of openings.

1912.1 Scope. The provisions of this section shall govern the strength design of anchors installed in concrete for purposes of transmitting structural loads from one connected element to the other. Headed bolts, headed studs and hooked (J- or L-) bolts cast in concrete and expansion anchors and undercut anchors installed in hardened concrete shall be designed in accordance with Appendix D of ACI 318 as modified by Section 1908.1.16, provided they are within the scope of Appendix D.

~~**Exception**: Where the basic concrete breakout strength in tension of a single anchor, Nb, is determined in accordance with Equation (D-7), the concrete breakout strength requirements of Section D.4.2.2 shall be considered satisfied by the design procedures of Sections D.5.2 and D.6.2 for anchors exceeding 2 inches (51 mm) in diameter or 25 inches (635 mm) tensile embedment depth.~~

The strength design of anchors that are not within the scope of Appendix D of ACI 318, and as amended ~~above~~ in Section 1908.1.9, shall be in accordance with an approved procedure.

TABLE 1704.3
REQUIRED VERIFICATION AND INSPECTION OF STEEL CONSTRUCTION

VERIFICATION AND INSPECTION	CONTINUOUS	PERIODIC	REFERENCED STANDARD[a]	IBC REFERENCE
5. Inspection of welding: a. Structural Steel:	—	—		
1) Complete and partial penetration groove welds.	X	—	AWS D1.1	1704.3.1
2) Multipass fillet welds	X	—		
3) Single-pass fillet welds > 5/16”	X	—		
4) Single-pass fillet welds ≤ 5/16”	—	X		
5) Floor and roof deck welds.	—	X	AWS D1.3	—
b. Reinforcing steel:	—	—	AWS D1.4 ACI 318: 3.5.2	—
1) Verification of weldability of reinforcing steel other than ASTM A706	—	X		
2) Reinforcing steel-resisting flexural and axial forces in intermediate and special moment frames, and boundary elements of special ~~reinforced~~ structural walls of concrete ~~shear walls~~ and shear reinforcement.	X	—		
3) Shear reinforcement.	X	—		
4) Other reinforcing steel	—	X		

(Portions of table and footnotes not shown remain unchanged)

1708.3 Reinforcing and prestressing steel. Certified mill test reports shall be provided for each shipment of reinforcing steel used to resist flexural, shear and axial forces in reinforced concrete intermediate frames, special moment frames and boundary elements of special ~~reinforced~~ structural walls of concrete or special reinforced masonry shear walls. Where ASTM A 615 reinforcing steel is used to resist earthquake-induced flexural and axial forces in special moment frames and in wall boundary elements of shear walls in structures assigned to Seismic Design Category D, E or F, as determined in Section 1613, the testing requirements of ACI 318 shall be met. Where ASTM A 615 reinforcing steel is to be welded, chemical tests shall be performed to determine weldability in accordance with Section 3.5.2 of ACI 318.

Reason: Item 1: The changes proposed are necessary to update the concrete provisions to be consistent with the provisions of ACI 318-08.

Item 2: The changes proposed are necessary to update or delete as necessary, the modifications to ACI 318 for consistency with the provisions of ACI 318-08. The following modifications are being deleted because the modifications have been incorporated into ACI 318-08. Changes to individual sections are indicated below. Section numbers cited are based on the number in the 2006 IBC, unless noted otherwise.

1908.1.1 – This modification has been incorporated into ACI 318-08, Section 21.2.2.

1908.1.2 – This modification has been incorporated into ACI 318-08, Section 21.2.3.

1908.1.3 – The definition of "special structural wall" from ACI 318-08 has been included and modified for the following reasons. In ACI 318-05, a "special reinforced concrete structural wall" was a cast-in-place wall, and a separate definition applied to a "special precast structural wall." Under ACI 318-08, the definition of "special precast structural wall" has been deleted, and the former definition of "special reinforced concrete structural wall" has been revised to "special structural wall" which is now defined as "a cast-in-place or precast wall." The modification to the ACI 318-08 definition of special structural wall, by adding another sentence, is necessary since ASCE 7-05 Table 12.2-1 – Design Coefficients and Factors for Seismic Force-Resisting Systems – uses the ACI 318-05 term "special reinforced concrete structural wall." In addition, it does not mention "special structural wall." Therefore, the sentence is being added to coordinate new terminology used in ACI 318-08 with that used in ASCE 7-05.

1980.1.4 - These modifications to ACI 318-08 are necessary because ACI 318-08 does not indicate the seismic-force-resisting systems permitted in the various seismic design categories.

1908.1.5 – This modification has been incorporated into ACI 318-08, Section 21.5.2.5(a).

1908.1.6 – This modification has been incorporated into ACI 318-08, Section 21.5.2.5(d).

1908.1.7 – This modification has been incorporated into ACI 318-08, Section 21.5.2.5(c).

1908.1.11 - This modification has been incorporated into ACI 318-08, Section 21.13.3.2 since it only references 21.6.3.1 and does not reference 21.6.3.2.

1908.1.12 – This modification has been incorporated into ACI 318-08, Section 21.3.5.6.

1908.1.16 – This modification has been incorporated into ACI 318-08, Section D.3.3.

1908.1.9 (new) – This modification to Section D.4.2.2 of ACI 318 is already in Section 1912.1 as an exception. It is being relocated to Section 1908 to consolidate all ACI 318 modifications in one section. Also, see "reason" for item 4.

All other proposed changes are necessary because Chapter 21 of ACI 318-08 has been reformatted which resulted in section numbers being changed.

Item 3: The changes proposed are necessary to update the structural plain concrete provisions to be consistent with the provisions of ACI 318-08.

Item 4: The changes proposed are necessary because the modification to ACI 318-05 in Section 1908.1.16 has been incorporated into ACI 318-08; therefore, the modification is no longer needed. In addition, the modification to ACI 318, Section D.4.2.2 in the exception is being relocated to Section 1908.1.9.

Item 5 – For consistency with the change in ACI 318-08 from "special reinforced concrete structural wall" to "Special structural wall." Also, see reason for item 1, Section 1908.1.3.

Cost Impact: The code change proposal will not increase the cost of construction.

Public Hearing:				
	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

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1908.1.16

Proponent: John W. Lawson, SE, representing the Structural Engineers Association of California Seismology Tilt-up Subcommittee; David L. McCormick, SE, representing the Structural Engineers Association of California Existing Building Tilt-up Subcommittee

Revise as follows:

1908.1.16 (Supp) ACI 318, Section D.3.3. Modify ACI 318, Sections D.3.3.2 through D.3.3.5 to read as follows:

D.3.3.2 - *In structures assigned to Seismic Design Category C, D, E or F,* post-installed anchors for use under D.2.3 shall have passed the Simulated Seismic Tests of ACI 355.2.

D.3.3.3 - *In structures assigned to Seismic Design Category C, D, E or F,* the design strength of anchors shall be taken as $0.75\Phi N_n$ and $0.75\Phi V_n$, where Φ is given in D.4.4 or D.4.5, and N_n and V_n are determined in accordance with D.4.1.

D.3.3.4 - *In structures assigned to Seismic Design Category C, D, E or F,* anchors shall be designed to be governed by tensile or shear strength of a ductile steel element, unless D.3.3.5 is satisfied.

Exceptions:

1. *Anchors in concrete designed to support nonstructural components in accordance with ASCE 7 Section 13.4.2 need not satisfy Section D.3.3.4*.
2. Concrete wall anchorage designed to the forces of ASCE 7 Equation 12.11-1 need not satisfy Section D.3.3.4.

D.3.3.5 - Instead of D.3.3.4, the attachment that the anchor is connecting to the structure shall be designed so that the attachment will undergo ductile yielding at a load level corresponding to anchor forces not greater that the design strength of anchor specified in D.3.3.3, *or the minimum design strength of the anchor shall be at least 2.5 times the factored forces transmitted by the attachment*.

Exceptions:

1. *Anchors in concrete designed to support nonstructural components in accordance with ASCE 7 Section 13.4.2 need not satisfy Section D.3.3.5*.
2. Concrete wall anchorage designed to the forces of ASCE 7 Equation 12.11-1 need not satisfy Section D.3.3.5.

Reason: Purpose: To remove the requirements for anchorage ductility at concrete wall anchorage already designed for maximum expected seismic forces.

Background: Following the 1994 Northridge earthquake, surveys of damage to concrete and masonry buildings with flexible roof diaphragms revealed that very limited amounts of wall anchorage ductility was present to resist the induced forces. Brittle tensile failures in steel wall anchorage straps were especially troublesome. In addition, boundary nailing in plywood diaphragms tore out of the plywood edges due to wall anchorage elongation. New code provisions were introduced into the 1997 UBC to address the nonductile wall anchorage behavior observed in the Northridge earthquake.

1997 UBC Section 1633.2.8.1 was chiefly written to address many of the wall anchorage issues spotlighted in the Northridge earthquake. The lack of observed ductility and the need for greater anchorage strength were the reasons behind Section 1633.2.8.1 Items 1 and 4. Wall anchorage forces to flexible diaphragms in Seismic Zones 3 & 4 were increased 50% (a_p=1.5) and steel elements had an additional 1.4 force multiplier.

The intent of Section 1633.2.8.1 (items 1, 4, and 5) was for the wall anchorage system to resist brittle failure when subjected to maximum expected roof accelerations. Based on observations of Northridge earthquake damage, it was deemed best to resist brittle failure through the use of significantly higher design forces in conjunction with anticipated material overstrength instead of any reliance on ductility. As a result, material-specific load factors were introduced to provide a uniform level of protection against brittle failure (1.4 steel, 0.85 wood, 1.0 concrete/masonry). This approach is well documented in the 1999 SEAOC Recommended Lateral Force Requirements and Commentary (The Blue Book) [Reference C108.2.8.1].

As further evidence of the intent of these wall anchorage provisions, the 1999 SEAOC Blue Book Commentary states that the reduced R_p value for nonductile and shallow anchorage does not apply to wall anchorage designed using this overstrength approach of Section 1633.2.8.1 [Reference C108.2.8.1].

Current Provisions: In the development of ASCE 7-05, the intent was to maintain the same wall anchorage equation between the 1997 UBC and ASCE 7-05 for flexible diaphragms in high seismic zones. The wall anchorage provisions of ASCE 7-05 Section 12.11.2.1 are directly incorporated from the 1997 UBC Section 1633.2.8.1. Substituting $C_a = 0.4S_{DS}$ (2003 NEHRP Commentary), it can be confirmed that Eq. 12.11-1 is generally equivalent to the 1997 UBC.

Through an unrelated parallel effort, ACI 318-05 Appendix D Sections D.3.3.4 and D.3.3.5 require anchorage ductility in moderate and high seismic zones. ACI's ductility requirement conflicts with the intent behind Section 12.11.2.1 at wall anchorage situations. Furthermore, 2006 IBC Section 1908.1.16 allows an additional 2.5 load factor on top of ASCE forces in lieu of the ACI ductility requirement. This stacking of load factors on top of load factors and ductility requirements is in conflict with the original intent of the wall anchorage provisions.

To summarize, the 1997 UBC and subsequent ASCE 7-05 implement very high wall anchorage force levels to achieve uniform protection against brittle failure without reliance upon ductility. This was achieved using a rational approach considering inherent overstrength. Through the incorporation of ACI 318 Appendix D, anchorage ductility requirements were inadvertently added to these special wall anchorage situations in conflict with the original intent of the provisions. Furthermore, the 2006 IBC force multiplier of 2.5 is redundant to the original force increase behind the UBC and ASCE wall anchorage provisions.

Impact to Design & Construction: Achieving anchorage ductility under ACI 318 Appendix D is very difficult for tilt-up construction with flexible diaphragms. For the ductility condition to be met, steel anchor strength must be weaker than the concrete breakout strength. Because tilt-up walls are inherently thin slender wall designs, anchor embedment depth is limited, making it difficult to increase. In several parametric studies, it is apparent that the ductility provision encourages smaller diameter steel anchors or thicker concrete walls for deeper embedments. Neither of these approaches seems beneficial.

Another unintended consequence of providing ductile anchorage is the potential elongation of the steel causing boundary nailing at plywood diaphragms to tear out of the sheathing edges under maximum seismic force levels. Similar concerns exist for edge welding along steel deck diaphragms at the wall panels.

Using the 2006 IBC 1908.1.16 alternative, the forces are increased to an extreme level due to the 2.5-times load increase previously mentioned. In several parametric studies, this results in a larger number of thin anchors rods spread out over a larger connection area. Spreading these anchor rods out will likely result in non-uniform anchorage force distribution, and instead concentrate the forces over the closest few rods, potentially resulting in a progressive rod failure.

References:

1. SEAOC, *Recommended Lateral Force Requirements and Commentary*, Structural Engineers Association of California, 1999.
2. HARRIS, "Response of Tilt-up Buildings to Seismic Demands: Case Studies from the 1994 Northridge Earthquake," By S.K. Harris, R.O. Hamburger, S.C. Martin, D.L. McCormick, and P.G. Somerville. *Proceedings of the NEHRP Conference and Workshop of Research on the Northridge, California Earthquake of January 17, 1994.* California Universities for Research in Earthquake Engineering (CUREE), Richmond, California. 1998.

Cost Impact: The code change proposal will not increase the cost of construction.

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

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1405.9, 1405.5.2, 1405.9, 1604.3.4, 1704.5, 1704.5.1 through 1704.5.3, Table 1704.5.1, Table 1704.5.3, 1708.1.1 through 1708.1.4, 1812.7, 2101.2.2 through 2101.2.6, 2106.1 through 2106.1.1.3, 2106.3, 2016.1.1.3.1, 2106.1.1.3.2, 2106.2, 2106.3, 2106.4, 2106.5, 2106.6, 2107.1 through 2107.8 through 2108.4, 2109.1, 2109.2.3.1, 2109.7.3

Proponent: Phillip Samblanet, The Masonry Society

1. Revise as follows:

1405.5 Anchored masonry veneer. Anchored masonry veneer shall comply with the provisions of Sections 1405.5, 1405.6, 1405.7 and 1405.8 and Sections 6.1 and 6.2 of ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5.

1405.5.1 Tolerances. Anchored masonry veneers in accordance with Chapter 14 are not required to meet the tolerances in Article 3.3 G1 of ~~ACI 530.1/ASCE 6/TMS 602~~ TMS 602/ACI 530.1/ASCE 6.

1405.5.2 (Supp) Seismic requirements. Anchored masonry veneer located in Seismic Design Category C, D, E or F shall conform to the requirements of Section 6.2.2.10 of ~~ACI 530/ASCE 5/ TMS 402~~ TMS 402/ACI 530/ASCE 5. Anchored masonry veneer located in Seismic Design Category D shall also conform to the requirements of Section 6.2.2.10.3.3 of ~~ACI 530/ ASCE 5/ TMS 402~~ TMS 402/ACI 530/ASCE 5.

1405.9 Adhered masonry veneer. Adhered masonry veneer shall comply with the applicable requirements in Section 1405.9.1 and Sections 6.1 and 6.3 of ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5.

1604.3.4 Masonry. The deflection of masonry structural members shall not exceed that permitted by ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5.

1704.5 Masonry construction. Masonry construction shall be inspected and evaluated in accordance with the requirements of Sections 1704.5.1 through 1704.5.3, depending on the classification of the building or structure or nature of the occupancy, as defined by this code.

Exception: Special inspections shall not be required for:

1. Empirically designed masonry, glass unit masonry or masonry veneer designed by Section 2109, 2110 or Chapter 14, respectively, or by Chapter 5, 7 or 6 of ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5, respectively, when they are part of structures classified as Occupancy Category I, II or III in accordance with Section 1604.5.
2. Masonry foundation walls constructed in accordance with Table 1805.5(1), 1805.5(2), 1805.5(3) or 1805.5(4).
3. Masonry fireplaces, masonry heaters or masonry chimneys installed or constructed in accordance with Section 2111, 2112 or 2113, respectively.

1704.5.1 Empirically designed masonry, glass unit masonry and masonry veneer in Occupancy Category IV. The minimum special inspection program for empirically designed masonry, glass unit masonry or masonry veneer designed by Section 2109, 2110 or Chapter 14, respectively, or by Chapter 5, 7 or 6 of ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5, respectively, in structures classified as Occupancy Category IV, in accordance with Section 1604.5, shall comply with Table 1704.5.1.

TABLE 1704.5.1
LEVEL 1 SPECIAL INSPECTION

INSPECTION TASK	FREQUENCYOF INSPECTION		REFERENCE FOR CRITERIA		
	Continuous during task listed	Periodically during task listed	IBC section	~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5 [a]	~~ACI 530.1/ASCE 6/TMS 602a~~ TMS 602/ACI 530.1/ASCE 6
2. The inspection program shall verify:					
a. Size and location of structural elements.	—	X	—	—	Art. ~~3.3G~~ 3.3F
b. Type, size and location of anchors, including other details of anchorage of masonry to structural members, frames or other construction.	—	X	—	Sec. 1.2.2(e), ~~2.1.4, 3.1.6~~, 1.16.1	—
c. Specified size, grade and type of reinforcement	—	X	—	Sec. ~~1.13~~ 1.15	Art. 2.4, 3.4
d. Welding of reinforcing bars.	X	—	—	Sec. ~~2.1.10.7.2~~ 2.1.9.7.2, 3.3.3.4(b)	—

(Portions of table not shown remain unchanged)

1704.5.2 Engineered masonry in Occupancy Category I, II or III. The minimum special inspection program for masonry designed by Section 2107 or 2108 or by chapters other than Chapters 5, 6 or 7 of ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5 in structures classified as Occupancy Category I, II or III, in accordance with Section 1604.5, shall comply with Table 1704.5.1.

1704.5.3 Engineered masonry in Occupancy Category IV. The minimum special inspection program for masonry designed by Section 2107 or 2108 or by chapters other than Chapters 5, 6 or 7 of ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5 in structures classified as Occupancy Category IV, in accordance with Section 1604.5, shall comply with Table 1704.5.3.

TABLE 1704.5.3
LEVEL 2 SPECIAL INSPECTION

INSPECTION TASK	FREQUENCYOF INSPECTION		REFERENCE FOR CRITERIA		
	Continuous during task listed	Periodically during task listed	IBC section	~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5 [a]	~~ACI 530.1/ASCE 6/TMS 602a~~ TMS 602/ACI 530.1/ASCE 6
1. From the beginning of masonry construction, the following shall be verified to ensure compliance:					
c. Placement of reinforcement, connectors and prestressing tendons and anchorages.	—	X	—	Sec. ~~1.13~~ 1.15	Art. 3.4, 3.6A
2. The inspection program shall verify:					
a. Size and location of structural elements.	—	X	—	—	Art. ~~3.3G~~ 3.3F
b. Type, size and location of anchors, including other details of anchorage of masonry to structural members, frames or other construction.	X	—	—	Sec. 1.2.2(e), ~~2.1.4, 3.1.6~~, 1.16.1	—
c. Specified size, grade and type of reinforcement	—	X	—	Sec. ~~1.13~~ 1.15	Art. 2.4, 3.4
d. Welding of reinforcing bars.	X	—	—	Sec. ~~2.1.10.7.2~~ 2.1.9.7.2, 3.3.3.4(b)	—

(Portions of table not shown remain unchanged)

1708.1.1 Empirically designed masonry and glass unit masonry in Occupancy Category I, II or III. For masonry designed by Section 2109 or 2110 or by Chapter 5 or 7 of ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5 in structures classified as Occupancy Category I, II or III, in accordance with Section 1604.5, certificates of compliance used in masonry construction shall be verified prior to construction.

1708.1.2 Empirically designed masonry and glass unit masonry in Occupancy Category IV. The minimum testing and verification prior to construction for masonry designed by Section 2109 or 2110 or by Chapter 5 or 7 of ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5in structures classified as Occupancy Category IV, in accordance with Section 1604.5, shall comply with the requirements of Table 1708.1.2.

1708.1.3 Engineered masonry in Occupancy Category I, II or III. The minimum testing and verification prior to construction for masonry designed by Section 2107 or 2108 or by chapters other than Chapter 5, 6 or 7 of ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5 in structures classified as Occupancy Category I, II or III, in accordance with Section 1604.5, shall comply with Table 1708.1.2.

1708.1.4 Engineered masonry in Occupancy Category IV. The minimum testing and verification prior to construction for masonry designed by Section 2107 or 2108 or by chapters other than Chapter 5, 6 or 7 of ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5in structures classified as Occupancy Category IV, in accordance with Section 1604.5, shall comply with Table 1708.1.4.

1805.5.2.2 Masonry foundation walls. Masonry foundation walls shall comply with the following:

1. Vertical reinforcement shall have a minimum yield strength of 60,000 psi (414 MPa).
2. The specified location of the reinforcement shall equal or exceed the effective depth distance, *d*, noted in Tables 1805.5(2), 1805.5(3) and 1805.5(4) and shall be measured from the face of the exterior (soil) side of the wall to the center of the vertical reinforcement. The reinforcement shall be placed within the tolerances specified in ~~ACI 530.1/ASCE 6/TMS 402, Article 3.4 B7~~ TMS 602/ACI 530.1/ASCE 6, Article 3.3 B.8 of the specified location.

(Portions not shown remain unchanged)

1812.7 Masonry. Where the unsupported height of foundation piers exceeds six times the least dimension, the allowable working stress on piers of unit masonry shall be reduced in accordance with ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5.

2101.2.2 (Supp) Strength design. Masonry designed by the strength design method shall comply with the provisions of Sections 2106 and 2108, except that autoclaved aerated concrete (AAC) masonry shall comply with the provisions of Section 2106, Section 1613.6.3 and Chapter 1 and Appendix A of ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5.

2101.2.3 Prestressed masonry. Prestressed masonry shall be designed in accordance with Chapters 1 and 4 of ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5 and Section 2106. Special inspection during construction shall be provided as set forth in Section 1704.5.

2101.2.4 Empirical design. Masonry designed by the empirical design method shall comply with the provisions of Sections 2106 and 2109 or Chapter 5 of ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5.

2101.2.5 Glass unit masonry. Glass unit masonry shall comply with the provisions of Section 2110 or Chapter 7 of ~~ACI 530/ASCE 5/ TMS 402~~ TMS 402/ACI 530/ASCE 5.

2101.2.6 Masonry veneer. Masonry veneer shall comply with the provisions of Chapter 14 or Chapter 6 of ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5.

2103.13.6 Prestressing tendons. Prestressing tendons shall conform to one of the following standards:

1. Wire..ASTM A 421
2. Low-relaxation wire ....ASTM A 421
3. Strand..ASTM A 416
4. Low-relaxation strand...ASTM A 416
5. Bar...ASTM A 722

Exceptions:

1. Wire, strands and bars not specifically listed in ASTM A 421, ASTM A 416 or ASTM A 722 are permitted, provided they conform to the minimum requirements in ASTM A 421, ASTM A 416 or ASTM A 722 and are approved by the architect/engineer.
2. Bars and wires of less than 150 kips per square inch (ksi) (1034 MPa) tensile strength and conforming to ASTM A 82, ASTM A 510, ASTM A 615, ASTM A 996 or ASTM A 706 are permitted to be used as prestressed tendons, provided that:
   2.1. The stress relaxation properties have been assessed by tests according to ASTM E 328 for the maximum permissible stress in the tendon.
   2.2. Other nonstress-related requirements of ~~ACI 530/ASCE 5/ TMS 402~~ TMS 402/ACI 530/ASCE 5, Chapter 4, addressing prestressing tendons are met.

2103.13.7 Corrosion protection. Corrosion protection for prestressing tendons shall comply with the requirements of ~~ACI 530.1/ASCE 6/TMS 602~~ TMS 602/ACI 530.1/ASCE 6, Article 2.4G. Corrosion protection for prestressing anchorages, couplers and end blocks shall comply with the requirements of ~~ACI 530.1/ASCE 6/TMS 602~~ TMS 602/ACI 530.1/ASCE 6, Article 2.4H. Corrosion protection for carbon steel accessories used in exterior wall construction or interior walls exposed to a mean relative humidity exceeding 75 percent shall comply with either Section 2103.13.7.2 or 2103.13.7.3. Corrosion protection for carbon steel accessories used in interior walls exposed to a mean relative humidity equal to or less than 75 percent shall comply with either Section 2103.13.7.1, 2103.13.7.2 or 2103.13.7.3.

2104.1 Masonry construction. Masonry construction shall comply with the requirements of Sections 2104.1.1 through 2104.5 and with ~~ACI 530.1/ASCE 6/TMS 602~~ TMS 602/ACI 530.1/ASCE 6.

2104.1.1 Tolerances. Masonry, except masonry veneer, shall be constructed within the tolerances specified in ~~ACI 530.1/ASCE 6/TMS 602~~ TMS 602/ACI 530.1/ASCE 6.

2104.3 Cold weather construction. The cold weather construction provisions of ~~ACI 530.1/ASCE 6/TMS 602~~ TMS 602/ACI 530.1/ASCE 6, Article 1.8 C, or the following procedures shall be implemented when either the ambient temperature falls below 40°F (4°C) or the temperature of masonry units is below 40°F (4°C).

2104.4 Hot weather construction. The hot weather construction provisions of ~~ACI 530.1/ASCE 6/TMS 602~~ TMS 602/ACI 530.1/ASCE 6, Article 1.8 D, or the following procedures shall be implemented when the temperature or the temperature and wind-velocity limits of this section are exceeded.

2106.1 Seismic design requirements for masonry. Masonry structures and components shall comply with the requirements in Section ~~1.14.2.2~~ 1.17.4.3.2 and Section ~~1.14.3, 1.14.4, 1.14.5, 1.14.6 or 1.14.7~~ 1.17.4.1, 1.17.4.2, 1.17.4.3, 1.17.4.4, or 1.17.4.5 of ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5 depending on the structure's seismic design category as determined in Section 1613. All masonry walls, unless isolated on three edges from in-plane motion of the basic structural systems, shall be considered to be part of the seismic-force-resisting system. In addition, the following requirements shall be met.

2106.1.1 Basic seismic-force-resisting system. Buildings relying on masonry shear walls as part of the basic seismic-force-resisting system shall comply with Section ~~1.14.2.2~~ 1.17.4.3.2 of ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5 or with Section 2106.1.1.1, 2106.1.1.2 or 2106.1.1.3.

2. Delete without substitution as follows:

~~**2106.1.1.1 Ordinary plain prestressed masonry shear walls.** Ordinary plain prestressed masonry shear walls shall comply with the requirements of Chapter 4 of ACI 530/ASCE 5/TMS 402 TMS 402/ACI 530/ASCE 5.~~

~~**2106.1.1.2 Intermediate prestressed masonry shear walls.** Intermediate prestressed masonry shear walls shall comply with the requirements of Section 1.14.2.2.4 of ACI 530/ASCE 5/TMS 402 TMS 402/ACI 530/ASCE 5 and shall be designed by Chapter 4, Section 4.4.3, of ACI 530/ASCE 5/TMS 402 TMS 402/ACI 530/ASCE 5 for flexural strength and by Section 3.3.4.1.2 of ACI 530/ASCE 5/TMS 402 TMS 402/ACI 530/ASCE 5 for shear strength. Sections 1.14.2.2.5, 3.3.3.5 and 3.3.4.3.2(c) of ACI 530/ASCE 5/TMS 402 TMS 402/ACI 530/ASCE 5 shall be applicable for reinforcement. Flexural elements subjected to load reversals shall be symmetrically reinforced. The nominal moment strength at any section along a member shall not be less than one-fourth the maximum moment strength. The cross-sectional area of bonded tendons shall be considered to contribute to the minimum reinforcement in Section 1.14.2.2.4 of ACI 530/ASCE 5/TMS 402 TMS 402/ACI 530/ASCE 5. Tendons shall be located in cells that are grouted the full height of the wall.~~

~~**2106.1.1.3 Special prestressed masonry shear walls.** Special prestressed masonry shear walls shall comply with the requirements of Section 1.14.2.2.5 of ACI 530/ASCE 5/TMS 402 TMS 402/ACI 530/ASCE 5 and shall be designed by Chapter 4, Section 4.4.3, of ACI 530/ASCE 5/TMS 402 TMS 402/ACI 530/ASCE 5 for flexural strength and by Section 3.3.4.1.2 of ACI 530/ASCE 5/TMS 402 TMS 402/ACI 530/ASCE 5 for shear strength. Sections 1.14.2.2.5(a), 3.3.3.5 and 3.3.4.3.2(c) of ACI 530/ASCE 5/TMS 402 TMS 402/ACI 530/ASCE 5 shall be applicable for reinforcement. Flexural elements subjected to load reversals shall be symmetrically reinforced. The nominal moment strength at any section along a member shall not be less than one-fourth the maximum moment strength. The cross-sectional area of bonded tendons shall be considered to contribute to the minimum reinforcement in Section 1.14.2.2.5 of ACI 530/ASCE 5/TMS 402 TMS 402/ACI 530/ASCE 5.~~

~~**2106.1.1.3.1 Prestressing tendons.** Prestressing tendons shall consist of bars conforming to ASTM A 722.~~

~~**2106.1.1.3.2 Grouting.** All cells of the masonry wall shall be grouted.~~

3. Revise as follows:

2106.3 Seismic Design Category B. Structures assigned to Seismic Design Category B shall conform to the requirements of Section ~~1.14.4~~ 1.17.4.2 of ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5 and to the additional requirements of this section.

2106.4 Design Category C. Structures assigned to Seismic Design Category C shall conform to the requirements of Section 2106.3, Section ~~1.14.5~~ 1.17.4.3 of ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5 and the additional requirements of this section.

2106.5 Additional requirements for structures in Seismic Design Category D. Structures assigned to Seismic Design Category D shall conform to the requirements of Section 2106.4, Section ~~1.14.6~~ 1.17.4.4 of ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5 and the additional requirements of this section.

2106.6 Additional requirements for structures in Seismic Design Category E or F. Structures assigned to Seismic Design Category E or F shall conform to the requirements of Section 2106.5 and Section ~~1.14.7~~ 1.17.4.5 of ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5.

2107.1 General. The design of masonry structures using allowable stress design shall comply with Section 2106 and the requirements of Chapters 1 and 2 of ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5 except as modified by Sections 2107.2 through 2107.8.

2107.2 ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5**, Section 2.1.2, load combinations.** Delete Section 2.1.2.1.

~~**2107.3** ACI 530/ASCE 5/TMS 402 TMS 402/ACI 530/ASCE 5**, Section 2.1.3, design strength.** Delete Sections 2.1.3.4 through 2.1.3.4.3.~~

~~**2107.4**~~ **2107.3** ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5**, Section 2.1.6, columns.** Add the following text to Section 2.1.6:

2.1.6.6 Light-frame construction. Masonry columns used only to support light-frame roofs of carports, porches, sheds or similar structures with a maximum area of 450 square feet (41.8 m^2) assigned to Seismic Design Category A, B or Care permitted to be designed and constructed as follows:

1. Concrete masonry materials shall be in accordance with Section 2103.1 of the *International Building Code*. Clay or shale masonry units shall be in accordance with Section 2103.2 of the *International Building Code*.
2. The nominal cross-sectional dimension of columns shall not be less than 8 inches (203 mm).
3. Columns shall be reinforced with not less than one No. 4 bar centered in each cell of the column.
4. Columns shall be grouted solid.
5. Columns shall not exceed 12 feet (3658 mm) in height.
6. Roofs shall be anchored to the columns. Such anchorage shall be capable of resisting the design loads specified in Chapter 16 of the *International Building Code*.
7. Where such columns are required to resist uplift loads, the columns shall be anchored to their footings with two No. 4 bars extending a minimum of 24 inches (610 mm) into the columns and bent horizontally a minimum of 15 inches (381 mm) in opposite directions into the footings. One of these bars is permitted to be the reinforcing bar specified in Item 3 above. The total weight of a column and its footing shall not be less than 1.5 times the design uplift load.

~~2107.5~~ 2107.4 ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5**, Section ~~2.1.10.7.1.1~~ 2.1.9.7.1.1, lap splices.** Modify Section ~~2.1.10.7.1.1~~ 2.1.9.7.1.1 as follows:

~~2.1.10.7.1.1~~ 2.1.9.7.1.1 The minimum length of lap splices for reinforcing bars in tension or compression,

$l_d = 0.002 d_b f_s$ **(Equation 21-2)**

For SI: $l_d = 0.29 d_b f_s$

but not less than 12 inches (305 mm). In no case shall the length of the lapped splice be less than 40 bar diameters.

where:

d_b = Diameter of reinforcement, inches (mm).

f_s = Computed stress in reinforcement due to design loads, psi (MPa).

In regions of moment where the design tensile stresses in the reinforcement are greater than 80 percent of the allowable steel tension stress, F_s, the lap length of splices shall be increased not less than 50 percent of the minimum required length. Other equivalent means of stress transfer to accomplish the same 50 percent increase shall be permitted.

Where epoxy coated bars are used, lap length shall be increased by 50 percent.

~~2107.6~~ 2107.5 ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5**, Section ~~2.1.10.7~~ 2.1.9.7, splices of reinforcement.** Modify Section ~~2.1.10.7~~ 2.1.9.7 as follows:

~~2.1.10.7~~ 2.1.9.7 Splices of reinforcement. Lap splices, welded splices or mechanical splices are permitted in accordance with the provisions of this section. All welding shall conform to AWS D1.4. Reinforcement larger than No. 9 (M #29) shall be spliced using mechanical connections in accordance with Section 2.1.10.7.3.

~~2107.7~~ 2107.6 ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5**, Section 2.3.6, maximum bar size.** Add the following to Chapter 2:

2.3.6 Maximum bar size. The bar diameter shall not exceed one-eighth of the nominal wall thickness and shall not exceed one-quarter of the least dimension of the cell, course or collar joint in which it is placed.

~~2107.8~~ 2107.7 ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5**, Section 2.3.7, maximum reinforcement percentage.** Add the following text to Chapter 2:

2.3.7 Maximum reinforcement percentage. Special reinforced masonry shear walls having a shear span ratio, *M/Vd*, equal to or greater than 1.0 and having an axial load, *P,* greater than $0.05 f'_m A_n$ that are subjected to in-plane forces shall have a maximum reinforcement ratio, ρ_{max}, not greater than that computed as follows:

$$\rho_{max} = \frac{n f'_m}{2 f_y \left(n + \frac{f_y}{f'_m} \right)} \qquad \textbf{(Equation 21-3)}$$

The maximum reinforcement ratio does not apply in the out-of-plane direction.

2108.1 General. The design of masonry structures using strength design shall comply with Section 2106 and the requirements of Chapters 1 and 3 of ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5, except as modified by Sections 2108.2 through 2108.4.

Exception: AAC masonry shall comply with the requirements of Chapter 1 and Appendix A of ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5.

2108.2 ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5**, Section 3.3.3.3 development.** Add the following text to Section 3.3.3.3:

The required development length of reinforcement shall be determined by Equation (3-15), but shall not be less than 12 inches (305 mm) and need not be greater than 72 d_b.

2108.3 ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5**, Section 3.3.3.4, splices.** Modify items (b) and (c) of Section 3.3.3.4 as follows:

3.3.3.4 (b). A welded splice shall have the bars butted and welded to develop at least 125 percent of the yield strength, f_y, of the bar in tension or compression, as required. Welded splices shall be of ASTM A 706 steel reinforcement. Welded splices shall not be permitted in plastic hinge zones of intermediate or special reinforced walls or special moment frames of masonry.

3.3.3.4 (c). Mechanical splices shall be classified as Type 1 or 2 according to Section 21.2.6.1 of ACI 318. Type 1 mechanical splices shall not be used within a plastic hinge zone or within a beam-column joint of intermediate or special reinforced masonry shear walls or special moment frames. Type 2 mechanical splices are permitted in any location within a member.

2108.4 ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5**, Section 3.3.3.5, maximum areas of flexural tensile reinforcement.** Add the following text to Section 3.3.3.5:

3.3.3.5.5 For special prestressed masonry shearwalls, strain in all prestressing steel shall be computed to be compatible with a strain in the extreme tension reinforcement equal to five times the strain associated with the reinforcement yield stress, f_y. The calculation of the maximum reinforcement shall consider forces in the prestressing steel that correspond to these calculated strains.

2109.1 General. Empirically designed masonry shall conform to this chapter or Chapter 5 of ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5.

2109.2.3.1 Strength. Dry-stacked, surface-bonded concrete masonry walls shall be of adequate strength and proportions to support all superimposed loads without exceeding the allowable stresses listed in Table 2109.2.3.1. Allowable stresses not specified in Table 2109.2.3.1 shall comply with the requirements of ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5.

2109.7.3 Floor and roof anchorage. Floor and roof diaphragms providing lateral support to masonry shall comply with the live loads in Section 1607.3 and shall be connected to the masonry in accordance with Sections 2109.7.3.1 through 2109.7.3.3. Roof loading shall be determined in accordance with Chapter 16 and, when net uplift occurs, uplift shall be resisted entirely by an anchorage system designed in accordance with the provisions of Sections 2.1 and 2.3, Sections 3.1 and 3.3 or Chapter 4 of ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5.

Reason: The purpose of this code change proposal is editorial and it updates references for the design and construction of masonry from the 2005 editions of ACI 530-05/ASCE 5-05/TMS 402-05 (Building Code Requirements for Masonry Structures) to TMS 402-08/ACI 530-08/ASCE 5-08 and ACI 530.1-05/ASCE 6-05/TMS 602-05 (Specification for Masonry Structures) to TMS 602-08/ACI 530.1-08/ASCE 6-08. This code change proposal is one of several to harmonize the design and construction requirements for masonry within the IBC with those in the reference standard. A complete list of revisions incorporated into the reference standards is available for download at www.masonrystandards.org.

With the publication of the 2008 edition of the Building Code Requirements for Masonry Structures and Specification for Masonry Structures, The Masonry Society (TMS) has become the lead sponsoring organization of the Masonry Standards Joint Committee (MSJC), which is charged with reviewing and maintaining the provisions in the referenced standards. As such, the official designation of these standards has changed from ACI 530/ASCE 5/TMS 402 to TMS 402/ACI 530/ASCE 5 and from ACI 530.1/ASCE 6/TMS 602 to TMS 602/ACI 530.1/ASCE 6 as reflected in the above proposed modifications.

Numerous sections references are proposed based on changes in the referenced standards. No intent is made to change the technical content of the IBC by these revisions. The deletion of requirements in 2106.1.1 reflect that consistent requirements have been added into the 2008 TMS 402/ACI 530/ASCE 5.

Cost Impact: The code change proposal will not increase the cost of construction.

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S175–07/08

2101.2.2 through 2101.3, 2102, 2103.8, Table 2103.8(1), Table 2103.8(2), 2103.11, 2103.11.1 through 2103.12, 2103.13, 2103.13.1 through 2103.13.8, 2104.1 through 2104.1.2, 2104.1.2.1 through 2104.1.2.7, 2104.1.5, 2104.1.7 through 2104.1.8, 2104.2, 2104.3, 2104.3.1 through 2104.3.3.5, 2104.4, 2104.4.1 through 2104.5, 2105.2.2.1.1, Table 2105.2.2.1.1, 2105.2.2.1.2, 2105.2.2.1.3, 2106.1, 2106.1.1 through 2106.6, 2107.1, 2107.2, 2107.4, 2107.5, 2107.6, 2107.7, 2107.8, 2108.1 through 2108.3, 2310.8.4, 2109(New) through 2110 (New)

Proponent: Jason Thompson, National Concrete Masonry Association, representing Masonry Alliance for Codes and Standards

1. Revise as follows:

2101.2.2 (Supp) Strength design. Masonry designed by the strength design method shall comply with the provisions of Sections 2106 and 2108, except that autoclaved aerated concrete (AAC) masonry shall comply with the provisions of Section 2106, Section 1613.6.3 and Chapter 1 and Appendix A of TMS 402/ACI 530/ASCE 5.

2101.2.3 Prestressed masonry. Prestressed masonry shall be designed in accordance with Chapters 1 and 4 of ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5 and Section 2106. Special inspection during construction shall be provided as set forth in Section 1704.5.

2101.2.4 Empirical design. Masonry designed by the empirical design method shall comply with the provisions of Sections 2106 and 2109 or Chapter 5 of ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5.

2101.2.5 Glass unit masonry. Glass unit masonry shall comply with the provisions of Section 2110 or Chapter 7 of ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5.

2101.2.6 Masonry veneer. Masonry veneer shall comply with the provisions of Chapter 14 or Chapter 6 of ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5.

2101.3 Construction documents. The construction documents shall show all of the items required by this code including the following:

1. Specified size, grade, type and location of reinforcement, anchors and wall ties.
2. Reinforcing bars to be welded and welding procedure.
3. Size and location of structural elements.
4. Provisions for dimensional changes resulting from elastic deformation, creep, shrinkage, temperature and moisture.
5. Loads used in the design of masonry.
6. Specified compressive strength of masonry at stated ages or stages of construction for which masonry is designed, except where specifically exempted by this code.
7. Details of anchorage of masonry to structural members, frames, and other construction, including the type, size, and location of connectors.
8. Size and location of conduits, pipes, and sleeves.
9. The minimum level of testing and inspection as defined in Chapter 17, or an itemized testing and inspection program that meets or exceeds the requirements of Chapter 17.

2. Delete without substitution:

SECTION 2102
DEFINITIONS AND NOTATIONS

~~**BOND REINFORCING.** The adhesion between steel reinforcement and mortar or grout.~~

~~**BUTTRESS.** A projecting part of a masonry wall built integrally therewith to provide lateral stability.~~

~~**COLUMN, MASONRY.** An isolated vertical member whose horizontal dimension measured at right angles to its thickness does not exceed three times its thickness and whose height is at least four times its thickness.~~

~~**COMPOSITE ACTION.** Transfer of stress between components of a member designed so that in resisting loads, the combined components act together as a single member.~~

~~**COMPOSITE MASONRY.** Multiwythe masonry members acting with composite action.~~

~~**DIAPHRAGM.** A roof or floor system designed to transmit lateral forces to shear walls or other lateral-load-resisting elements.~~

~~**EFFECTIVE HEIGHT.** For braced members, the effective height is the clear height between lateral supports and is used for calculating the slenderness ratio. The effective height for unbraced members is calculated in accordance with engineering mechanics.~~

~~**HEADER (Bonder).** A masonry unit that connects two or more adjacent wythes of masonry.~~

~~**MEAN DAILY TEMPERATURE.** The average daily temperature of temperature extremes predicted by a local weather bureau for the next 24 hours.~~

~~**PLASTIC HINGE.** The zone in a structural member in which the yield moment is anticipated to be exceeded under loading combinations that include earthquakes.~~

3. Revise as follows:

NOTATIONS

~~A_n = Net cross-sectional area of masonry, square inches (mm^2).~~
~~b = Effective width of rectangular member or width of flange for T and I sections, inches (mm).~~
~~f_y = Specified yield stress of the reinforcement or the anchor bolt, psi (MPa).~~
~~L_w = Length of wall, inches (mm).~~
~~l_{de} = Embedment length of reinforcement, inches (mm).~~
~~P_w = Weight of wall tributary to section under consideration, pounds (N).~~
~~t = Specified wall thickness dimension or the least lateral dimension of a column, inches (mm).~~
~~V_n = Nominal shear strength, pounds (N).~~
~~V_u = Required shear strength due to factored loads, pounds (N).~~
~~W = Wind load, or related internal moments in forces.~~
~~γ = Reinforcement size factor.~~
~~ρ_n = Ratio of distributed shear reinforcement on plane perpendicular to plane of A_{mv}.~~
~~ρ_{max} = Maximum reinforcement ratio.~~
~~ϕ = Strength reduction factor.~~

<u>P = The applied load at failure, pounds (N).</u>
<u>S_t = Thickness of the test specimen measured parallel to the direction of load, inches (mm).</u>
<u>S_w = Width of the test specimen measured parallel to the loading cylinder, inches (mm).</u>

2103.8 Mortar. Mortar for use in masonry construction shall conform to ASTM C 270 ~~and shall conform to the proportion specifications of Table 2103.8(1) or the property specifications of Table 2103.8(2)~~. Type S or N mortar <u>conforming to ASTM C 270</u> shall be used for glass unit masonry. ~~The amount of water used in mortar for glass unit masonry shall be adjusted to account for the lack of absorption. Retempering of mortar for glass unit masonry shall not be permitted after initial set. Unused mortar shall be discarded within 2½ hours after initial mixing, except that unused mortar for glass unit masonry shall be discarded within 1½ hours after initial mixing.~~

4. Delete without substitution:

~~TABLE 2103.8(1)~~
~~MORTAR PROPORTIONS~~

~~MORTAR~~	~~TYPE~~	~~PROPORTIONS BY VOLUME (cementitious materials)~~								~~AGGREATE MEASURED IN A DAMP, LOOSE CONDITION~~
		~~Portland cement[a] or blended cement[b]~~	~~Mortar cement[d]~~			~~Masonry cement[c]~~			~~HYDRATED LIME[e] OR LIME PUTTY~~	
			~~M~~	~~S~~	~~N~~	~~M~~	~~S~~	~~N~~		
~~Cement-Lime~~	~~M~~	~~1~~	—	—	—	—	—	—	~~1/4~~	~~Not less than 2 1/4 and not more than 3 times the sum of the separate volumes of cementitious materials~~
	~~S~~	~~1~~	—	—	—	—	—	—	~~over 1/4 to 1/2~~	
	~~N~~	~~1~~	—	—	—	—	—	—	~~over 1/2 to 1 1/4~~	
	~~O~~	~~1~~	—	—	—	—	—	—	~~over 1 1/4 to 2 1/2~~	
~~Mortar Cement~~	~~M~~	~~1~~	—	—	~~1~~	—	—	—		
	~~M~~	—	~~1~~	—	—	—	—	—		
	~~S~~	~~1/2~~	—	—	~~1~~	—	—	—		
	~~S~~	—	—	~~1~~	—	—	—	—		
	~~N~~	—	—	—	~~1~~	—	—	—		
	~~O~~	—	—	—	~~1~~	—	—	—		
~~Masonry Cement~~	~~M~~	~~1~~	—	—	—	—	—	~~1~~		
	~~M~~	—	—	—	—	~~1~~	—	—		
	~~S~~	~~1/2~~	—	—	—	—	—	~~1~~		
	~~S~~	—	—	—	—	—	~~1~~	—		
	~~N~~	—	—	—	—	—	—	~~1~~		
	~~O~~	—	—	—	—	—	—	~~1~~		

~~a. Portland cement conforming to the requirements of ASTM C 150.~~
~~b. Blended cement conforming to the requirements of ASTM C 595.~~
~~c. Masonry cement conforming to the requirements of ASTM C 91.~~
~~d. Mortar cement conforming to the requirements of ASTM C 1329.~~
~~e. Hydrated lime conforming to the requirements of ASTM C 207.~~

~~TABLE 2103.8(2)~~
~~MORTAR PROPERTIES~~[a]

~~MORTAR~~	~~TYPE~~	~~AVERAGE COMPRESSIVE~~[b] ~~STRENGTH AT 28 DAYS minimum (psi)~~	~~WATER RETENTION minimum (%)~~	~~AIR CONTENT maximum (%)~~
~~Cement-Lime~~	~~M~~	~~2,500~~	~~75~~	~~12~~
	~~S~~	~~1,800~~	~~75~~	~~12~~
	~~N~~	~~750~~	~~75~~	~~14~~[c]
	~~O~~	~~350~~	~~75~~	~~14~~[c]
~~Mortar Cement~~	~~M~~	~~2,500~~	~~75~~	~~12~~
	~~S~~	~~1,800~~	~~75~~	~~12~~
	~~N~~	~~750~~	~~75~~	~~14~~[c]
	~~O~~	~~350~~	~~75~~	~~14~~[c]
~~Masonry Cement~~	~~M~~	~~2,500~~	~~75~~	~~18~~
	~~S~~	~~1,800~~	~~75~~	~~18~~
	~~N~~	~~750~~	~~75~~	~~20~~[d]
	~~O~~	~~350~~	~~75~~	~~20~~[d]

~~For SI: 1 inch = 25.4 mm, 1 pound per square inch = 6.895kPa.~~

a. ~~This aggregate ratio (measured in damp, loose condition) shall not be less than~~ $2^{1}/_{4}$ ~~and not more than 3 times the sum of the separate volumes of cementitious materials.~~
b. ~~Average of three 2-inch cubes of laboratory-prepared mortar, in accordance with ASTM C 270.~~
c. ~~When structural reinforcement is incorporated in cement-lime or mortar cement mortars, the maximum air content shall not exceed 12 percent.~~
d. ~~When structural reinforcement is incorporated in masonry cement mortar, the maximum air content shall not exceed 18 percent.~~

5. Revise as follows:

2103.11 Mortar for AAC masonry. Thin-bed mortar for AAC masonry shall comply with Article 2.1 C.1 of TMS 602/ACI 530.1/ASCE 6~~Section 2103.11.1. Mortar for leveling courses of AAC masonry shall comply with Section 2103.11.2~~. Mortar used for the leveling courses of AAC masonry shall comply with Article 2.1 C.2 of TMS 602/ACI 530.1/ASCE 6.

6. Delete without substitution:

~~**2103.11.1 Thin-bed mortar for AAC masonry.** Thin-bed mortar for AAC masonry shall be specifically manufactured for use with AAC masonry. Testing to verify mortar properties shall be conducted by the thin-bed mortar manufacturer and confirmed by an independent testing agency:~~

1. ~~The compressive strength of thin-bed mortar, as determined by ASTM C 109, shall meet or exceed the strength of the AAC masonry units.~~
2. ~~The shear strength of thin-bed mortar shall meet or exceed the shear strength of the AAC masonry units for wall assemblages tested in accordance with ASTM E 519.~~
3. ~~The flexural tensile strength of thin-bed mortar shall not be less than the modulus of rupture of the masonry units. Flexural strength shall be determined by testing in accordance with ASTM E 72 (transverse load test), ASTM E 518 Method A (flexural bond strength test) or ASTM C 1072 (flexural bond strength test).~~
   - ~~3.1. For conducting flexural strength tests in accordance with ASTM E 518, at least five test specimens shall be constructed as stack-bonded prisms at least 32 inches (810 mm) high. The type of mortar specified by the AAC unit manufacturer shall be used.~~
   - ~~3.2. For flexural strength tests in accordance with ASTM C 1072, test specimens shall be constructed as stack-bonded prisms comprised with at least three bed joints. A total of at least five joints shall be tested using the type of mortar specified by the AAC unit manufacturer.~~
4. ~~The splitting tensile strength of AAC masonry assemblages composed of two AAC masonry units bonded with one thin-bed mortar joint shall be determined in accordance with ASTM C 1006 and shall equal or exceed~~ $\cancel{2.4\sqrt{f'_{AAC}}}$~~.~~

7. Revise as follows:

2103.12 Grout. Grout shall comply with Article 2.1 C.1 of TMS 602/ACI 530.1/ASCE 6. ~~conform to Table 2103.12 or to ASTM C 476. When grout conforms to ASTM C 476, the grout shall be specified by proportion requirements or property requirements.~~

~~8~~**. Delete without substitution:**

~~**2103.12 Grout.** Grout shall conform to Table 2103.12 or to ASTMC476. When grout conforms toASTMC476, the grout shall be specified by proportion requirements or property requirements.~~

~~**TABLE 2103.12**~~
~~**GROUT PROPORTIONS BY VOLUME FOR MASONRY CONSTRUCTION**~~

~~**TYPE**~~	~~**PARTS BY VOLUME OF PORTLAND CEMENT OR BLENDED CEMENT**~~	~~**PARTS BY VOLUME OF HYDRATED LIME OR LIME PUTTY**~~	~~**AGGREGATE, MEASURED IN A DAMP, LOOSE CONDITION**~~	
			~~Fine~~	~~Coarse~~
~~Fine Grout~~	~~1~~	~~0-1/10~~	~~2 1/4–3 times the sum of the volumes of the cementitious materials~~	—
~~Coarse Grout~~	~~1~~	~~0-1/10~~	~~2 1/4–3 times the sum of the volumes of the cementitious materials~~	~~1–2 times the sum of the volumes of the cementitious materials~~

9. Revise as follows:

2103.13 Metal reinforcement and accessories. Metal reinforcement and accessories shall conform to ~~Sections 2103.13.1 through 2103.13.8~~ <u>Article 2.4 of TMS 602/ACI 530.1/ASCE 6 Where unidentified reinforcement is approved for use, not less than three tension and three bending tests shall be made on representative specimens of the reinforcement from each shipment and grade of reinforcing steel proposed for use in the work</u>.

10. Delete without substitutions:

~~**2103.13.1 Deformed reinforcing bars.** Deformed reinforcing bars shall conform to one of the following standards: ASTM A 615 for deformed and plain billet-steel bars for concrete reinforcement; ASTM A 706 for low-alloy steel deformed bars for concrete reinforcement; ASTM A 767 for zinc-coated reinforcing steel bars; ASTM A 775 for epoxy-coated reinforcing steel bars; and ASTM A 996 for rail and axle steel-deformed bars for concrete reinforcement.~~

~~**2103.13.2 Joint reinforcement.** Joint reinforcement shall comply with ASTM A 951. The maximum spacing of cross wires in ladder-type joint reinforcement and point of connection of cross wires to longitudinal wires of truss-type reinforcement shall be 16 inches (400 mm).~~

~~**2103.13.3 Deformed reinforcing wire.** Deformed reinforcing wire shall conform to ASTM A 496.~~

~~**2103.13.4 Wire fabric.** Wire fabric shall conform to ASTM A 185 for plain steel-welded wire fabric for concrete reinforcement or ASTM A 497 for welded deformed steel wire fabric for concrete reinforcement.~~

~~**2103.13.5 Anchors, ties and accessories.** Anchors, ties and accessories shall conform to the following standards: ASTM A 36 for structural steel; ASTM A 82 for plain steel wire for concrete reinforcement; ASTM A 185 for plain steel-welded wire fabric for concrete reinforcement; ASTM A 240 for chromium and chromium-nickel stainless steel plate, sheet and strip; ASTM A 307 Grade A for anchor bolts; ASTM A 480 for flat rolled stainless and heat-resisting steel plate, sheet and strip; and ASTM A 1008 for cold-rolled carbon steel sheet.~~

~~**2103.13.6 Prestressing tendons.** Prestressing tendons shall conform to one of the following standards:~~

1. ~~Wire .... ASTMA421~~
2. ~~Low-relaxation wire .... ASTMA421~~
3. ~~Strand... ASTMA416~~
4. ~~Low-relaxation strand.................. ASTMA416~~
5. ~~Bar.. ASTMA722~~

~~**Exceptions:**~~

1. ~~Wire, strands and bars not specifically listed in ASTM A 421, ASTM A 416 or ASTM A 722 are permitted, provided they conform to the minimum requirements in ASTM A 421, ASTM A 416 or ASTM A 722 and are approved by the architect/engineer.~~

~~2. Bars and wires of less than 150 kips per square inch (ksi) (1034 MPa) tensile strength and conforming to ASTM A 82, ASTM A 510, ASTM A 615, ASTM A 996 or ASTM A 706 are permitted to be used as prestressed tendons, provided that:~~
   ~~2.1. The stress relaxation properties have been assessed by tests according to ASTM E 328 for the maximum permissible stress in the tendon.~~
   ~~2.2. Other nonstress-related requirements of ACI 530/ASCE 5/TMS 402, Chapter 4, addressing prestressing tendons are met.~~

~~**2103.13.7 Corrosion protection.** Corrosion protection for prestressing tendons shall comply with the requirements of ACI 530.1/ASCE 6/TMS 602, Article 2.4G. Corrosion protection for prestressing anchorages, couplers and end blocks shall comply with the requirements of ACI 530.1/ASCE 6/TMS 602, Article 2.4H. Corrosion protection for carbon steel accessories used in exterior wall construction or interior walls exposed to a mean relative humidity exceeding 75 percent shall comply with either Section 2103.13.7.2 or 2103.13.7.3. Corrosion protection for carbon steel accessories used in interior walls exposed to a mean relative humidity equal to or less than 75 percent shall comply with either Section 2103.13.7.1, 2103.13.7.2 or 2103.13.7.3.~~

~~**2103.13.7.1 Mill galvanized.** Mill galvanized coatings shall be applied as follows:~~

~~1. For joint reinforcement, wall ties, anchors and inserts, a minimum coating of 0.1 ounce per square foot (31g/m^2) complying with the requirements of ASTM A 641 shall be applied.~~
~~2. For sheet metal ties and sheet metal anchors, a minimum coating complying with Coating Designation G-60 according to the requirements of ASTM A 653 shall be applied.~~
~~3. For anchor bolts, steel plates or bars not exposed to the earth, weather or a mean relative humidity exceeding 75 percent, a coating is not required.~~

~~**2103.13.7.2 Hot-dipped galvanized.** Hot-dipped galvanized coatings shall be applied after fabrication as follows:~~

~~1. For joint reinforcement, wall ties, anchors and inserts, a minimum coating of 1.5 ounces per square foot (458 g/m^2) complying with the requirements of ASTM A 153, Class B shall be applied.~~
~~2. For sheet metal ties and anchors, the requirements of ASTM A 153, Class B shall be met.~~
~~3. For steel plates and bars, the requirements of either ASTM A 123 or ASTM A 153, Class B shall be met.~~

~~**2103.13.7.3 Epoxy coatings.** Carbon steel accessories shall be epoxy coated as follows:~~

~~1. For joint reinforcement, the requirements of ASTM A 884, Class A, Type 1 having a minimum thickness of 7 mils (175 *μm*) shall be met.~~
~~2. For wire ties and anchors, the requirements of ASTM A 899, Class C having a minimum thickness of 20 mils (508 *μm*) shall be met.~~
~~3. For sheet metal ties and anchors, a minimum thickness of 20 mils (508 μ*m*) per surface shall be provided or a minimum thickness in accordance with the manufacturer's specification shall be provided.~~

~~**2103.13.8 Tests.** Where unidentified reinforcement is approved for use, not less than three tension and three bending tests shall be made on representative specimens of the reinforcement from each shipment and grade of reinforcing steel proposed for use in the work.~~

11. Revise as follows:

2104.1 Masonry construction. Masonry construction shall comply with the requirements of Sections 2104.1.1 through 2104.5 and with ~~ACI 530.1/ASCE 6/TMS 602~~ TMS 602/ACI 530.1/ASCE 6.

2104.1.1 Tolerances. Masonry, except masonry veneer, shall be constructed within the tolerances specified in ~~ACI 530.1/ASCE 6/TMS 602~~ TMS 602/ACI 530.1/ASCE 6.

2104.1.2 Placing mortar and units. Placement of mortar, grout, and clay, concrete, glass, and AAC masonry ~~and concrete~~ units shall comply with ~~Sections 2104.1.2.1, 2104.1.2.2, 2104.1.2.3 and 2104.1.2.5. Placement of mortar and glass unit masonry shall comply with Sections 2104.1.2.4 and 2104.1.2.5. Placement of thin-bed mortar and AAC masonry shall comply with Section 2104.1.2.6~~ TMS 602/ACI 530.1/ASCE 6.

12. Delete without substitution:

~~**2104.1.2.1 Bed and head joints.** Unless otherwise required or indicated on the construction documents, head and bed joints shall be $^3/_8$ inch (9.5 mm) thick, except that the thickness of the bed joint of the starting course placed over foundations shall not be less than ¼ inch (6.4 mm) and not more than ¼ inch (19.1 mm).~~

~~**2104.1.2.1.1 Open-end units.** Open-end units with beveled ends shall be fully grouted. Head joints of open-end units with beveled ends need not be mortared. The beveled ends shall form a grout key that permits grouts within 5/8 inch (15.9 mm) of the face of the unit. The units shall be tightly butted to prevent leakage of the grout.~~

~~**2104.1.2.2 Hollow units.** Hollow units shall be placed such that face shells of bed joints are fully mortared. Webs shall be fully mortared in all courses of piers, columns, pilasters, in the starting course on foundations where adjacent cells or cavities are to be grouted, and where otherwise required. Head joints shall be mortared a minimum distance from each face equal to the face shell thickness of the unit.~~

~~**2104.1.2.3 Solid units.** Unless otherwise required or indicated on the construction documents, solid units shall be placed in fully mortared bed and head joints. The ends of the units shall be completely buttered. Head joints shall not be filled by slushing with mortar. Head joints shall be constructed by shoving mortar tight against the adjoining unit. Bed joints shall not be furrowed deep enough to produce voids.~~

~~**2104.1.2.4 Glass unit masonry.** Glass units shall be placed so head and bed joints are filled solidly. Mortar shall not be furrowed. Unless otherwise required, head and bed joints of glass unit masonry shall be 1/4 inch (6.4 mm) thick, except that vertical joint thickness of radial panels shall not be less than 1/8 inch (3.2 mm). The bed joint thickness tolerance shall be minus 1/16 inch (1.6 mm) and plus 1/8 inch (3.2 mm). The head joint thickness tolerance shall be plus or minus 1/8 inch (3.2 mm).~~

~~**2104.1.2.5 Placement in mortar.** Units shall be placed while the mortar is soft and plastic. Any unit disturbed to the extent that the initial bond is broken after initial positioning shall be removed and relaid in fresh mortar.~~

~~**2104.1.2.6 Thin-bed mortar and AAC masonry units.** AAC masonry construction shall begin with a leveling course of masonry meeting the requirements of Section 2104.1.2. Subsequent courses of AAC masonry units shall be laid with thin-bed mortar using a special notched trowel manufactured for use with thin-bed mortar to spread the mortar so that it completely fills the bed joints. Unless otherwise specified, the head joints shall be similarly filled. Joints in AAC masonry shall be approximately 1/16 inch (1.5 mm) and shall be formed by striking on the ends and tops of AAC masonry units with a rubber mallet. Minor adjustments in unit position shall be made while the mortar is still soft and plastic by tapping it into the proper position. Minor sanding of the exposed faces of AAC masonry shall be permitted to provide a smooth and plumb surface.~~

~~**2104.1.2.7 Grouted masonry.** Between grout pours, a horizontal construction joint shall be formed by stopping all wythes at the same elevation and with the grout stopping a minimum of 1 1/2 inches (38 mm) below a mortar joint, except at the top of the wall. Where bond beams occur, the grout pour shall be stopped a minimum of ½ inch (12.7 mm) below the top of the masonry.~~

13. Revise as follows:

2104.1.3 Installation of wall ties. ~~The ends of wall ties shall be embedded in mortar joints. Wall tie ends shall engage outer face shells of hollow units by at least ½ inch 12.7 mm). Wire wall ties shall be embedded at least 1½ inches (38 mm) into the mortar bed of solid masonry units or solid-grouted hollow units. Wall ties shall not be bent after being embedded in grout or mortar~~ Wall ties shall be installed in accordance with TMS 602/ACI 530.1/ASCE 6.

2104.1.5 Lintels. The design for lintels shall be in accordance with the masonry design provisions of either Section 2107 or 2108. ~~Minimum length of end support shall be 4 inches (102 mm).~~

14. Delete without substitution:

~~**2104.1.7 Masonry protection.** The top of unfinished masonry work shall be covered to protect the masonry from the weather.~~

~~**2104.1.8 Weep holes.** Weep holes provided in the outside wythe of masonry walls shall be at a maximum spacing of 33 inches (838 mm) on center (o.c.).Weep holes shall not be less than 3/16 inch (4.8 mm) in diameter.~~

15. Delete and substitute as follows:

~~**2104.2 Corbeled masonry.** Except for corbels designed per Section 2107 or 2108, the following shall apply:~~

1. ~~Corbels shall be constructed of solid masonry units.~~
2. ~~The maximum corbeled projection beyond the face of the wall shall not exceed:~~

2.1. ~~One-half of the wall thickness for multiwythe walls bonded by mortar or grout and wall ties or masonry headers or~~
2.2. ~~One-half the wythe thickness for single wythe walls, masonry bonded hollow walls, multiwythe walls with open collar joints and veneer walls.~~

3. ~~The maximum projection of one unit shall not exceed:~~
   3.1. ~~One-half the nominal unit height of the unit or~~
   3.2. ~~One-third the nominal thickness of the unit or wythe.~~
4. ~~The back surface of the corbelled section shall remain within 1 inch (25 mm) of plane.~~

<u>**2104.2 Corbeled masonry.** Corbeled masonry shall comply with the requirements of Section 1.12 of TMS 402/ACI 530/ASCE 5.</u>

16. Revise as follows:

2104.3 Cold weather construction. The cold weather construction provisions of ~~ACI 530.1/ASCE 6/TMS 602~~ <u>TMS 602/ACI 530.1/ASCE 6</u>, Article 1.8 C, ~~or the following procedures~~ shall be implemented when ~~either~~ the ambient temperature falls below 40°F (4°C) ~~or the temperature of masonry units is below 40°F (4°C)~~.

17. Delete without substitution:

~~**2104.3.1 Preparation.**~~

1. ~~Temperatures of masonry units shall not be less than 20°F (-7°C) when laid in the masonry. Masonry units containing frozen moisture, visible ice or snow on their surface shall not be laid.~~
2. ~~Visible ice and snow shall be removed from the top surface of existing foundations and masonry to receive new construction. These surfaces shall be heated to above freezing, using methods that do not result in damage.~~

~~**2104.3.2 Construction.** The following requirements shall apply to work in progress and shall be based on ambient temperature.~~

~~**2104.3.2.1Construction requirements for temperatures between 40°F (4°C) and 32°F (0°C).** The following construction requirements shall be met when the ambient temperature is between 40°F (4°C) and 32°F (0°C):~~

1. ~~Glass unit masonry shall not be laid.~~
2. ~~Water and aggregates used in mortar and grout shall not be heated above 140°F (60°C).~~
3. ~~Mortar sand or mixing water shall be heated to produce mortar temperatures between 40°F (4°C) and 120°F (49°C) at the time of mixing. When water and aggregates for grout are below 32°F(0°C), they shall be heated.~~

~~**2104.3.2.2 Construction requirements for temperatures between 32°F (0°C) and 25°F (-4°C).** The requirements of Section 2104.3.2.1 and the following construction requirements shall be met when the ambient temperature is between 32°F (0°C) and 25°F (-4°C):~~

1. ~~The mortar temperature shall be maintained above freezing until used in masonry.~~
2. ~~Aggregates and mixing water for grout shall be heated to produce grout temperature between 70°F (21°C) and 120°F (49°C) at the time of mixing. Grout temperature shall be maintained above 70°F (21°C) at the time of grout placement.~~
3. ~~Heat AAC masonry units to a minimum temperature of 40°F (4°C) before installing thin-bed mortar.~~

~~**2104.3.2.3 Construction requirements for temperatures between 25°F (-4°C) and 20°F (-7°C).** The requirements of Sections 2104.3.2.1 and 2104.3.2.2 and the following construction requirements shall be met when the ambient temperature is between 25°F (-4°C) and 20°F (-7°C):~~

1. ~~Masonry surfaces under construction shall be heated to 40°F (4°C).~~
2. ~~Wind breaks or enclosures shall be provided when the wind velocity exceeds 15 miles per hour (mph) (24 km/h).~~
3. ~~Prior to grouting, masonry shall be heated to a minimum of 40°F (4°C).~~

~~**2104.3.2.4 Construction requirements for temperatures below 20°F (-7°C).** The requirements of Sections 2104.3.2.1, 2104.3.2.2 and 2104.3.2.3 and the following construction requirement shall be met when the ambient temperature is below 20°F (-7°C): Enclosures and auxiliary heat shall be provided to maintain air temperature within the enclosure to above 32°F (0°C).~~

~~**2104.3.3 Protection.** The requirements of this section and Sections 2104.3.3.1 through 2104.3.3.5 apply after the masonry is placed and shall be based on anticipated minimum daily temperature for grouted masonry and anticipated mean daily temperature for ungrouted masonry.~~

~~**2104.3.3.1 Glass unit masonry.** The temperature of glass unit masonry shall be maintained above 40°F (4°C) for 48 hours after construction.~~

~~**2104.3.3.2 AAC masonry.** The temperature of AAC masonry shall be maintained above 32°F (0°C) for the first 4 hours after thin-bed mortar application.~~

~~**2104.3.3.3 Protection requirements for temperatures between 40°F (4°C) and 25°F (-4°C).** When the temperature is between 40°F (4°C) and 25°F (-4°C), newly constructed masonry shall be covered with a weather-resistive membrane for 24 hours after being completed.~~

~~**2104.3.3.4 Protection requirements for temperatures between 25°F (-4°C) and 20°F (-7°C).** When the temperature is between 25°F (-4°C) and 20°F (-7°C), newly constructed masonry shall be completely covered with weather-resistive insulating blankets, or equal protection, for 24 hours after being completed. The time period shall be extended to 48 hours for grouted masonry, unless the only cement in the grout is Type III portland cement.~~

~~**2104.3.3.5 Protection requirements for temperatures below20°F (-7°C).** When the temperature is below20°F (-7°C), newly constructed masonry shall be maintained at a temperature above 32°F (0°C) for at least 24 hours after being completed by using heated enclosures, electric heating blankets, infrared lamps or other acceptable methods. The time period shall be extended to 48 hours for grouted masonry, unless the only cement in the grout is Type III portland cement.~~

18. Revise as follows:

2104.4 Hot weather construction. The hot weather construction provisions of ~~ACI 530.1/ASCE 6/TMS 602~~ <u>TMS 602/ACI 530.1/ASCE 6</u>, Article 1.8 D, ~~or the following procedures~~ shall be implemented when the ~~temperature or the temperature and wind-velocity limits of this section are exceeded~~ <u>ambient air temperature exceeds 100°F (37.8°C), or exceeds 90°F (32.2°C) with a wind velocity greater than 8 mph (12.9 km/hr)</u>.

19. Delete without substitution:

~~**2104.4.1 Preparation.** The following requirements shall be met prior to conducting masonry work.~~

~~**2104.4.1.1 Temperature.** When the ambient temperature exceeds 100°F (38°C), or exceeds 90°F (32°C) with a wind velocity greater than 8 mph (3.5 m/s):~~

1. ~~Necessary conditions and equipment shall be provided to produce mortar having a temperature below 120°F (49°C).~~
2. ~~Sand piles shall be maintained in a damp, loose condition.~~

~~**2104.4.1.2 Special conditions.** When the ambient temperature exceeds 115°F (46°C), or 105°F (40°C) with a wind velocity greater than 8 mph (3.5 m/s), the requirements of Section 2104.4.1.1 shall be implemented, and materials and mixing equipment shall be shaded from direct sunlight.~~

~~**2104.4.2 Construction.** The following requirements shall be met while masonry work is in progress.~~

~~**2104.4.2.1 Temperature.** When the ambient temperature exceeds 100°F (38°C), or exceeds 90°F (32°C) with a wind velocity greater than 8 mph (3.5 m/s):~~

1. ~~The temperature of mortar and grout shall be maintained below 120°F (49°C).~~
2. ~~Mixers, mortar transport containers and mortar boards shall be flushed with cool water before they come into contact with mortar ingredients or mortar.~~
3. ~~Mortar consistency shall be maintained by retempering with cool water.~~
4. ~~Mortar shall be used within 2 hours of initial mixing.~~
5. ~~Thin-bed mortar shall be spread no more than 4 feet (1219 mm) ahead of AAC masonry units.~~
6. ~~AAC masonry units shall be placed within one minute after spreading thin-bed mortar.~~

~~**2104.4.2.2 Special conditions.** When the ambient temperature exceeds 115°F (46°C), or exceeds 105°F (40°C) with a wind velocity greater than 8 mph (3.5 m/s), the requirements of Section 2104.4.2.1 shall be implemented and cool mixing water shall be used for mortar and grout. The use of ice shall be permitted in the mixing water prior to use. Ice shall not be permitted in the mixing water when added to the other mortar or grout materials.~~

~~**2104.4.3 Protection.** When the mean daily temperature exceeds 100°F (38°C) or exceeds 90°F (32°C) with a wind velocity greater than 8 mph (3.5 m/s), newly constructed masonry shall be fog sprayed until damp at least three times a day until the masonry is three days old.~~

~~**2104.5 Wetting of brick.** Brick (clay or shale) at the time of laying shall require wetting if the unit's initial rate of water absorption exceeds 30 grams per 30 square inches (19 355 mm^2) per minute or 0.035 ounce per square inch (1 g/645mm^2) per minute, as determined by ASTM C 67.~~

20. Revise as follows:

2105.2.2.1.1 Clay masonry. The compressive strength of masonry shall be determined based on the strength of the units and the type of mortar specified using Table 2105.2.2.1.1, provided:

1. Units ~~conform to~~ are sampled and tested to verify conformance with ASTM C 62, ASTM C 216 or ASTM C 652.
2. Thickness of bed joints does not exceed 5/8 inch (15.9 mm).
3. For grouted masonry, the grout meets one of the following requirements:
   3.1. Grout conforms to ~~ASTM C 476~~ Article 2.2 of TMS 602/ACI 530.1/ASCE 6.
   3.2. Minimum grout compressive strength equals or exceeds f_m but not less than 2,000 psi (13.79MPa). The compressive strength of grout shall be determined in accordance with ASTM C 1019.

TABLE 2105.2.2.1.1
COMPRESSIVE STRENGTH OF CLAY MASONRY

NET AREA COMPRESSIVE STRENGTH OF CLAY MASONRY UNITS (psi)		NET AREA COMPRESSIVE STRENGTH OF MASONRY (psi)
Type M or S mortar	Type N mortar	
1,700	2,100	1,000
3,350	4,150	1,500
4,950	6,200	2,000
6,600	8,250	2,500
8,250	10,300	3,000
9,900	___	3,500
~~13,200~~11,500	___	4,000

For SI: 1 pound per square inch = 0.00689MPa.

2105.2.2.1.2 Concrete masonry. The compressive strength of masonry shall be determined based on the strength of the unit and type of mortar specified using Table 2105.2.2.1.2, provided:

1. Units ~~conform to~~ are sampled and tested to verify conformance with ASTM C 55 or ASTM C 90 ~~and are sampled and tested in accordance with ASTM C 140~~.
2. Thickness of bed joints does not exceed 5/8 inch (15.9 mm).
3. For grouted masonry, the grout meets one of the following requirements:
   3.1. Grout conforms to ~~ASTM C 476~~ Article 2.2 of TMS 602/ACI 530.1/ASCE 6.
   3.2. Minimum grout compressive strength equals or exceeds f_m but not less than 2,000 psi (13.79 MPa). The compressive strength of grout shall be determined in accordance with ASTM C 1019.

2105.2.2.1.3 AAC masonry. The compressive strength of AAC masonry shall be based on the strength of the AAC masonry unit only and the following shall be met:

1. Units conform to ASTM C 1386.
2. Thickness of bed joints does not exceed 1/8 inch (3.2 mm).
3. For grouted masonry, the grout meets one of the following requirements:
   3.1. Grout conforms to ~~ASTM C 476~~ Article 2.2 of TMS 602/ACI 530.1/ASCE 6.
   3.2. Minimum grout compressive strength equals or exceeds f'_{AAC} but not less than 2,000 psi (13.79 MPa). The compressive strength of grout shall be determined in accordance with ASTM C 1019.

21. Delete and substitute:

~~**2106.1 Seismic design requirements for masonry.** Masonry structures and components shall comply with the requirements in Section 1.14.2.2 and Section 1.14.3, 1.14.4, 1.14.5, 1.14.6 or 1.14.7 of ACI 530/ASCE 5/TMS 402 depending on the structure's seismic design category as determined in Section 1613. All masonry walls, unless isolated on three edges from in-plane motion of the basic structural systems, shall be considered to be part of the seismic-force-resisting system. In addition, the following requirements shall be met.~~

2106.1 Seismic design requirements for masonry. Masonry structures and components shall comply with the requirements in Section 1.17 of TMS 402/ACI 530/ASCE 5 depending on the structure's seismic design category as determined in Section 1613.

22. Delete without substitution:

~~**2106.1.1 Basic seismic-force-resisting system.** Buildings relying on masonry shear walls as part of the basic seismic-force-resisting system shall comply with Section 1.14.2.2 of ACI 530/ASCE 5/TMS 402 or with Section 2106.1.1.1, 2106.1.1.2 or 2106.1.1.3.~~

~~**2106.1.1.1 Ordinary plain prestressed masonry shear walls.** Ordinary plain prestressed masonry shear walls shall comply with the requirements of Chapter 4 of ACI 530/ASCE 5/TMS 402.~~

~~**2106.1.1.2 Intermediate prestressed masonry shear walls.** Intermediate prestressed masonry shear walls shall comply with the requirements of Section 1.14.2.2.4 of ACI 530/ASCE 5/TMS 402 and shall be designed by Chapter 4, Section 4.4.3, of ACI 530/ASCE 5/TMS 402 for flexural strength and by Section 3.3.4.1.2 of ACI 530/ASCE 5/TMS 402 for shear strength. Sections 1.14.2.2.5, 3.3.3.5 and 3.3.4.3.2(c) of ACI 530/ASCE 5/TMS 402 shall be applicable for reinforcement. Flexural elements subjected to load reversals shall be symmetrically reinforced. The nominal moment strength at any section along a member shall not be less than one-fourth the maximum moment strength. The cross-sectional area of bonded tendons shall be considered to contribute to the minimum reinforcement in Section 1.14.2.2.4 of ACI 530/ASCE 5/TMS 402. Tendons shall be located in cells that are grouted the full height of the wall.~~

~~**2106.1.1.3 Special prestressed masonry shear walls.** Special prestressed masonry shear walls shall comply with the requirements of Section 1.14.2.2.5 of ACI 530/ASCE 5/TMS 402 and shall be designed by Chapter 4, Section 4.4.3, of ACI 530/ASCE 5/TMS 402 for flexural strength and by Section 3.3.4.1.2 of ACI 530/ASCE 5/TMS 402 for shear strength. Sections 1.14.2.2.5(a), 3.3.3.5 and 3.3.4.3.2(c) of ACI 530/ASCE 5/TMS 402 shall be applicable for reinforcement. Flexural elements subjected to load reversals shall be symmetrically reinforced. The nominal moment strength at any section along a member shall not be less than one-fourth the maximum moment strength. The cross-sectional area of bonded tendons shall be considered to contribute to the minimum reinforcement in Section 1.14.2.2.5 of ACI 530/ASCE 5/TMS 402.~~

~~**2106.1.1.3.1 Prestressing tendons.** Prestressing tendons shall consist of bars conforming to ASTM A 722.~~

~~**2106.1.1.3.2 Grouting.** All cells of the masonry wall shall be grouted.~~

~~**2106.2 Anchorage of masonry walls.** Masonry walls shall be anchored to the roof and floors that provide lateral support for the wall in accordance with Section 1604.8.2.~~

~~**2106.3 Seismic Design Category B.** Structures assigned to Seismic Design Category B shall conform to the requirements of Section 1.14.4 of ACI 530/ASCE 5/TMS 402 and to the additional requirements of this section.~~

~~**2106.3.1 Masonry walls not part of the lateral-force-resisting system.** Masonry partition walls, masonry screen walls and other masonry elements that are not designed to resist vertical or lateral loads, other than those induced by their own mass, shall be isolated from the structure so that the vertical and lateral forces are not imparted to these elements. Isolation joints and connectors between these elements and the structure shall be designed to accommodate the design story drift.~~

~~**2106.4 Additional requirements for structures in Seismic Design Category C.** Structures assigned to Seismic Design Category C shall conform to the requirements of Section 2106.3, Section 1.14.5 of ACI 530/ASCE 5/TMS 402 and the additional requirements of this section.~~

~~**2106.4.1 Design of discontinuous members that are part of the lateral-force-resisting system.** Columns and pilasters that are part of the lateral-force-resisting system and that support reactions from discontinuous stiff members~~

~~such as walls shall be provided with transverse reinforcement spaced at no more than one-fourth of the least nominal dimension of the column or pilaster. The minimum transverse reinforcement ratio shall be 0.0015. Beams supporting reactions from discontinuous walls or frames shall be provided with transverse reinforcement spaced at no more than one-half of the nominal depth of the beam. The minimum transverse reinforcement ratio shall be 0.0015.~~

~~**2106.5 Additional requirements for structures in Seismic Design Category D.** Structures assigned to Seismic Design Category D shall conform to the requirements of Section 2106.4, Section 1.14.6 of ACI 530/ASCE 5/TMS 402 and the additional requirements of this section.~~

~~**2106.5.1 Loads for shear walls designed by the allowable stress design method.** When calculating in-plane shear or diagonal tension stresses by the allowable stress design method, shear walls that resist seismic forces shall be designed to resist 1.5 times the seismic forces required by Chapter 16. The 1.5 multiplier need not be applied to the overturning moment.~~

~~**2106.5.2 Shear wall shear strength.** For a shear wall whose nominal shear strength exceeds the shear corresponding to development of its nominal flexural strength, two shear regions exist.~~

~~For all cross sections within a region defined by the base of the shear wall and a plane at a distance~~ $\cancel{L_w}$ ~~above the base of the shear wall, the nominal shear strength shall be determined by Equation 21-1.~~

$\cancel{V_n = A_{n\,n} f_y}$ **~~(Equation 21-1)~~**

~~The required shear strength for this region shall be calculated at a distance~~ $\cancel{L_w/2}$ ~~above the base of the shear wall, but not to exceed one-half story height. For the other region, the nominal shear strength of the shear wall shall be determined from Section 2108.~~

~~**2106.6 Additional requirements for structures in Seismic Design Category E or F.** Structures assigned to Seismic Design Category E or F shall conform to the requirements of Section 2106.5 and Section 1.14.7 of ACI 530/ASCE 5/TMS 402.~~

23. Revise as follows:

2107.1 General. The design of masonry structures using allowable stress design shall comply with Section 2106 and the requirements of Chapters 1 and 2 of ~~ACI 530/ASCE 5/TMS 402~~ <u>TMS 402/ACI 530/ASCE 5</u> except as modified by Sections 2107.2 through ~~2107.8~~ <u>2107.5</u>.

2107.2 ~~ACI 530/ASCE 5/TMS 402~~ <u>TMS 402/ACI 530/ASCE 5</u>, Section 2.1.2, load combinations. Delete Section 2.1.2.1.

24. Delete without substitution:

~~**2107.3 ACI 530/ASCE 5/TMS 402, Section 2.1.3, design strength.** Delete Sections 2.1.3.4 through 2.1.3.4.3.~~

~~**2107.4 ACI 530/ASCE 5/TMS 402, Section 2.1.6, columns.** Add the following text to Section 2.1.6:~~
~~2.1.6.6 Light-frame construction. Masonry columns used only to support light-frame roofs of carports, porches, sheds or similar structures with a maximum area of 450 square feet (41.8 m²) assigned to Seismic Design Category A, B or Care permitted to be designed and constructed as follows:~~

1. ~~Concrete masonry materials shall be in accordance with Section 2103.1 of the *International Building Code*. Clay or shale masonry units shall be in accordance with Section 2103.2 of the *International Building Code*.~~
2. ~~The nominal cross-sectional dimension of columns shall not be less than 8 inches (203 mm).~~
3. ~~Columns shall be reinforced with not less than one No. 4 bar centered in each cell of the column.~~
4. ~~Columns shall be grouted solid.~~
5. ~~Columns shall not exceed 12 feet (3658 mm) in height.~~
6. ~~Roofs shall be anchored to the columns. Such anchorage shall be capable of resisting the design loads specified in Chapter 16 of the *International Building Code*.~~
7. ~~Where such columns are required to resist uplift loads, the columns shall be anchored to their footings with two No. 4 bars extending a minimum of 24 inches (610 mm) into the columns and bent horizontally a minimum of 15 inches (381 mm) in opposite directions into the footings. One of these bars is permitted to be the reinforcing bar specified in Item 3 above. The total weight of a column and its footing shall not be less than 1.5 times the design uplift load.~~

25. Revise as follows:

~~2107.5~~ <u>2107.3</u> ~~ACI 530/ASCE 5/TMS 402~~ <u>TMS 402/ACI 530/ASCE 5</u>, Section 2.1.10.7.1.1, lap splices. Modify Section ~~2.1.10.7.1.1~~ <u>2.1.9.7.1.1</u> as follows:

~~2.1.10.7.1.1~~ <u>2.1.9.7.1.1</u> The minimum length of lap splices for reinforcing bars in tension or compression, l_d, shall be

$l_d = 0.002 d_b f_s$ **(Equation ~~21-2~~ <u>21-1</u>)**
For SI: $l_d = 0.29 d_b f_s$
but not less than 12 inches (305 mm). In no case shall the length of the lapped splice be less than 40 bar diameters.

where:

d_b = Diameter of reinforcement, inches (mm).
f_s = Computed stress in reinforcement due to design loads, psi (MPa).

In regions of moment where the design tensile stresses in the reinforcement are greater than 80 percent of the allowable steel tension stress, F_s, the lap length of splices shall be increased not less than 50 percent of the minimum required length. Other equivalent means of stress transfer to accomplish the same 50 percent increase shall be permitted. Where epoxy coated bars are used, lap length shall be increased by 50 percent.

~~2107.6~~ <u>2107.4</u> ~~ACI 530/ASCE 5/TMS 402~~ <u>TMS 402/ACI 530/ASCE 5</u>, Section ~~2.1.10.7~~ <u>2.1.9.7</u>, splices of reinforcement. Modify Section 2.1.10.7 as follows:

~~2.1.10.7~~ <u>2.1.9.7</u> Splices of reinforcement. Lap splices, welded splices or mechanical splices are permitted in accordance with the provisions of this section. All welding shall conform to AWS D1.4. <u>Welded splices shall be of ASTM A 706 steel reinforcement.</u> Reinforcement larger than No. 9 (M #29) shall be spliced using mechanical connections in accordance with Section 2.1.10.7.3.

~~2107.7~~ <u>2107.5</u> ~~ACI 530/ASCE 5/TMS 402~~ <u>TMS 402/ACI 530/ASCE 5</u>, Section 2.3.6, maximum bar size. Add the following to Chapter 2:

2.3.6 Maximum bar size. The bar diameter shall not exceed one-eighth of the nominal wall thickness and shall not exceed one-quarter of the least dimension of the cell, course or collar joint in which it is placed.

26. Delete without substitution:

~~**2107.8 ACI 530/ASCE 5/TMS 402, Section 2.3.7, maximum reinforcement percentage.** Add the following text to Chapter 2:~~

~~2.3.7 Maximum reinforcement percentage. Special reinforced masonry shear walls having a shear span ratio, *M/Vd*, equal to or greater than 1.0 and having an axial load, *P*, greater than 0.05~~ $\cancel{f'_m A_n}$ ~~that are subjected to in-plane forces shall have a maximum reinforcement ratio,~~ $\cancel{\rho_{max}}$, ~~not greater than that computed as follows:~~

$$\cancel{\rho_{\max} = \frac{n f'_m}{2 f_y \left(n + \frac{f_y}{f'_m} \right)}} \qquad \text{~~(Equation 21-3)~~}$$

~~The maximum reinforcement ratio does not apply in the out-of-plane direction.~~

27. Revise as follows:

2108.1 General. The design of masonry structures using strength design shall comply with Section 2106 and the requirements of Chapters 1 and 3 of ~~ACI 530/ASCE 5/TMS 402~~ <u>TMS 402/ACI 530/ASCE 5</u>, except as modified by Sections 2108.2 through 2108.4.

> **Exception:** AAC masonry shall comply with the requirements of Chapter 1 and Appendix A of ~~ACI 530/ASCE 5/TMS 402~~ <u>TMS 402/ACI 530/ASCE 5</u>.

2108.2 ~~ACI 530/ASCE 5/TMS 402~~ <u>TMS 402/ACI 530/ASCE 5</u>, Section 3.3.3.3 development. Add the following text to Section 3.3.3.3:

The required development length of reinforcement shall be determined by Equation (3-15), but shall not be less than 12 inches (305 mm) and need not be greater than 72 d_b.

2108.3 ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5, Section 3.3.3.4, splices. Modify items (b) and (c) of Section 3.3.3.4 as follows:

3.3.3.4 (b). A welded splice shall have the bars butted and welded to develop at least 125 percent of the yield strength, f_y, of the bar in tension or compression, as required. Welded splices shall be of ASTM A 706 steel reinforcement. Welded splices shall not be permitted in plastic hinge zones of intermediate or special reinforced walls or special moment frames of masonry. 3.3.3.4 (c). Mechanical splices shall be classified as Type 1 or 2 according to Section 21.2.6.1 of ACI 318. Type 1 mechanical splices shall not be used within a plastic hinge zone or within a beam-column joint of intermediate or special reinforced masonry shear walls or special moment frames. Type 2 mechanical splices are permitted in any location within a member.

28. Delete without substitution:

~~**2108.4 ACI 530/ASCE 5/TMS 402, Section 3.3.3.5, maximum areas of flexural tensile reinforcement.** Add the following text to Section 3.3.3.5:~~

~~3.3.3.5.5 For special prestressed masonry shear walls, strain in all prestressing steel shall be computed to be compatible with a strain in the extreme tension reinforcement equal to five times the strain associated with the reinforcement yield stress, f_y. The calculation of the maximum reinforcement shall consider forces in the prestressing steel that correspond to these calculated strains.~~

29. Delete Section 2109 in its entirety and substitute as follows:

SECTION 2109
EMPIRICAL DESIGN OF MASONRY

2109.1 General. Empirically designed masonry shall conform to the requirements of Chapter 5 of TMS 402/ACI 530/ASCE 5, except where otherwise noted in this section.

2109.1.1 Limitations. The use of empirical design of masonry shall be limited as noted in Section 5.1.2 of TMS 402/ACI 530/ASCE 5. In buildings that exceed one or more of the limitations of Section 5.1.2 of TMS 402/ACI 530/ASCE 5, masonry shall be designed in accordance with the engineered design provisions of Section 2101.2.1, 2101.2.2, or 2101.2.3 or the foundation wall provisions of Section 1805.5.

2109.2 Surface-bonded walls. Dry-stacked, surface-bonded concrete masonry walls shall comply with the requirements of Chapter 5 of TMS 402/ACI 530/ASCE 5, except where otherwise noted in this section.
2109.2.1 Strength. Dry-stacked, surface-bonded concrete masonry walls shall be of adequate strength and proportions to support all superimposed loads without exceeding the allowable stresses listed in Table 2109.2.1. Allowable stresses not specified in Table 2109.2.1 shall comply with the requirements of TMs 402/ACI 530/ASCE 5.

TABLE 2109.2.1
ALLOWABLE STRESS GROSS CROSS-SECTIONAL AREA
FOR DRY-STACKED, SURFACE-BONDED CONCRETE MASONRY WALLS

DESCRIPTION	MAXIMUM ALLOWABLE STRESS (psi)
Compression standard block	45
Flexural tension	
Horizontal span	30
Vertical span	18
Shear	10

For SI: 1 pound per square inch = 0.006895MPa.

2109.2.2 Construction. Construction of dry-stacked, surface-bonded masonry walls, including stacking and leveling of units, mixing and application of mortar and curing and protection shall comply with ASTM C 946.

2109.3 Adobe construction. Adobe construction shall comply with this section and shall be subject to the requirements of this code for Type V construction, Chapter 5 of TMS 402/ACI 530/ASCE 5, and this section.

2109.3.1 Unstabilized adobe.

2109.3.1.1 Compressive strength. Adobe units shall have an average compressive strength of 300 psi (2068 kPa) when tested in accordance with ASTM C 67. Five samples shall be tested and no individual unit is permitted to have a compressive strength of less than 250 psi (1724 kPa).

2109.3.1.2 Modulus of rupture. Adobe units shall have an average modulus of rupture of 50 psi (345 kPa) when tested in accordance with the following procedure. Five samples shall be tested and no individual unit shall have a modulus of rupture of less than 35 psi (241 kPa).

2109.3.1.2.1 Support conditions. A cured unit shall be simply supported by 2-inch-diameter (51 mm) cylindrical supports located 2 inches (51 mm) in from each end and extending the full width of the unit.

2109.3.1.2.2 Loading conditions. A 2-inch-diameter (51 mm) cylinder shall be placed at midspan parallel to the supports.

2109.3.1.2.3 Testing procedure. A vertical load shall be applied to the cylinder at the rate of 500 pounds per minute (37 N/s) until failure occurs.

2109.3.1.2.4 Modulus of rupture determination. The modulus of rupture shall be determined by the equation:

$$f_r = 3\,PL_s/2S_w\,(S_t^2) \qquad \textbf{(Equation 21-2)}$$

where, for the purposes of this section only:

S_w = Width of the test specimen measured parallel to the loading cylinder, inches (mm).
f_r = Modulus of rupture, psi (MPa).
L_s = Distance between supports, inches (mm).
S_t = Thickness of the test specimen measured parallel to the direction of load, inches (mm).
P = The applied load at failure, pounds (N).

2109.3.1.3 Moisture content requirements. Adobe units shall have a moisture content not exceeding 4 percent by weight.

2109.3.1.4 Shrinkage cracks. Adobe units shall not contain more than three shrinkage cracks and any single shrinkage crack shall not exceed 3 inches (76 mm) in length or $^1/_8$ inch (3.2 mm) in width.

2109.3.2 Stabilized adobe.

2109.3.2.1 Material requirements. Stabilized adobe shall comply with the material requirements of unstabilized adobe in addition to Sections 2109.3.2.1.1 and 2109.3.2.1.2.

2109.3.2.1.1 Soil requirements. Soil used for stabilized adobe units shall be chemically compatible with the stabilizing material.

2109.3.2.1.2 Absorption requirements. A 4-inch (102 mm) cube, cut from a stabilized adobe unit dried to a constant weight in a ventilated oven at 212°F to 239°F (100°C to 115°C), shall not absorb more than $2^1/_2$ percent moisture by weight when placed upon a constantly water-saturated, porous surface for seven days. A minimum of five specimens shall be tested and each specimen shall be cut from a separate unit.

2109.3.3 Allowable stress. The allowable compressive stress based on gross cross-sectional area of adobe shall not exceed 30 psi (207 kPa).

2109.3.3.1 Bolts. Bolt values shall not exceed those set forth in Table 2109.3.3.1.

TABLE 2109.3.3.1
ALLOWABLE SHEAR ON BOLTS IN ADOBE MASONRY

DIAMETER OF BOLTS (inches)	MINIMUM EMBEDMENT (inches)	SHEAR (pounds)
$^{1}/_{2}$	___	___
$^{5}/_{8}$	12	200
$^{3}/_{4}$	15	300
$^{7}/_{8}$	18	400
1	21	500
$1^{1}/_{8}$	24	600

For SI: 1 inch = 25.4 mm, 1 pound = 4.448 N.

2109.3.4 Construction.

2109.3.4.1 General.

2109.3.4.1.1 Height restrictions. Adobe construction shall be limited to buildings not exceeding one story, except that two-story construction is allowed when designed by a registered design professional.

2109.3.4.1.2 Mortar restrictions. Mortar for stabilized adobe units shall comply with Chapter 21 or adobe soil. Adobe soil used as mortar shall comply with material requirements for stabilized adobe. Mortar for unstabilized adobe shall be portland cement mortar.

2109.3.4.1.3 Mortar joints. Adobe units shall be laid with full head and bed joints and in full running bond.

2109.3.4.1.4 Parapet walls. Parapet walls constructed of adobe units shall be waterproofed.

2109.3.4.2 Wall thickness. The minimum thickness of exterior walls in one-story buildings shall be 10 inches (254 mm). The walls shall be laterally supported at intervals not exceeding 24 feet (7315 mm). The minimum thickness of interior load-bearing walls shall be 8 inches (203 mm). In no case shall the unsupported height of any wall constructed of adobe units exceed 10 times the thickness of such wall.

2109.3.4.3 Foundations.

2109.3.4.3.1 Foundation support. Walls and partitions constructed of adobe units shall be supported by foundations or footings that extend not less than 6 inches (152 mm) above adjacent ground surfaces and are constructed of solid masonry (excluding adobe) or concrete. Footings and foundations shall comply with Chapter 18.

2109.3.4.3.2 Lower course requirements. Stabilized adobe units shall be used in adobe walls for the first 4 inches (102 mm) above the finished first-floor elevation.

2109.3.4.4 Isolated piers or columns. Adobe units shall not be used for isolated piers or columns in a load-bearing capacity. Walls less than 24 inches (610 mm) in length shall be considered isolated piers or columns.

2109.3.4.5 Tie beams. Exterior walls and interior load-bearing walls constructed of adobe units shall have a continuous tie beam at the level of the floor or roof bearing and meeting the following requirements.

2109.3.4.5.1 Concrete tie beams. Concrete tie beams shall be a minimum depth of 6 inches (152 mm) and a minimum width of 10 inches (254 mm). Concrete tie beams shall be continuously reinforced with a minimum of two No. 4 reinforcing bars. The ultimate compressive strength of concrete shall be at least 2,500 psi (17.2 MPa) at 28 days.

2109.3.4.5.2 Wood tie beams. Wood tie beams shall be solid or built up of lumber having a minimum nominal thickness of 1 inch (25 mm), and shall have a minimum depth of 6 inches (152 mm) and a minimum width of 10 inches (254 mm). Joints in wood tie beams shall be spliced a minimum of 6 inches (152 mm). No splices shall be allowed within 12 inches (305 mm) of an opening. Wood used in tie beams shall be approved naturally decay-resistant or pressure-treated wood.

2109.3.4.6 Exterior finish. Exterior walls constructed of unstabilized adobe units shall have their exterior surface covered with a minimum of two coats of portland cement plaster having a minimum thickness of 3/4 inch (19.1 mm) and conforming to ASTM C 926. Lathing shall comply with ASTM C 1063. Fasteners shall be spaced at 16 inches (406 mm) o.c. maximum. Exposed wood surfaces shall be treated with an approved wood preservative or other protective coating prior to lath application.

2109.3.4.7 Lintels. Lintels shall be considered structural members and shall be designed in accordance with the applicable provisions of Chapter 16.

30. Delete Section 2110 in its entirety and substitute as follows:

SECTION 2110
GLASS UNIT MASONRY

2110.1 General. Glass unit masonry construction shall comply with Chapter 7 of TMS 402/ACI 530/ASCE 5 and this section.

2110.1.1 Limitations. Solid or hollow approved glass block shall not be used in fire walls, party walls, fire barriers, fire partitions or smoke barriers, or for load-bearing construction. Such blocks shall be erected with mortar and reinforcement in metal channel-type frames, structural frames, masonry or concrete recesses, embedded panel anchors as provided for both exterior and interior walls or other approved joint materials. Wood strip framing shall not be used in walls required to have a fire-resistance rating by other provisions of this code.

Exceptions:

1. Glass-block assemblies having a fire protection rating of not less than 3/4 hour shall be permitted as opening protectives in accordance with Section 715 in fire barriers, fire partitions, and smoke barriers that have a required fire-resistance rating of 1 hour or less and do not enclose exit stairways, exit ramps, or exit passageways.
2. Glass-block assemblies as permitted in Section 404.5, Exception 2.

Reason: The primary purpose of this code change proposal is to update the existing reference standards for the design and construction of masonry from the 2005 editions to the 2008 editions. The revisions proposed in this code change, while relatively minor in nature, reflect revisions incorporated into the 2008 edition of the Building Code Requirements for Masonry Structures and Specification for Masonry Structures. This code change proposal is one of several to harmonize the design and construction requirements for masonry within the IBC with those in the reference standard. A complete list of revisions incorporated into the referenced standards is available for download at www.masonrystandards.org.

An editorial change to the titles of ACI 530.1 and ASCE/SEI 6 as shown in Chapter 35 is also proposed.

With the publication of the 2008 edition of the Building Code Requirements for Masonry Structures and Specification for Masonry Structures, The Masonry Society (TMS) has become the lead sponsoring organization of the Masonry Standards Joint Committee (MSJC), which is charged with reviewing and maintaining the provisions in the referenced standards. As such, the official designation of these standards has changed from ACI 530/ASCE 5/TMS 402 to TMS 402/ACI 530/ASCE 5 and from ACI 530.1/ASCE 6/TMS 602 to TMS 602/ACI 530.1/ASCE 6 as reflected in the above proposed modifications.

The additional information proposed to be included in Section 2101.3 simply reflect revisions to the reference standards regarding the minimum information to be included on the construction documents. The proposed additions are consistent in intent with existing requirements in Chapters 1 and 16 as well as other material chapters.

The remainder of the changes proposed herein remove provisions transcribed into the IBC from the MSJC standards. While this series of changes may appear substantive, it is instead intended to be editorial as the provisions proposed for removal are already contained in the reference standards. For several years the ICC has been moving toward removing provisions transcribed into the body of the I-Codes, and where applicable, referencing a governing standard. This change proposes just that, with the one exception related to Section 2105. The provisions of this section are retained for the benefit of inspectors and regulators.

Some have voiced a concern in the past that by removing provisions from the IBC it forces them to purchase numerous reference standards. In the case of the masonry design and construction provisions, this statement would be true regardless of whether the transcribed provisions proposed to be removed by this change proposal were accepted or not as only a very small fraction of the MSJC provisions are transcribed. It can be argued that maintaining the transcribed provisions in the current form is a disservice to the safety and welfare of the general public; who may assume or are told that this Chapter contains to complete body of information related to masonry construction, materials, and design. Whether one assumes the role of contractor, designer, building official, inspector, plan checker, or specifier, they still must have the reference document to perform their tasks.

Cost Impact: The code change proposal will not increase the cost of construction.

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S176–07/08
2101.2, 2101.3

Proponent: Jason Thompson, National Concrete Masonry Association, representing Masonry Alliance for Codes and Standards

Revise as follows:

2101.2.2 (Supp) Strength design. Masonry designed by the strength design method shall comply with the provisions of Sections 2106 and 2108, except that AAC masonry shall comply with the provisions of Section 2106, Section 1613.6.3, and Chapter 1 and Appendix A of TMS 402/ACI 530/ASCE 5~~/TMS 402~~.

2101.2.3 Prestressed masonry. Prestressed masonry shall be designed in accordance with Chapters 1 and 4 of TMS 402/ACI 530/ASCE 5~~/TMS 402~~ and Section 2106. Special inspection during construction shall be provided as set forth in Section 1704.5.

2101.2.4 Empirical design. Masonry designed by the empirical design method shall comply with the provisions of Sections 2106 and 2109 or Chapter 5 of TMS 402/ACI 530/ASCE 5~~/TMS 402~~.

2101.2.5 Glass unit masonry. Glass unit masonry shall comply with the provisions of Section 2110 or Chapter 7 of TMS 402/ACI 530/ASCE 5~~/TMS 402~~.

2101.2.6 Masonry veneer. Masonry veneer shall comply with the provisions of Chapter 14 or Chapter 6 of TMS 402/ACI 530/ASCE 5~~/TMS 402~~.

2101.3 Construction documents. The construction documents shall show all of the items required by this code including the following:

1. Specified size, grade, type and location of reinforcement, anchors and wall ties.
2. Reinforcing bars to be welded and welding procedure.
3. Size and location of structural elements.
4. Provisions for dimensional changes resulting from elastic deformation, creep, shrinkage, temperature and moisture.
5. Loads used in the design of masonry.
6. Specified compressive strength of masonry at stated ages or stages of construction for which masonry is designed, except where specifically exempted by this code.
7. Details of anchorage of masonry to structural members, frames, and other construction, including the type, size, and location of connectors.
8. Size and location of conduits, pipes, and sleeves.
9. The minimum level of testing and inspection as defined in Chapter 17, or an itemized testing and inspection program that meets or exceeds the requirements of Chapter 17.

Reason: The primary purpose of this code change proposal is to update the existing reference standards for the design and construction of masonry from the 2005 editions to the 2008 editions. The revisions proposed in this code change, while relatively minor in nature, reflect revisions incorporated into the 2008 edition of the Building Code Requirements for Masonry Structures and Specification for Masonry Structures. This code change proposal is one of several to harmonize the design and construction requirements for masonry within the IBC with those in the reference standard. A complete list of revisions incorporated into the referenced standards is available for download at www.masonrystandards.org.

An editorial change to the titles of ACI 530.1 and ASCE/SEI 6 as shown in Chapter 35 is also proposed.

With the publication of the 2008 edition of the Building Code Requirements for Masonry Structures and Specification for Masonry Structures, The Masonry Society (TMS) has become the lead sponsoring organization of the Masonry Standards Joint Committee (MSJC), which is charged with reviewing and maintaining the provisions in the referenced standards. As such, the official designation of these standards has changed from ACI 530/ASCE 5/TMS 402 to TMS 402/ACI 530/ASCE 5 and from ACI 530.1/ASCE 6/TMS 602 to TMS 602/ACI 530.1/ASCE 6 as reflected in the above proposed modifications.

The additional information proposed to be included in Section 2101.3 simply reflect revisions to the reference standards regarding the minimum information to be included on the construction documents. The proposed additions are consistent in intent with existing requirements in Chapters 1 and 16 as well as other material chapters.

Cost Impact: The code change proposal will not increase the cost of construction.

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF