CHAPTER 18

FOUNDATIONS AND RETAINING WALLS

SECTION 1801
GENERAL

1801.1 Scope. The provisions of this chapter shall govern the design and construction of foundations and retaining walls.

1801.2 Design. Foundations shall be designed to provide adequate load bearing capacity while limiting settlement, heave and lateral movement to tolerable levels. 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 structural materials shall 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.

1801.3 Foundation types not covered by code provisions. Types of foundations not specifically covered by the provisions of this code, and ground modification treatments to improve soils with inadequate load bearing capacity or settlement characteristics, may be permitted subject to approval by the building official. A report shall be submitted to the building official that identifies the foundation as a type not covered by existing code provisions, and contains sufficient data and analyses to substantiate the adequacy of the proposed foundation. The report shall be prepared by a registered design professional who is knowledgeable in the design of the proposed type of foundation or ground modification. The building official may require that an independent peer review be performed to evaluate the adequacy of the proposed design.

1801.4 Qualifications of registered design professional. Where the services of a registered design professional are required by the provisions of this Chapter, the registered design professional shall be knowledgeable in the field of geotechnical engineering and shall be qualified by appropriate training and experience to provide the specific services required by the code provisions.

SECTION 1802
FOUNDATION AND SOILS INVESTIGATIONS

1802.1 General. Foundation and soils investigations shall be conducted in conformance with Sections 1802.2 through 1802.6. The scope of the investigation shall be determined by a registered design professional. The registered design professional shall oversee the performance of the investigation and prepare a report containing the results of the investigation.

1802.2 Where required. Foundation and soil investigations shall be required for all new structures, and for alterations of existing structures that result in increased foundation loads or decreased load resistance, except as noted below. Where there is sufficient local experience that indicates satisfactory performance of similar structures in the vicinity of the proposed structure, the following types of structures do not require a foundation and soils investigation, unless specifically required by the building official:

1. one- and two-family dwellings and their accessory buildings;
2. unoccupied structures that do not pose a significant risk to public safety in the event of failure; or
3. structures used for agricultural purposes.

1802.3 Soil and rock classification. Classification of soil materials shall be done in accordance with ASTM D 2487. Refer to commentary in 780 CMR 120.R for guidelines regarding determination of soil and rock Material Classes and Consistency in Place for use with Table 1804.3 and related provisions.

1802.4 Investigation. The investigation shall consist of subsurface explorations and engineering studies. The borings, test pits or other subsurface explorations shall be adequate in number and depth and so located to accurately define the nature of the subsurface materials necessary for the support of the structure. When it is proposed to support the structure directly on bedrock, the building official shall require core borings to be made into the rock; or shall require adequate information from other sources to define the nature and quality of the bedrock. The results of previous subsurface explorations may be utilized where deemed to be adequate by the registered design professional. Engineering studies shall be made as necessary to identify suitable bearing strata and evaluate soil strength, compressibility, bearing capacity, settlement, liquefaction potential, lateral earth pressures, slope stability and impacts of groundwater.

1802.4.1 Groundwater table. The investigation shall determine the location of the groundwater level relative to the proposed structures and evaluate the potential groundwater impacts. The evaluation shall consider both the impacts of groundwater on the structures and the potential impacts of the structures on the local groundwater levels.

1802.4.2 Pile and pier foundations. Where pier or pile foundations are used, the investigation shall include the following:

1. Recommended pier or pile types, dimensions and installed capacities.
2. Recommended center-to-center spacing of piles or piers.
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.
10. Evaluation of conditions that might promote deterioration of pile or pier foundations, in order to satisfy the requirements of Section 1808.2.17.

1802.4.3 Rock foundations. Where subsurface explorations at the project site indicate variations or doubtful characteristics in the structure of the rock upon which foundations are to be constructed, a sufficient number of borings shall be made to a depth of not less than ten feet below the level of the foundations to provide assurance of the soundness of the foundation stratum and its bearing capacity.

1802.4.4 Liquefaction potential. Where the foundation investigation indicates subsoils of Material Classes 8 or 9, as defined in Table 1804.3, the investigation shall include an evaluation of the potential for earthquake-induced liquefaction in accordance with Section 1804.6.

1802.5 Borings, sampling and testing. The scope of the subsurface exploration, including the number and types of borings, soundings or test pits, the equipment used to drill and sample, the in-situ testing equipment and the laboratory testing program, shall be determined by a registered design professional. The boring and sampling procedure and apparatus shall be in accordance with generally accepted engineering practice. The registered design professional shall have a qualified representative on the site during the subsurface exploration and shall perform suitable checks to verify the adequacy of information obtained from others.

1802.6 Reports. The design bearing strata and load-bearing capacity shall be shown on the construction document. The written report of the investigation shall include, but need not be limited to, the following information:

1. A plot showing the locations of the subsurface explorations.
2. Logs or other suitable records of the subsurface explorations.
3. An interpretative description of the subsurface profile.
4. Elevation and/or depth of the water table, if encountered, and an evaluation of groundwater impacts in accordance with Section 1802.4.1.
5. Results of in-situ testing and/or laboratory testing, if performed.
6. Recommendations for foundation type and design criteria.
7. Evaluation of earthquake liquefaction potential
8. Expected total and differential settlement.
9. Pile and pier foundation information in accordance with Section 1802.4.2
10. Compacted fill material properties and testing in accordance with Section 1803.5.
11. Lateral earth pressures on foundation walls and retaining walls.

SECTION 1803
EXCAVATION, GRADING AND FILL

1803.1 Excavations near footings or foundations. Excavations for any purpose shall not remove vertical or lateral support from any footing or foundation without first underpinning or protecting the footing or foundation against settlement or lateral translation.

1803.2 Placement of fill adjacent to structures. Fill placed adjacent to structures shall be a suitable material placed and compacted in a manner that does not damage the foundation or the waterproofing or damp-proofing material.

1803.3 Site grading. The ground immediately adjacent to the foundation shall be sloped away from the building at a slope of not less than one unit vertical in 20 units horizontal (5-percent slope) for a minimum distance of 10 feet (3048 mm) measured perpendicular to the face of the wall or an alternate method of diverting water away from the foundation shall be used. The procedure used to establish the final ground level adjacent to the foundation shall account for potential settlement of the backfill.

1803.4 Compacted structural fill material. Where footings or structures will bear on compacted fill material, the compacted fill shall comply with the provisions of the report prepared by a registered design professional in accordance with Section 1802.6. The report shall contain the following:

1. Specifications for the preparation of the site prior to placement of compacted fill material.
2. Specifications for material to be used as compacted fill.
3. Lateral and vertical limits of the compacted fill.
4. Type and size of compaction equipment and compaction procedures.
5. Maximum allowable thickness of each lift of compacted fill material.
6. Test method to be used to determine the maximum dry density and optimum moisture content of the material to be used as compacted fill.
7. Field test methods for determining the in-place dry density of the compacted fill.
8. Minimum acceptable in-place dry density expressed as a percentage of the maximum dry density determined in accordance with Item 6, and the number and frequency of field tests required to determine compliance with this criterion.
9. Alternative compaction control criteria for materials than cannot be effectively tested by the methods described in Items 6, 7 and 8.

Exception: Compacted fill material less than 12 inches (305 mm) in total thickness does not require a soils investigation report, provided it has been compacted to a minimum of 95 percent Modified Proctor in accordance with ASTM D 1557. The compaction shall be verified by a qualified inspector approved by the building official.

1803.5 Field Control. The building official will require that a registered design professional or his representative be on the project at all times while fill is being placed and compacted. The representative shall make an accurate record of the types of materials used, including grain-size curves, thickness of lifts, densities, percent compaction, type of compacting equipment and number of coverages, the use of water and other pertinent data.

1803.6 Controlled low-strength material (CLSM). Controlled low strength material (CLSM) is defined as a flowable cementitious fill material meeting the requirements of ACI 229R-94 that is placed without compaction. Where footings or structures will bear on CLSM, the CLSM shall comply with the provisions of a report prepared by a registered design professional, which shall contain the following:

1. Specifications for the preparation of the site prior to placement of the CLSM.
2. Specifications for the CLSM.
3. Laboratory or field test method(s) to be used to determine the compressive strength or bearing capacity of the CLSM.
4. Test methods for determining the acceptance of the CLSM in the field.
5. Number and frequency of field tests required to determine compliance with Item 4.

SECTION 1804
ALLOWABLE LOAD-BEARING VALUES

1804.1 Satisfactory foundation materials. Foundations for structures shall be supported on natural strata of rock, gravel, sand, inorganic silt, inorganic clay, or combination of these materials provided that they do not overlie an appreciable amount of peat, organic silt, soft clay or other objectionable materials. Compacted fills and CLSM may be used for support of foundations, when designed and monitored by a registered design professional. Other conditions of unsatisfactory bearing materials which are improved in accordance with the recommendations of, and monitored by, a registered design professional, may be accepted by the building official.

1804.2 Protection of bearing strata. The bearing strata shall be adequately protected against disturbance. If the bearing materials are disturbed from any cause, for example, by the inward or upward flow of water, by freezing or by construction activities, the extent of the disturbance shall be evaluated by a registered design professional and appropriate remedial measures shall be taken.

1804.3 Presumptive load bearing values. The maximum allowable pressure on the bearing strata supporting spread footings, mats and piers shall not exceed the values specified in Table 1804.3 or the maximum allowable pressure shall be determined by load tests conducted in the field or as otherwise provided herein. Presumptive load-bearing values shall apply to all materials with similar physical characteristics. Where the bearing strata are underlain by weaker materials, the vertical pressure on the underlying materials shall be checked in accordance with Section 1804.7. Higher allowable bearing pressures may used if substantiated by the results of investigations, analyses or testing prepared by a registered design professional and approved by the building official.

1804.3.1 Net Bearing Pressure. For the purposes of Section 1804.3, the applied bearing pressure shall be the net bearing pressure computed using the allowable stress design load combinations specified in Section 1605.3, including the weight of the foundation and of any immediately overlying material, minus the pressure applied by the column of soil, including the water in its voids, that extends from the lowest immediately adjacent surface of the soil to the bottom of the footing, pier or mat.

1804.3.2 Classification of bearing materials. The terms used in Section 1804 shall be interpreted in accordance with generally accepted engineering nomenclature. Refer to commentary in 780 CMR 120.R for guidelines regarding soil and rock classification and description.

1804.3.3 Compacted fill. Materials from Classes 6 through 8, Table 1804.3, or dense graded crushed stone, which contain no plastic fines, shall have a maximum allowable bearing pressure of up to five tons per square foot when compacted to 95% or greater of the maximum dry density as determined by ASTM D1557 listed in. For compacted fills which do not meet the above criteria or materials which cannot be tested as above, a registered design professional shall be engaged to provide recommendations for compaction and maximum allowable design bearing pressures.

TABLE 1804.3
ALLOWABLE BEARING PRESSURES FOR FOUNDATION MATERIALS

Material Class	Description	Notes	Consistency in Place[1]	Allowable Net Bearing Pressure (tons/ft^2)
1a	Massive bedrock: Granite, diorite gabbro, basalt, gneiss	3	Hard, sound rock, minor jointing	100
1b	Quartzite, well cemented conglomerate	3	Hard, sound rock moderate jointing	60
2	Foliated bedrock: slate, schist	3	Medium hard rock, minor jointing	40
3	Sedimentary bedrock: cementation shale, siltstone, sandstone, limestone, dolomite, conglomerate	3, 4	Soft rock, moderate jointing	20
4	Weakly cemented sedimentary bedrock: compaction shale or other similar rock in sound condition	3	Very soft rock	10
5	Weathered bedrock: any of the above except shale.	3, 5	Very soft rock, weathered and/or major jointing and fracturing	8
6	Slightly cemented sand and/or gravel, glacial till (basal or lodgment), hardpan	7,8	Very dense	10
7	Gravel, widely graded sand and gravel; and granular ablation till	6, 7, 8	Very dense Dense Medium dense Loose Very loose	8 6 4 2 Note 11
8	Sands and non-plastic silty sands with little or no gravel (except for Class 9 materials)	6, 7, 8, 9	Dense Medium dense Loose Very loose	4 3 1 Note 11
9	Fine sand, silty fine sand, and non-plastic inorganic silt	6, 7, 9	Dense Medium dense Loose Very loose	3 2 1 Note 11
10	Inorganic sandy or silty clay, clayey sand, clayey silt, clay, or varved clay; low to high plasticity	5, 6, 10	Hard Stiff Medium Soft	4 2 1 Note 11
11	Organic soils: peat, organic silt, organic clay	11		Note 11

Notes for Table 1804.3:

1. Refer to commentary in 780 CMR 120.R regarding typical index test values that may be helpful as guides for evaluation of consistency in place.
2. Refer to Section 1804.3.1 for the definition of net bearing pressure.
3. The allowable bearing pressures may be increased by an amount equal to 10% for each foot of depth below the surface of sound rock; however, the increase shall not exceed two times the value given in the table.
4. For limestone and dolomite, the bearing pressures given are acceptable only if an exploration program performed under the direction of a registered design professional demonstrates that there are no cavities within the zone of influence of the foundations. If cavities exist, a special study of the foundation conditions is required.
5. Weathered shale and/or weathered compaction shale shall be included in Material Class 10. Other highly weathered rocks and/or residual soils shall be treated as soil under the appropriate description in Material Classes 6 to 10. Where the transition between residual soil and bedrock is gradual, a registered design professional shall make a judgment as to the appropriate bearing pressure.

6. Settlement analyses in accordance with Section 1804.8 should be performed if the ability of a given structure to tolerate settlements is in question, particularly for, but not limited to, soft or very soft clays and silts and loose granular materials.
7. Allowable bearing pressures may be increased by an amount equal to 5% for each foot of depth of the bearing area below the minimum required in Section 1805.2; however, the bearing pressure shall not exceed two times the value given in the table. For foundation bearing areas having a least lateral dimension smaller than three feet, the allowable bearing pressure shall be 1/3 of the tabulated value times the least dimension in feet.
8. Refer to Section 1804.3.3 when these materials are used as compacted fills.
9. These materials are subject to the provisions in Section 1804.6 (Liquefaction).
10. Alternatively, the allowable bearing pressure may be taken as 1.5 times the peak unconfined compressive strength of undisturbed samples for square and round footings or 1.25 times that strength for footings with length to width ratio of 4 or greater. For intermediate cases, interpolation may be used.
11. A registered design professional shall be engaged to provide recommendations for these special cases. Direct bearing on organic soils is not permitted. Organic soils are allowed under foundations for those cases defined in Section 1804.4.2 (Preloaded materials).

1804.4 Lightweight structures. One-story structures without masonry walls and not exceeding 800 square feet in area may be founded on a layer of satisfactory bearing material not less than three feet (1 m) thick, which is underlain by highly compressible material, provided that the stresses induced in the unsatisfactory material by the live and dead loads of the structure, and the weight of any new fill within or adjacent to the building area, will not exceed 250 pounds per square foot (12 KPa).

1804.4.1 Bearing capacity for lightweight structures. Lightweight structures and accessory structures, such as garages and sheds, may be founded on normally unacceptable bearing strata, provided such material is determined by a registered design professional as being satisfactory for the intended use.

1804.4.2 Preloaded materials. The building official may allow the use of certain otherwise unsatisfactory natural soils and uncompacted fills for support of one-story structures after these materials have been preloaded to effective stresses not less than 150% of the effective stresses which will be induced by the live and dead loads of the structure and any additional fill materials. The preloading shall be conducted under the direction of a registered design professional who shall submit a report documenting that adequate consolidation of the material was achieved prior to removal of the preload.

1804.5 Dynamic Loading. For load combinations that include short duration dynamic loading, the allowable load bearing value of foundations may be increased based on consideration of both the duration of the loading and the dynamic properties of the soil if substantiated by the results of investigations, analyses or testing prepared by a registered design professional and approved by the building official.

1804.6 Liquefaction.

1. The potential for liquefaction induced by the design earthquake in saturated clean to silty medium to fine sands (Soil Classes 8 and 9 in Table 1804.3) shall be evaluated on the basis of Figures 1804.6(a) through (c) for cases where lateral sliding cannot occur. Figure 1804.6(a) shall be used if the Standard Penetration Test blowcounts, N (blows per foot), were determined using the standard 140-pound donut drop weight, or if the type of hammer is not known. Figures 1804.6(b) and (c) shall be used only for cases where the specific type of hammer (safety hammer or automatic hammer) is known to have been used. Figures 1804.6(b) and (c) reflect the greater energy efficiency with these two specific types of hammers. Hammer type shall be as described in ASTM Standard Method D6066. N-values to be used in these figures are uncorrected field values.
2. Figures 1804.6(a) through (c) are intended to be used as a screening tool for Site Classes A through D, and are based on a rock spectral rock acceleration, $S_{D1} = 0.077$, a soil amplification factor, $Fv = 2.4$ for Site Class D and a factor of safety of 1.1.
3. If the standard penetration resistances, N, in all strata of medium to fine sand lie above the applicable curve in Figure 1804.6(a) through (c), the sands at the site shall be considered not susceptible to liquefaction. Liquefaction below a depth of 60 ft (18m) from final grade need not be considered for level ground. For pressure-injected footings, the ten-foot (3m) thickness of soil immediately below the bottom of the driven shaft shall be considered not susceptible to liquefaction.
4. Compacted granular fills shall be considered not susceptible to liquefaction provided they are systematically compacted to at least 93% of the maximum dry density determined in accordance with ASTM Standard Method D1557, or to a relative density of at least 60% in the case of granular soils having less than 10% by weight material passing the No. 200 U.S. sieve.
5. Soils identified in Section 1804.6.1 that do not meet the criteria in Sections 1804.6.3 and 1804.6.4 shall be considered susceptible to liquefaction. Appropriate studies, analyses and designs shall be made by a registered design professional to determine that the structural loads can be safely supported. Such studies, analyses and designs might include the following:
    a. Investigations to establish that the soils at the site are not subject to liquefaction during the design earthquake, or
    b. Design of foundations that will not fail either by loss of bearing capacity or excessive settlement or lateral deflection if liquefaction occurs, or

c. Replacement or densification of the potentially liquefiable soils such that liquefaction will not occur.

6. For sites underlain by saturated sands where lateral sliding (slope instability) may occur, studies by the registered design professional shall be made to establish the safety against sliding during the design earthquake.
7. For sites underlain by saturated silty sands and inorganic non-plastic silts, with standard penetration resistances, N, lying below (to the left of) the appropriate curve in Figure 1804.6(a) through (c), studies shall be made by the registered design professional to determine the susceptibility to liquefaction of these soils.

1804.7 Vertical pressure. The computed vertical pressure at any level beneath a foundation shall not exceed the allowable bearing pressure for the material at that level. Computation of the vertical pressure in the bearing materials at any depth below a foundation shall be made on the assumption that the load is spread uniformly at an angle of 30 degrees with the vertical, or another suitable method. Interaction of adjacent foundations shall be considered in computation of the vertical pressure.

1804.8 Settlement analysis. Whenever a structure is to be supported by medium or soft clay (materials of Class 10) or other materials which may be subject to settlement or consolidation, the settlements of the structure and of neighboring structures due to consolidation shall be given careful consideration, particularly if the subsurface material or the loading is subject to significant variation. The effects of fill loads within the building area or fill and other loads adjacent to the building shall be included in the settlement analysis.

1804.9 Lateral loads on foundations. Where foundations are required to resist lateral loads, the allowable values of sliding friction, adhesion and passive pressure for design shall be determined by a registered design professional.

SECTION 1805
FOOTINGS AND MAT FOUNDATIONS

1805.1 General. Footings and mat foundations shall be designed and constructed in accordance with Sections 1805.1 through 1805.12. Footings and mat foundations shall be supported on undisturbed natural soil, compacted fill, CLSM or bedrock in accordance with Section 1804. Compacted fill material shall be placed in accordance with Section 1803.4 and CLSM shall be placed in accordance with Section 1803.6.

1805.2 Minimum depth. The bottom surface of any foundation bearing on material other than sound bedrock shall be at least 18 inches (460 mm) below the adjacent ground surface or the top surface of a floor slab bearing directly on the soil immediately adjacent to the foundation.

1805.3 Slope of bearing surface. Where the slope of the bearing surface exceeds one vertical in ten horizontal, the bottom surface of the foundation shall be stepped to prevent lateral sliding along the bearing surface, unless sufficient resistance against sliding is provided by other means.

1805.4 Frost protection. All foundations for buildings and structures shall extend to a minimum of four feet (1.2m) below finished grade, unless:

1. Frost protection is provided in accordance with ASCE-32;
2. The foundation bears on sound bedrock; or
3. The bearing materials below the foundation, to a depth of four feet below finished grade, are not susceptible to frost heave.

Where the foundation grade is established at a depth of less than four feet below finished grade, appropriate supporting data prepared by a registered design professional shall be submitted to the building official to substantiate the design foundation grade.

Exception: Frost protection is not required for free-standing buildings meeting all of the following requirements:

1. Classified in Importance Category I (see Table 1604.5);
2. Area of 400 square feet (37 m2) or less; and
3. Eave height of 10 feet (3.1 m) or less.

1805.5 Isolated footings. Footings on granular soil of Classes 7, 8 and 9 of Table 1804.3 and compacted fill shall be so located that the line drawn between the lower edges of adjoining footings shall not have a steeper slope 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 established by a registered design professional.

1805.6 Foundations on or adjacent to slopes. Where buildings and structures are located on or adjacent to slopes the following impacts shall be addressed in the design. Foundations located on a slope, or near the top edge of a slope, shall have sufficient embedment depth below the slope surface and/or setback from the edge of the slope to prevent slope stability failure or detrimental settlement due to the presence of the slope. Buildings and structures located near the bottom of a slope shall be setback a sufficient distance from the slope to provide protection from slope drainage, erosion, slope failure and falling debris.

1805.7 Design. Foundations shall be designed so as not to exceed the allowable bearing pressure determined in accordance with Section 1804.3. The minimum width of the foundations shall not be less than 12 inches (305 mm). Footings shall be designed and constructed in accordance with Sections 1805.8 through 1805.12.

1805.7.1 Vibratory loads. Where machinery operations or other vibrations are transmitted through the foundation, consideration shall be given in the foundation design to prevent detrimental disturbance of the soil.

1805.7.2 Eccentric loads. Where foundations are subject to eccentric loadings, the maximum pressure on the basis of straight-line distribution shall not exceed the allowable bearing pressure.

1805.8 Concrete footings. The design, materials and construction of concrete footings shall comply with Sections 1805.8.1 through 1805.8.6 and the provisions of Chapter 19.

1805.8.1 Concrete strength. Concrete in footings shall have a specified compressive strength (f'_c) of not less than 2,500 psi (17.2 MPa) at 28 days.

1805.8.2 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 and buildings less than two stories in height of light- frame construction, 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.

1805.8.3 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.

1805.8.4 Protection of concrete. Concrete footings shall be protected from freezing during depositing and for a period of not less than 5 days thereafter. Water shall not be allowed to flow through the deposited concrete.

1805.8.5 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, forming shall be in accordance with Chapter 6 of ACI 318.

1805.9 Masonry-unit footings. The design, materials and construction of masonry-unit footings shall comply with Sections 1805.9.1 and 1805.9.2, and the provisions of Chapter 21.

1805.9.1 Dimensions. Masonry-unit footings shall be laid in Type M or S mortar complying with Section 2103.7 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.

1805.9.2 Offsets. The maximum offset of each course in brick foundation walls stepped up from the footings shall be $1^1/_2$ inches (38 mm) where laid in single courses, and 3 inches (76 mm) where laid in double courses.

1805.10 Steel grillage footings. Grillage footings of structural steel shapes shall be separated with approved steel spacers and shall 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.

1805.11 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 C2 or C3. 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 piles shall not exceed 70 percent of the allowable stresses for the species and grade of timber as specified in the AFPA NDS.

1805.12 Wood foundations. Wood foundation systems shall be designed and installed in accordance with AFPA Technical Report No. 7. Lumber and plywood shall be treated in accordance with AWPA C22 and shall be identified in accordance with Section 2303.1.8.1.

1805.13 Seismic requirements. See Section 1910 for additional requirements for footings and foundations of structures assigned to Seismic Design Categories C and D. For structures assigned to Seismic Design Categories D, provisions of ACI 318, Sections 21.8.1 to 21.8.3, 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.

Exception: Group R or Group 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.

1805.14 Footing seismic ties. Where a structure is assigned to Seismic Design Category D in accordance with Section 1616, individual spread footings founded on soil defined in Section 1615.1.1 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, by the lateral confinement provided by competent soil or rock surrounding the footing or by other approved means.

SECTION 1806
FOUNDATION WALLS AND RETAINING WALLS

1806.1 Definitions. The following definitions shall apply for the purposes of Section 1806.

FOUNDATION WALL. A wall extending partially or fully below grade, that supports vertical loads from walls, piers, columns or other structural elements of a building or other structure, and may also support unbalanced lateral loads from soil, water or other materials adjacent to the wall.

RETAINING WALL. A wall or other retaining structure or system designed to provide lateral support of soil, water or other materials, that does not provide vertical support for structural elements of a building and has any portion of its exposed face inclined steeper than 1 horizontal to 1 vertical. This definition shall include concrete walls, crib and bin walls, reinforced or mechanically stabilized earth systems, anchored walls, soil nail walls, multi-tiered systems, boulder walls, or other retaining structures.

Exception: This definition does not include slope facings, armor or riprap placed for the sole purpose of protection against surface erosion.

1806.2 Design Earth Pressures. The design earth pressures for foundation walls and retaining walls shall be determined by a registered design professional based on the data and recommendations provided in the foundation and soils investigation report prepared in accordance with Section 1802. The design earth pressures shall account for the following factors:

1. The nature of the native soil and backfill material
2. Drainage provisions and potential hydrostatic pressures
3. The rigidity of the wall and its supports
4. Pressures due to compaction of backfill material
5. Pressures due to surcharge loadings
6. Seismic earth pressures

1806.3 Foundation Wall Design. Concrete and masonry foundation walls shall be designed in accordance with Section 1806 and Chapter 19 or 21. Foundation walls for single level basements that are within the parameters of Tables 1806.3(1) through 1806.3(4) may be designed and constructed in accordance with Sections 1806.3.1 through 1806.3.5, if they meet the following criteria:

1. The wall is laterally supported at the top and bottom
2. There is no net hydrostatic pressure on the wall
3. There are no surcharge loadings
4. The backfill adjacent to the wall is not subjected to heavy compaction loads

1806.3.1 Foundation wall thickness. The minimum thickness of concrete and masonry foundation walls shall comply with Sections 1806.3.1.1 through 1806.3.1.3.

1806.3.1.1 Thickness based on walls supported. 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 1806.3.1.2 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 be a full course of headers at least 6 inches (152 mm) in length, extending not higher than the bottom of the floor framing.

Insert Tables 1806.3(1) through 1806.3(4), including notes beneath the Table

1806.3.1.2 Thickness based on soil loads, unbalanced backfill height and wall height. The thickness of foundation walls shall comply with the requirements of Table 1806.3(1) for plain masonry and plain concrete walls or Table 1806.3(2), 1806.3(3) or 1806.3(4) for reinforced concrete and masonry walls. When using the tables, masonry shall be laid in running bond and the mortar shall be Type M or S.

Unbalanced backfill height is the difference in height of the exterior and interior finish ground levels. Where an interior concrete slab is provided, the unbalanced backfill height shall be measured from the exterior finish ground level to the top of the interior concrete slab.

1806.3.1.3 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 and D.

1806.3.2 Foundation wall materials. Foundation walls constructed in accordance with Tables 1806.3(1), 1806.3(2), 1806.3(3) or 1806.3(4) 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 1806.3(2), 1806.3(3) and 1806.3(4) and shall be measured from the face of the soil side of the wall to the center of 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.
3. Concrete shall have a specified compressive strength of not less than 2,500 psi (17.2 MPa) at 28 days.
4. Grout shall have a specified compressive strength of not less than 2,000 psi (13.8 MPa) at 28 days.
5. Hollow masonry units shall comply with ASTM C 90 and shall be installed with Type M or S mortar.

1806.3.3 Alternative foundation wall reinforcement. In lieu of the reinforcement provisions in Table 1806.3(2), 1806.3(3) or 1806.3(4), alternative reinforcing bar sizes and spacings having an equivalent cross-sectional area of reinforcement per linear foot (mm) of wall are 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.

1806.3.4 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.

1806.3.5 Seismic requirements. Tables 1806.3(1) through 1806.3(4) shall be subject to the following limitations in Sections 1806.3.5.1 and 1806.3.5.2 based on the seismic design category assigned to the structure as defined in Section 1616.

1806.3.5.1 Seismic requirements for concrete foundation walls. Concrete foundation walls designed using Tables 1806.3(1) through 1806.3(4) shall be subject to the following limitations:

1. Seismic Design Categories A and B. No limitations, 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. Tables shall not be used except as allowed for plain concrete members in Section 1910.4.
3. Seismic Design Category D. Tables shall not be used except as allowed for plain concrete members in ACI 318, Section 22.10.

1806.3.5.2 Seismic requirements for masonry foundation walls. Masonry foundation walls designed using Tables 1806.3(1) through 1806.3(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 1806.3(1) through 1806.3(4) subject to the seismic requirements of Section 2106.4.
3. Seismic Design Category D. A design using Tables 1806.3(2) through 1806.3(4) subject to the seismic requirements of Section 2106.5.

1806.4 Retaining Wall Design. Retaining walls shall be designed to resist the static and seismic pressures of the retained materials, water pressures, and dead and live load surcharges to which such walls are subjected, and to ensure stability against excessive movements, overturning, sliding, excessive foundation pressure and water uplift. The structural design of retaining walls shall conform to the requirements of Chapters 16, 19, 21, 22 and 23. Retaining walls that support an unbalanced height of retained material greater than six feet (1.83 m), and any retaining system or slope that could impact public safety or the stability of an adjacent structure, shall be designed by a registered design professional.

1806.4.1 Overturning and Sliding Stability. Retaining walls shall be designed for a minimum safety factor of 1.5 against overturning and lateral sliding for static loading conditions.

1806.4.2 Overall Stability. The overall, global stability of a retaining wall, considering potential failure surfaces extending through the materials located below, in front of and behind the wall, shall be evaluated.

1806.4.3 Discrete Elements. For retaining walls constructed of discrete elements, such as unmortared masonry, rock, boulders or stacked modular units, the elements shall be bonded or fastened together to prevent dislodgment under static and seismic loading conditions where dislodgment of the elements could pose a risk to public safety.

1806.4.4 Coping. All masonry retaining walls, other than reinforced concrete walls, shall be protected with an approved coping.

1806.4.5 Guards. Where retaining walls with differences in grade level on either side of the wall in excess of four feet (1.2 m) are located closer than two feet (0.6 m) to a walk, path, parking lot or driveway on the high side, such retaining walls shall be provided with guards or other approved protective measures.

1806.5 Wall drainage. Foundation walls and retaining walls shall be designed to support a hydrostatic head of water pressure equal to the full height of the wall, unless a drainage system is installed in accordance with Sections 1807.4.2 and 1807.4.3. Where a drainage system is provided to reduce or eliminate the design hydrostatic pressure on the wall, it shall be designed with sufficient permeability and discharge capacity to permanently prevent hydrostatic pressure on the wall, and shall be provided with appropriate filters and other design features to prevent blockage due to siltation, clogging or freezing.

1806.6 Seismic Design. Foundation walls and retaining walls shall be designed to resist seismic lateral pressures in accordance with 780 CMR 16.

SECTION 1807
DAMPPROOFING, WATERPROOFING AND GROUNDWATER CONTROL

1807.1 Where required. Walls or portions thereof that retain earth and enclose interior spaces and floors below grade shall be waterproofed or 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 1202.4.

1807.1.1 Story above grade. Where a basement is considered a story above grade 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 and a foundation drain shall be installed in accordance with Section 1807.4.2. The foundation drain shall be installed around the portion of the perimeter where the basement floor is below ground level. The provisions of Sections 1802.4.1, 1807.3 and 1807.4.1 shall not apply in this case.

1807.1.2 Underfloor space. The finished ground level of an underfloor 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 where there is evidence that the surface water does not readily drain from the building site, the ground level of the underfloor space shall be as high as the outside finished ground level, unless an approved drainage system is provided. The provisions of Sections 1802.4.1, 1807.2, 1807.3 and 1807.4 shall not apply in this case.

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 underfloor space such as a crawl space shall be equal to or higher than the outside finished ground level.

1807.1.3 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. 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 pump; and the rated capacity of the disposal area of the system. The design shall also take into account any adverse impacts on utilities, structures or other facilities in the vicinity which would result from the lowering of groundwater levels.

1807.2 Dampproofing required. Where hydrostatic pressure will not occur as determined by Section 1802.4.1, 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 AFPA TR7.

1807.2.1 Floors. Dampproofing materials for floors shall be installed between the floor and the base course required by Section 1807.4.1, 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 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/m^2) of acrylic modified cement, $^1/_8$-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, or other approved methods or materials.

1807.2.2.1 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 below ground level with not less than $^3/_8$ 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 Waterproofing required. Where the ground-water investigation required by Section 1802.4.1 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, walls and floors shall be waterproofed in accordance with this section.

1807.3.1 Floors. Floors required to be waterproofed shall be of concrete, 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, 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 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. 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 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.

1807.3.3 Joints and penetrations. Joints in walls and floors, joints between the wall and floor, and penetrations of the wall and floor shall be made watertight utilizing approved methods and materials.

1807.4 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 shall be deemed adequate for lowering the ground-water table.

1807.4.1 Floor base course. Floors of basements, except as provided for in Section 1807.1.1, 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 Foundation drain. A drain shall be placed around the perimeter of the 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, and shall be covered with not less than 6 inches (152 mm) of the same material.

1807.4.3 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.

1807.4.4 Erosion protection. Where water impacts the ground from the edge of the roof, downspout, scupper or other rainwater collection or diversion device, provisions shall be made to prevent soil erosion and direct the water away from the foundation.

1807.5 Impacts on groundwater levels. Below-grade structures and their appurtenances shall be designed and constructed so as not to cause changes to the temporary or permanent groundwater level if such changes could adversely impact nearby structures or facilities including deterioration of timber piles, settlement, flooding or other impacts.

SECTION 1808
PIER AND PILE FOUNDATIONS

1808.1 Definitions. The following words and terms shall, for the purposes of this section, have the meanings shown herein.

PIER FOUNDATIONS. Piers are defined herein as circular or non-circular deep foundation elements constructed of concrete that are installed or cast-in-place in holes advanced to the bearing stratum using non-displacement excavation methods. Piers develop their load-bearing capacity through end-bearing and/or skin friction in the bearing stratum. Piers shall have a minimum lateral dimension of not less than 12 inches (305 mm). The base of the pier may be enlarged to increase the bearing area (referred to as a belled pier). Pier foundations include foundation types referred to as caissons and drilled shafts.

PILE FOUNDATIONS. Piles are deep foundation elements constructed of wood, steel or concrete, driven, jacked, cast in-place, or a combination thereof. Piles develop their load-bearing capacity through end-bearing and/or skin friction in the bearing stratum. Types of pile foundations covered by the provisions of this code include timber, precast concrete, structural steel, concrete-filled steel pipe or tube, steel-cased concrete (mandrel-driven shell), enlarged base piles (pressure-injected footings), hollow-stem augered piles, small diameter grouted piles and caisson piles (concrete-filled pipe with steel core socketed into bedrock).

1808.2 Piers and piles - general requirements.

1808.2.1 General. Pier and pile foundations shall be designed and installed on the basis of a foundation investigation as defined in Section 1802. The design and installation shall be under the direct supervision of a registered design professional knowledgeable in the field of geotechnical engineering and pier or pile foundations who shall verify that the piers or piles as installed satisfy the design criteria. The investigation and report shall conform to the provisions of Section 1802.

1808.2.2 Special types of piers and piles. Types of piers and piles not specifically covered by the provisions of Section 1808 may be permitted in accordance with Section 1801.3.

1808.2.3 Pier and pile caps. Pier and pile caps shall be of reinforced concrete, and shall include all elements to which piles or piers are connected, including beams and mats. The soil immediately below the cap shall not be considered as carrying any vertical load. The tops of piles or piers shall be embedded not less than 3 inches (76 mm) into caps and the caps shall extend at least 4 inches (102 mm) beyond the edges of piles or piers. The tops of piles or piers shall be cut back to sound material before capping.

1808.2.4 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 or 35 feet (10,668 mm) in height, provided the centers of the piles are located within the width of the foundation wall.

Individual columns supported on piles shall be designed for eccentricity between the column and the centroid of the supporting piles equal to a minimum of three inches (76 mm) or the actual eccentricity, whichever is greater. The design shall account for such eccentricity through one of the following methods:

1. By supporting the column on a minimum of three piles in a triangular pattern.
2. By designing walls, grade beams or structural floors to resist the bending moment induced by the eccentricity.
3. By designing the piles, column or both to resist the bending moment induced by the eccentricity and providing adequate lateral restraint at the top of the piles to resist the lateral thrust due to the bending moment.

1808.2.5 Structural integrity. Piers or piles shall be installed in such a manner and sequence as to prevent distortion or damage to piles being installed or already in place to the extent that such distortion or damage affects the structural integrity of the piles.

When piles have been damaged in driving, or have been driven in locations and alignment other than those indicated on the plans, or have capacities less than required by the design, the affected pile groups and pile caps shall be investigated, and if necessary, the pile groups or pile caps shall be redesigned or additional piles shall be driven to replace the defective piles.

1808.2.6 Spacing. The minimum center-to-center spacing of piles shall be not less than twice the average diameter of a round pile, nor less than 1-3/4 times the diagonal dimension of a rectangular pile. When driven to or penetrating into rock, the spacing shall be not less than 24 inches (610 mm). When receiving principal support from end-bearing on materials other than rock or through frictional resistance, the spacing shall be not less than 30 inches (762 mm) or as provided in Section 1810.2 Enlarged Base Piles.

The minimum center-to-center spacing between adjacent piers designed for friction support shall be not less than two times the shaft diameter.

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 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 1808.2.4 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 General. The allowable load on piles or piers shall not exceed the allowable load-bearing capacity in the bearing strata determined in accordance with Section 1808.2.8.2 for piles or Section 1808.2.8.3 for piers, or the allowable load on the pile or pier cross-section computed using the allowable stresses for the particular type of pile or pier in accordance with Sections 1809 through 1812. The allowable load may also be limited by the bearing capacity of weaker strata underlying the bearing stratum (Section 1808.2.8.4), group action (Section 1808.2.8.4), drivability (Section 1808.2.16) or settlement (Section 1808.2.12).

1808.2.8.2 Piles

1808.2.8.2.1 Allowable load-bearing capacity. The allowable load-bearing capacity obtained from bearing on or embedment in the pile bearing strata shall be determined by load tests performed in accordance with Section 1808.2.8.2.4, or by use of applicable formulas as permitted in Section 1808.2.8.2.2. The allowable load-bearing capacity of jacked piles shall be determined in accordance with Section 1808.2.8.2.5.

The allowable pile capacity determined from load tests can be applied to other piles within the area of substantially similar subsurface conditions as those for the test pile, providing the performance of the test pile has been satisfactory and the remaining piles are of the same type, shape and size as the test pile and are installed using the same methods and equipment and are driven into the same bearing strata as the load-tested pile to an equal or greater penetration resistance.

Pile load tests required by the provisions of this Section may be waived by the building official, where justified, upon submittal of substantiating data prepared by a registered design professional which include experience and/or performance records for the proposed pile installation under similar soil and loading conditions.

1808.2.8.2.2 Pile capacity not exceeding 50 tons. For piles with a design load capacity not exceeding 50 tons, the allowable load-bearing capacity can be determined using either the driving formula or friction formula in clay as permitted in this Section, or the formula for enlarged base piles in Section 1810.2.4.

Driving formula. Where the design load capacity of the pile does not exceed 50 tons, the allowable load may be computed by means of the following driving formula:

$$R = \frac{2E}{(S + C)}$$

where:

R = allowable pile load in pounds;
E = energy per blow in foot-pounds;
S = penetration of last blow or average penetration of last few blows expressed in inches; and
C = constant equal to 1.0 for drop hammer and 0.1 for steam or air hammer.

2. When the design load capacity of a pile exceeds 50 tons, the required driving resistance shall be increased above that required by the driving formula based on load tests or past experience under similar conditions.
3. The value of S must be determined with the hammer operating at 100% of the rated number of blows per minute for which the hammer is designed.
4. Any driving resistance developed in strata overlying the bearing material shall be discounted.
5. If the driving of the pile has been interrupted for more than one hour, the value of S shall not be determined until the pile is driven at least an additional 12 inches (305 mm), except when it encounters refusal on or is in a material of Classes 1 through 6.
6. When any pile is driven through a layer of gravel, sand or hard clay exceeding five feet in thickness, and through an underlying soft stratum to reach the bearing stratum, the bearing capacity shall not be determined in accordance with the driving formula, unless jetting is used during the entire driving of the pile through the layer of gravel, sand or hard clay or unless a hole is pre-excavated through said layer for each pile.

Friction formula in clay. Where the design load does not exceed 50 tons for driven concrete or timber piles, where predrilling does not remove more than 75 percent of the pile cross section, the allowable load on a pile stopped in soil of Material Class 10 (Table 1804.3) of medium to hard consistency may be based on a friction value of 500 psf of embedded pile surface. Higher design loads or other friction values shall be determined by pile load tests in accordance with Section 1808.2.8.2.4 or Section 1808.2.8.2.6.

The embedded length shall be the length of the pile below the surface of the Class 10 soil or below the surface of immediately overlying satisfactory bearing material. The area of embedded pile surface shall be computed by multiplying the embedded length by the perimeter of the smallest circle or polygon that can be circumscribed around the average section of the embedded length of the pile. The method of determining the allowable load described in this Section shall not be used for a pile in which the drive pipe is withdrawn or for piles which are driven through the clay to or into firmer bearing materials.

In case these piles are in clusters, the allowable load on the cluster shall be computed for the smaller of the following two areas: the sum of the embedded pile surfaces of individual piles; or the area obtained by multiplying the perimeter of the polygon circumscribing the cluster at the surface of the satisfactory bearing material by the average embedded length of the piles.

1808.2.8.2.3 Pile capacity exceeding 50 tons. When the design load capacity of a pile exceeds 50 tons, the required driving resistance shall be determined based on one or more compression load tests unless waiver of such test(s) is approved by the building official.

1808.2.8.2.4 Compression load test. Compression load tests shall be performed in accordance with the following:

Required test load. A single pile shall be load-tested to not less than twice the allowable design load. When two or more piles are to be tested as a group, the total load shall be not less than 1.5 times the allowable design load for the group. In no case should the load reaching the top of the bearing stratum under maximum test load for a single pile or pile group be less than the following:

Case A-piles designed as end-bearing piles: 100% of the allowable design load.

Case B-piles designed as friction piles: 150% of the allowable design load.

For piles designed as combination end-bearing and friction piles, Case A applies if the pile is designed to support more than 50% of its design in bearing; otherwise, Case B applies.

Internal instrumentation. The test pile shall be instrumented in accordance with the requirements in paragraph 4.4.1 of ASTM D1143 to enable measurement or computation of the load in the pile where it enters the bearing stratum. For piles containing concrete, instrumentation shall be installed in the test pile to permit direct measurement of the elastic modulus of the pile. This requirement is waived for the following cases:

1. The test pile is installed within a casing that extends to within ten feet above the bearing stratum.
2. The pile to be tested has been functioning satisfactorily under load for a period of one year or more.
3. The pile is 30 feet long or less and no appreciable load will be supported above the bearing stratum.

Loading procedure. Pile load tests shall be conducted in accordance with ASTM D1143, Standard Method of Testing Piles Under Static Axial Compressive Load, except that Section 5, Loading Procedures, shall be deleted and replaced by the following provisions:

1. Apply 25% of the allowable design load every 0.5 hour. Longer time increments may be used, but each time increment should be the same. In no case shall a load be changed if the rate of settlement is not decreasing with time.
2. At 200% of the allowable design load (or 150% for pile groups), maintain the load for a minimum of one hour and until the settlement (measured at the lowest point on the pile at which measurements are made) over a one-hour period is not greater than 0.01 in.
3. Remove 50% of the design load every 15 minutes until zero load is reached. Longer time increments may be used, but each should be the same.
4. Measure rebound at zero load for a minimum of one hour.
5. For each load increment or decrement, take readings at the top of the pile and on the internal instrumentation at one, two, four, eight and 15 minutes and at 15-minute intervals thereafter.

A load greater than 200% of the allowable design load (or 150% of the allowable design load for pile groups) may be applied at the top of the pile, using the above loading procedure, to ensure that the requirement for minimum load reaching the bearing stratum is fulfilled. Other optional methods listed in ASTM D1143 may be approved by the building official upon submittal in advance of satisfactory justification prepared by a registered design professional who is qualified in this field.

Selection of design load. Provided that the design load does not exceed 100% of the load supported in the bearing stratum (or 2/3 of the load supported in the bearing stratum for friction piles or pile groups) when the maximum test load is applied, then the allowable design load shall be the greater of the following:

1. Allowable design load based on settlement during loading: 50% of the applied test load which causes a

gross settlement at the pile cutoff grade equal to the sum of: a) the theoretical elastic compression of the pile in inches assuming all the load on the butt is transmitted to the tip, plus b) 0.15 inch (3.8 mm), plus c) 1% of the pile tip diameter or pile width in inches. If the settlements are so small that the load-settlement curve does not intersect the failure criterion, the allowable design load shall be 50% of the maximum test load.

2. Allowable design load based on the net settlement after rebound: 50% of the applied test load which results in a net settlement at the top of the pile of 0.5 inch (13 mm) after rebound for a minimum of one hour at zero load.

1808.2.8.2.5 Jacked piles. Load testing of jacked piles shall be performed in accordance with the following:

1. Not less than 10% of jacked piles shall be load-tested to twice the design load (load test piles). All other jacked piles shall be founded in the same bearing stratum as the load test piles and shall be proof-loaded to 125% of design load (production piles).
2. For production piles, the 125% of design load shall be maintained for at least 30 minutes. Acceptability criteria: during final 15 minutes of load, the rate is not progressive (plot is linear or decreasing when settlement is plotted against logarithm of time); and the rate of settlement is equal to or less than that observed for load test piles during the corresponding time period under 125% of design load.
3. Settlement readings shall be plotted after one, two, four, eight, and 15 minutes, and at 15-minute intervals thereafter. Load shall be maintained on production piles until acceptability criteria are met.
4. For load test piles, the load shall be applied directly to 125% of design load and maintained for not less than 30 minutes, and until the settlement rate is not progressive (as defined above). Load shall then be increased to twice the design load and maintained constant for not less than four hours. Settlement during the four hour period shall not exceed 0.050 inches (1.3 mm). In the event that settlement exceeds 0.050 inches (1.3 mm) in four hours, the pile shall be deemed unacceptable for 50% of the final load. The allowable load on the rejected pile may be established by performing an additional load test at the lesser design load. The design load shall not exceed 50% of the load maintained for a four hour period during which time settlement did not exceed 0.050 inches (1.3 mm).

1808.2.8.2.6 Tension load tests. The allowable load on piles in tension shall be verified by test unless it is waived by the building official. Pile load test may be waived by the building official, where justified, upon submittal of substantiating data which includes experience and/or performance records for pile installations under similar soil and loading conditions prepared by a registered design professional experienced in the geotechnical aspects of foundation design.

Required load test. A single pile or a pile group shall be load tested to not less than 200% of the design load for transient loads (i.e., earthquake and wind) and 250% for sustained loads.

Test setup and loading procedure. The load test setup, instrumentation and loading procedure shall be in accordance with ASTM D3689.

Selection of design load. Provided the allowable design load does not exceed the allowable stresses in the pile materials, the allowable design load shall be the lower of the following:

1. 50% (for transient loads) or 40% (for sustained loads) of the applied test load which results in a net upward movement of 0.5 inch at the top of the pile after removal of the maximum test load (The gross upward movement minus the rebound movement).
2. 50% (for transient loads) or 40% (for sustained loads) of the applied test load which results in continuous upward movement with no increase in load.

1808.2.8.3 Piers. The allowable bearing pressure on the bottom of the pier shall be in accordance with Section 1804. Additional load may be carried by using higher bearing pressures than allowed by Section 1804 and/or by friction on the sides of the pier embedded in suitable bearing material based on recommendations by a registered design professional and subject to the approval of the building official. Such recommendations shall be based on the results of load tests or other suitable tests or analyses carried out to determine side friction and/or end bearing of piers installed in the same bearing stratum.

1808.2.8.4 Load-bearing capacity. Piers, individual piles and groups of piles or piers 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.

Group Action. In cohesive soils, the compressive load capacity of a group of friction piles shall be analyzed by a generally accepted engineering method, and, where such analysis indicates, the individual allowable pile load shall be reduced accordingly.

1808.2.8.5 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.

1808.2.8.6 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 determined otherwise by a registered design professional.

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 load test setup instrumentation and loading procedure shall be in accordance with ASTM D3966. The design load shall be selected by the responsible registered design professional, based upon interpretation of the load-deflection data from the load test. 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 through 1812 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, wave equation analyses of pile driveability, analyses of load transfer during testing, and prediction of performance during long term service.
2. Pier or pile load tests in accordance with Section 1808.2.8.2.4, regardless of the load supported by the pier or pile.

1808.2.11 Piles and piers in subsiding areas. Where piles and piers 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 or piers by the subsiding upper strata.

Where the influence of subsiding fills is considered as imposing loads on the pile or pier, the allowable stresses specified in Sections 1809 through 1812 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 neither cause harmful distortion of, or instability in, the structure, nor cause any stresses to exceed allowable values.

1808.2.13 Pre-excavation. The use of jetting, augering or other methods of pre-excavation shall be in accordance with the recommendations of the registered design professional. Where used, pre-excavation 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 pre-excavated 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 1808.2.8.2.4. 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 handling and driving stresses without damage to the pile and to provide sufficient stiffness to transmit the required driving forces. Driven piles of uniform cross section or tapered piles shall have a minimum nominal diameter of eight inches (200 mm) except as provided in Section 1809.1.3.3 for timber piles. Tapered shoes or points of lesser dimensions may be attached to the pile unit.

1808.2.17 Protection of pile and pier 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 in accordance with the recommendations of the registered design professional. 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 that demonstrates their effectiveness.

Steel and steel-concrete piles. At locations where steel and steel-concrete piles will be in contact with any material which is corrosive to the steel, one of the following procedures shall be used for protection, or any other method which will satisfy the requirements of the building official:

1. Remove all objectionable material.
2. Effectively protect the steel surface from pile cutoff grade to a grade 15 feet (4.6 m) below the bottom of the objectionable material by means of:
    a. cathodic protection as approved by the building official;
    b. an approved encasement of not less than three inches (76 mm) of dense concrete;
    c. an effective protective coating subject to the approval of the building official; or
    d. providing an excess steel thickness of 1/8 inch (3.2 mm) beyond design requirements on all exposed steel surfaces.

1808.2.18 Use of existing piers or piles. Piles or piers that have previously supported a partially or fully demolished structure shall not be used for support of new construction unless satisfactory evidence is submitted to the building official which indicates that the piles or piers have not been adversely impacted by the demolition, are sound, have adequate capacity to support the new design loads, and meet all of the requirements of Chapter 18. The capacities of such piles or piers shall be determined by analyses, load testing or redriving.

Requirements for piles. The design load for piles to be reused shall not exceed the greater of the following values:

1. Actual sustained load determined to have been previously supported satisfactorily by the piles, up to a maximum of 120 tons
2. The documented, as-built design capacity of the piles, as confirmed by prior load testing
3. Design capacity determined by analyses and confirmed by new load testing or by redriving per Section 1808.2.8.2.2 on one or more piles representative of each configuration (s) of pile and subsurface conditions.

1808.2.19 Heaved piles. Adequate provision shall be made to observe pile heave. Accurate reference points shall be established on each pile immediately after installation; for cast-in-place piles with unfilled corrugated shells, the reference point shall be at the bottom of the pile. If, following the installation of other piles in the vicinity, heaving of 1/2 inch (13 mm) or more occurs, the heaved piles shall be re-driven to develop the required capacity and penetration, or the capacity of the pile may be verified by load tests in accordance with Section 1808.2.8.2.4.

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 shall be provided for piles and piers, respectively. The owner shall engage a registered design professional who shall submit his qualifications in writing to the building official. This design professional, or his representative, who must be qualified by experience and training, shall be present at all times while piles and piers are being installed to observe all work. The design professional or his representative shall make an accurate record of the following:

1. For driven piles, the material and the principal dimensions of each pile, weight and fall of the ram, the type, size and make of hammer, cushion blocks, the number of blows per minute, the energy per blow, the number of blows per inch for the last six inches (150 mm) of driving, together with the grades at point and cutoff and any other pertinent details. For cast-in-place piles, installation equipment and methods used, casings, pile dimensions, reinforcement, grouting volumes and procedures used, and all other pertinent installation data.
2. For piers, pier installation procedures including observations and tests of the bearing material in place, pier dimensions and placement of concrete and reinforcement. When direct inspection of the bearing surface is impossible, a suitable method shall be employed, to verify the condition of the bearing material and to make the measurements and tests.

A copy of these records shall be signed by the registered design professional, and filed in the office of the building official.

1808.3 Seismic design of piers or piles.

1808.3.1 Seismic reinforcement. Piles and piers shall have minimum longitudinal reinforcement and confining reinforcement in accordance with the provisions for specific pile types and piers in Sections 1809 through 1812.

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.

Where a minimum length for reinforcement or the extent of closely spaced confining 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 cut-off.

Where confining reinforcement is specified at the top of the pile or pier, alternative measures for laterally confining concrete and maintaining toughness and ductile-like behavior at the top of the pile or pier will be permitted provided the design is such that any hinging occurs in the confined region.

1808.3.2 Seismic bending design. Piles or piers for structures assigned to Seismic Design Category D in accordance with Section 1616 shall be designed and constructed to withstand the 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. Concrete piers or piles on Site Class E or F sites, as determined in 1615.1.1, 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 the interfaces of soft to medium stiff clay or liquefiable strata.

1808.3.3 Pile cap seismic connection. Piles or piers for structures assigned to Seismic Design Category C and D in accordance with Section 1616 shall be connected to the pile or pier caps in accordance with the following provisions.

1808.3.3.1 Concrete piles and piers. These provisions apply to the pile types and piers covered in Sections 1809.2, 1810 and 1812. The piles or piers shall be connected to the pile or pier cap by extending the longitudinal reinforcing into the cap for a distance equal to the development length as specified in ACI 318. The development length to be provided in the cap is the full development length for compression without reduction in length for excess area. Field-placed dowels anchored in the pile or pier concrete at least twice the required cap development length are acceptable.

For concrete-filled steel pipe and tube piles, longitudinal reinforcement shall be provided at the top of the pile in accordance with Section 1810.6.4.1, and the reinforcement shall be anchored into the pile cap in accordance with the requirements of this Section.

For precast prestressed concrete piles in structures assigned to Seismic Design Category C, the pile cap connection can be made by extending the exposed pile reinforcing strand into the pile cap with an embedment length sufficient to develop the strength of the reinforcing strand. This is not permitted for piles in structures assigned to Seismic Design Category D.

1808.3.3.2 Structural steel piles. Structural steel piles covered in Section 1809.3 shall be connected to the pile cap with a connection detail designed for a minimum tensile force equal to 10% of the pile compression design load. The pile cap connection shall be made by means other than concrete bond to the bare steel section.

1808.3.3.3 Seismic Design Category D. For structures assigned to Seismic Design Category D the pile or pier cap connection shall meet the following additional requirements. For piles or piers required to resist uplift forces or provide rotational restraint, the design of the anchorage into the pile or pier cap shall consider the combined effect of axial forces due to uplift and bending moments due to fixity at the cap. The anchorage shall develop a minimum of 25 percent of the strength of the pile or pier in tension and shall be capable of developing the following load capacities:

1. In the case of uplift, the lesser of the nominal tensile strength of the longitudinal reinforcement in a concrete pile or pier, or the nominal tensile strength of a steel pile, or the pile or pier 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 or pier.

1808.3.4 Seismic foundation ties. Pile or pier foundations for structures assigned to Seismic Design Category C and D in accordance with Section 1616 shall be interconnected by ties 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. Ties are not required where 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: Piers supporting foundation walls, isolated interior posts detailed so the pier is not subject to lateral loads, lightly loaded exterior decks and patios, of Group R, Division 3 and Group U, Division 1 occupancies not exceeding two stories of light-frame construction, are not subject to interconnection if it can be shown the soils are of adequate stiffness, subject to the approval of the building official.

SECTION 1809
DRIVEN PILE FOUNDATIONS

1809.1 Timber piles. Timber piles shall be designed in accordance with the AFPA NDS.

1809.1.1 Materials. Round timber piles shall conform to ASTM D 25. Round timber piling shall be new longleaf, shortleaf, loblolly or slash species of Southern pine, oak, Douglas fir or other woods of similar strength and physical characteristics. 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 with AWPA C3 for round timber piles, and AWPA C24 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 Allowable load.

1809.1.3.1 Allowable stress. The allowable stress in the timber shall not exceed 1,000 psi (6.9 MPa) in compression at the critical cross-sectional area taken at the top of the bearing stratum. Piles designed for end bearing on materials of Classes 1 and 2 in Table 1804.3 shall be designed for a maximum stress of 500 psi (3.4 MPa) in compression on the pile cross-sectional area at the tip.

1809.1.3.2 Maximum Load. The load on timber piles shall not exceed the allowable load specified in 1808.2.8 nor 35 tons, whichever is smaller.

1809.1.3.3 Minimum dimensions. Timber piles shall be no less than six inches (152 mm) in diameter at the tip.

1809.1.4 Precautions during driving.

1809.1.4.1 Hammer energy. Pile hammer energy shall be selected to prevent damage to the pile, but in no case shall the maximum hammer energy, as rated by the manufacturer, exceed 18,000 ft.-lbs.

1809.1.4.2 End-supported piles. Any sudden decrease in driving resistance of an end-supported timber pile shall be investigated with regard to the possibility of damage. If the sudden decrease in driving resistance cannot be correlated to load-bearing data, the pile shall be removed for inspection or rejected.

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. When driven to or into bearing materials of Classes 1 and 2 inclusive, or through materials containing boulders, piles shall have metal tips of approved design.

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 shall 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 (25.4 mm) center to center. The gage of ties and spirals shall be as follows:

1. 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).
2. 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).
3. For piles having a diameter of 20 inches (508 mm) and larger, wire shall not be smaller than $^1/_4$ 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 non-prestressed 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 4,000 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 1616, 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 (9.5mm) 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. Where a structure is assigned to Seismic Design Category D in accordance with Section 1616, the requirements for Seismic Design Category C in Section 1809.2.2.2.1 shall apply except as modified in 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, it shall be permitted to use a volumetric ratio of spiral reinforcement of not less than one-half that required by Section 21.4.4.1(a) of ACI 318.

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), nor 1,600 psi (11.0 MPa), 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 $1^1/_4$ inches (32 mm) for No. 5 bars and smaller, and not less than $1^1/_2$ inches (38 mm) for No. 6 through No. 11 bars except that longitudinal bars spaced less than $1^1/_2$ 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 sea water 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 (2.76 MPa) 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 1616, 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:

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

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 1803.3.3 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. Where a structure is assigned to Seismic Design Category D in accordance with Section 1616, 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 longitudinal 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 Sect. 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:

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

but not less than:

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

and need not exceed:

$\rho_s = 0.021$ **(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, hc, shall conform to:

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

but not less than:

$A_{sh} = 0.12sh_c(f'_c/f_{yh})[0.5 + 1.4P/(f'_c A_g)]$ **(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 transverse 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 Allowable stresses. The maximum allowable design compressive stress, f_c, in concrete shall be determined as follows:

$f_c = 0.33\,f'_c - 0.27f_{pc},$ but not greater than 1600 psi (11.0 MPa) **(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 sea water, 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 ASTM A 36, A 252, A 283, A 572, A 588 or A 913.

1809.3.2 Allowable stresses. The allowable design compressive stress shall not exceed 35% of the minimum specified yield strength of the steel nor 12,600 psi (86.9 MPa).

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 and Tip Reinforcement 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).

1809.3.3.1 Tip reinforcement. The tips of all steel H piles having a thickness of metal less than 0.5 in. (12.7 mm) which are driven to end bearing on rock of Classes 1 through 3 by an impact hammer shall be reinforced. The installation of all steel H piles by impact hammer to end bearing on rock of Classes 1 through 3 shall be conducted so as to terminate driving when the pile reaches refusal on the rock surface.

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 of 0.34 square inch (219 mm^2) of steel in cross section to resist each 1,000 foot-pounds (1356 N-m) of pile hammer energy or the equivalent strength for steels having a yield strength greater than 35,000 psi (241 MPa). Where pipe wall thickness less than 0.188 inch (4.8 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 Section 1810.

1810.1.1 Material. All concrete shall have a 28-day specified compressive strength ($f'c$) of not less than 3,000 psi (20.7 MPa). The maximum size of coarse aggregate for all concrete shall be ¾ inch (19 mm), and the concrete shall have a slump of four to seven inches (102 mm to 178 mm). If 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 Seismic reinforcement in Seismic Design Category C. Where a structure is assigned to Seismic Design Category C in accordance with Section 1616, the following shall apply. A minimum longitudinal reinforcement ratio of 0.0025 shall be provided in the top one-third of the pile length or the length determined by analysis, but not less than 10 feet (3048 mm) below the ground. The reinforcing shall be a minimum of four longitudinal bars, with closed ties of a minimum ¼ inch (6 mm) diameter provided at 16-longitudinal-bar-diameter maximum spacing, or with equivalent spirals. Within a distance below the bottom of the pile cap equal to three times the least pile dimension, transverse confinement reinforcing shall have a maximum spacing of 6 inches (152 mm) or 8-longitudinal-bar-diameters, whichever is less.

1810.1.2.2 Seismic reinforcement in Seismic Design Category D. Where a structure is assigned to Seismic Design Category D in accordance with Section 1616, 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 in the top one-half of the pile length, a minimum of 10 ft below the 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. The reinforcing shall be a minimum of four longitudinal bars with transverse confinement reinforcing 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 a distance below the bottom of the pile cap equal to three times the least pile dimension. It shall be permitted to use a transverse spiral reinforcing 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. Tie spacing throughout the remainder of the concrete section shall not 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.

Exception: Confinement by ties or spirals is not required where permanent metal casing (steel pipe, steel tube or spiral-welded steel shell) is used, provided the casing has minimum thickness as follows: for Seismic Performance Category C structures, 0.058 inch (1.5 mm), and for Seismic Performance Category D structures, 0.070 inch (1.8mm). The steel casing must be adequately protected from corrosion due to soil, changing water levels or other subsurface conditions indicated by the site soil investigation.

1810.1.3 Concrete placement. For all cased piles, the inside of the pipe or casing shall be thoroughly cleaned to the bottom and visually inspected prior to filling with concrete. Concrete shall be placed in such a manner as to ensure the exclusion of any foreign matter and to secure a full-sized shaft. The concreting shall be subject to the following limitations:

1. The diameter shall not vary more than 20% from the specified value.
2. Concrete shall not be placed through water except where tremie methods are approved.
3. When depositing concrete from the top of the pile, the concrete flow shall be rapid and continuous through a funnel hopper centered at the top of the pile.
4. After filling with concrete, the top ten feet (3 m) shall be thoroughly rodded.
5. No pile shall be installed within a distance of nine feet (2.7 m) from a pile which has been filled with concrete for less than 12 hours, unless approved.

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 stress on shaft concrete shall be 33% of the 28-day strength, but not exceeding 1,600 psi (11.0 MPa). The maximum allowable design stress on permanent steel casing, if at least 1/10-inch (2.5 mm) thick, and on steel reinforcing shall be 40% of the minimum specified yield strength, but not exceeding 24,000 psi (165 MPa).

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 re-establish lateral support by the soil. The installation of enlarged base piles having compacted concrete bases shall fulfill the following requirements:

1810.2.3.1 Base.

1. The enlarged base shall be formed on or in bearing materials of Classes 1 to 9 inclusive. The Class 9 material (fine sand) shall have a maximum of 15% by weight finer than the No. 200 mesh sieve and shall be non-plastic, unless satisfactory load test results or other substantiating data are submitted to, and approved by, the building official.
2. The compacted concrete placement shall be in measured batches, to establish impact energy required per unit volume of concrete. A minimum of one Standard Batch Volume of concrete, as defined in Table 1810.4, shall be injected in the base, after expulsion of the concrete plug or boot used to close the tube during the driving process.

1810.2.3.2 Shaft installation.

1810.2.3.2.1 Uncased compacted-concrete shaft.

1. Concrete shall be placed at zero slump, in small batches, and shall be compacted in place in a controlled manner as the drive-tube is withdrawn.
2. Enlarged base piles formed through soils of Classes 10 and 11, located less than nine feet or within the heave range from an uncased shaft, shall be pre-drilled through such soil.
3. An uncased compacted-concrete shaft shall not be formed through very soft to soft soils of Classes 10 and 11. The building official may waive this requirement based upon satisfactory evidence prepared by a registered design professional that the soil has sufficient strength for proper shaft construction.
4. A suitable method shall be employed by the contractor and the design professional to verify and record that the entire length of the shaft is completely filled with concrete. Such means shall include the ability to determine the incremental volume of concrete installed in relation to the calculated shaft volume.

1810.2.3.2.2 Uncased high-slump concrete shaft.

1. Concrete shall be placed at not less than eight-inch slump, except that slump as low as four inches may be allowed if adequate vibration is applied to the drive-tube during the entire withdrawal process. During withdrawal, the level of concrete within the tube shall have a positive differential head over external soil and water pressures at all times.
2. The shaft shall be provided with full-length reinforcing steel anchored in the enlarged base. At a minimum, provide a cage with four, full length, number five reinforcing bars evenly spaced near the shaft perimeter.
3. Enlarged base piles located less than nine feet (2.7 m) from a completed uncased high-slump shaft shall not be installed until at least 12 hours after shaft pour.
4. A suitable method shall be employed by the contractor and the design professional to verify and record that the entire length of the shaft is completely filled with concrete. Such means shall include the ability to determine the incremental volume of concrete installed in relation to the calculated shaft volume.

1810.2.3.2.3 Cased shaft. The permanent metal casing shall be fastened to the enlarged base in such a manner that the two will not separate.

1810.2.4 Load-bearing capacity. Load-bearing capacity shall be verified by load tests in accordance with Section 1808.2.8.2.4.

1. For enlarged base piles with compacted concrete bases and loads up to 120 tons, the allowable load may be computed by the following formula:

   $R = [(B \times E)/C]\ V^{2/3}$

   where:

   R = allowable load in pounds;

   B = average number of blows required to inject one cubic foot of concrete, during injection of the last batch;

   E = Energy per blow in foot-pounds;

   C = constant; and

   V = total volume of base concrete in cubic feet.

   The values of R, E, and C shall conform to Table 1810.2.4 unless other values are determined by load test, in which case the latter values shall control. Use of Table 1810.2.4 is limited by the provisions of Section 1808.2.8.2.2.

   The value of V shall include an allowance of one Standard Batch Volume of concrete, if concrete is used in the tube during the driving process, plus the additional volume of concrete injected during formation of the base.
2. During injection of the last batch of concrete in the base, the height of concrete within the drive tube shall not be more than a of the drive-tube inside diameter.

TABLE 1810.2.4

R (tons)	Energy, E (foot-pounds)	C	Standard Batch Volume

			(cubic feet)
over 100	140,000	18	5
51 to 100	100,000	18	5
25 to 50	60,000	30	2

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.

1810.2.6 Seismic Reinforcement. Where a structure is assigned to Seismic Design Category C or D in accordance with Section 1616, the corresponding requirements of Sections 1810.1.2.1 and 1810.1.2.2 shall be met.

1810.2.7 Spacing. The center-to-center spacing with uncased shafts shall be not less than 2.5 times the outside diameter of the drive tube and not less than 3.5 feet. The center-to-center spacing with cased shafts shall be not less than 3.0 times the shaft diameter.

1810.2.8 Loading on underlying strata. Where the bearing stratum is underlain by weaker material, or if the bearing stratum becomes weaker with depth, the vertical pressure on the underlying weaker material shall not exceed the allowable bearing pressure of the weaker material determined in accordance with Section 1804.3. The vertical pressure can be computed by assuming that the load is spread uniformly at an angle of 30 degrees with the vertical, starting at a polygon circumscribing the pile or pile group at the junction of the shaft and enlarged base, but not extending beyond intersecting 30 degree planes from adjacent piles or pile groups.

1810.3 Augered uncased piles. Augered uncased piles shall conform to Sections 1810.3.1 through 1810.3.5.

1810.3.1 Allowable stresses. The design stresses shall not exceed the following values:

1. For compression loads: The maximum allowable design stress on the cement grout or concrete shall be 33% of the specified 28- day unconfined compressive strength, but not exceeding 1,600 psi (11.0 MPa). The maximum allowable design stress on the steel reinforcing shall be 40% of the minimum specified yield strength, but not exceeding 24,000 psi (165 MPa).
2 For tension loads: The maximum allowable design tensile stress on the steel reinforcing shall be 60% of the minimum specified yield strength. The allowable design tensile stress on the cement grout shall be zero.

1810.3.2 Dimensions. An augered uncased pile is defined as a structural member installed utilizing a hollow-stem auger no less than 12 inches (305 mm) in outside diameter which extends to satisfactory bearing materials to develop support by end bearing and/or friction in those materials. The design pile diameter shall be taken as the outside diameter of the hollow stem auger. The minimum center-to-center spacing between adjacent piles shall not be less than 30 inches (760 mm) or two times the pile diameter, whichever is greater. In addition, for groups of friction piles, the overall circumference of a pile group shall exceed the sum of the circumferences of all of the individual piles within the group.

1810.3.3 Installation. Augered uncased piles shall be formed by advancing a closed-end continuous-flight hollow-stem auger of uniform diameter through unsuitable material and into a satisfactory bearing material followed by removal of the tip closure and pumping cement grout or concrete through the hollow-stem while the hollow-stem auger is extracted. During advancement, the hollow-stem auger shall be rotated rapidly such that the material through which the auger is being advanced is removed by the auger flights and is not displaced laterally by the auger. During withdrawal, if the hollow stem auger is rotated, it shall be rotated in a positive (advancing) direction.

1. The grout or concrete shall be pumped under continuous pressure and in one continuous operation. Grout or concrete pump pressures shall be measured and maintained at all times sufficiently high to offset hydrostatic and lateral earth pressures. The rate of withdrawal of the auger shall be carefully controlled to exclude all foreign matter and ensure that the augered hole is completely filled with grout or concrete as the auger is withdrawn. The actual volume of grout or concrete pumped into each hole shall be equal to, or greater than, the theoretical volume of the augered hole.
2. If the grouting or concreting process of any pile is interrupted, or a loss of concreting pressure occurs, the pile shall be redrilled to its original depth plus six inches (152 mm) (unless bearing on rock) and filled from the bottom.
3. Augered uncased piles shall not be installed within six pile diameters (center-to-center) of a pile filled with grout or concrete less than 24-hours old except where approved by the registered design professional.

1810.3.3.1 Records. The owner shall engage a registered design professional to monitor the installation of augered uncased piles in accordance with Section 1808.2.22. The design professional or his representative shall make an accurate record of the installation equipment used, pile dimensions, grout or concrete volumes, reinforcement, interruptions or delays in pile installation, and all other pertinent installation data.

1810.3.3.2 Instrumentation. The continuous-flight auger rig utilized to install augered uncased piles shall be equipped with data logging equipment that automatically monitors and produces a real-time printout of depth, grout or concrete pressure, grout or concrete flow, and rate of auger withdrawal. The automatic monitoring equipment shall immediately indicate to the equipment operator, and record on the printed record, any instance during the withdrawal of the hollow-stem auger where the rate of auger withdrawal times the theoretical pile cross-sectional area exceeds the rate of grout or concrete placement. Printed instrumentation readout for each pile shall be provided to the design professional's representative upon completion of each pile.

1810.3.4 Reinforcement. 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 semi-fluid state.

1810.3.5 Seismic Reinforcement. Where a structure is assigned to Seismic Design Category C or D in accordance with Section 1616, the corresponding requirements of Sections 1810.1.2.1 and 1810.1.2.2 shall be met.

1810.4 Driven uncased piles. No provisions.

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 watertight 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). Concrete shall satisfy the provisions of Section 1810.1.1. The shape of the pile may be cylindrical, or conical, or a combination thereof, or it may be a succession of cylinders of equal length, with the change in diameter of adjoining cylinders not exceeding one inch.

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), nor 1,600 psi (11.0 MPa). The allowable concrete compressive stress shall be 0.40 (f'_c) for that portion of the pile meeting the following conditions.

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 shall be 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).

The load on cased poured concrete piles shall be applied to the cross-sectional area computed on the following basis:

1. For metal-cased piles driven to and into materials of Classes 1 to 4 inclusive, using the diameter measured one foot (0.3 m) above the point, except that when the rock is immediately overlain by a bearing stratum consisting of one or a combination of bearing materials of Classes 5, 6, and 7, using the diameter at the surface of the bearing stratum.
2. For metal-cased piles, driven through compressible materials including Classes 10 and 11 and into a bearing stratum consisting of one or a combination of bearing materials of Classes 5-9 inclusive, using the diameter at the surface of the bearing stratum.

1810.5.3 Installation. Steel shells shall be mandrel-driven their full length in contact with the surrounding soil.

Steel shells shall be driven in such order and with such spacing as to ensure against distortion of or damage 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 registered design professional. Concrete shall not be placed in steel shells within heave range of driving. The requirements of Section 1810.1.3 shall apply.

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 or D in accordance with Section 1616, the corresponding requirements of Sections 1810.1.2.1 and 1810.1.2.2 shall apply.

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 A 283. Concrete shall conform to Section 1810.1.1. The maximum coarse aggregate size shall be 0.75 inch (19.1 mm).

1810.6.2 Allowable stresses. The allowable design compressive stresses shall not exceed the values indicated in Sections 1810.6.2.1 through 1810.6.2.3.

1810.6.2.1 Top driven piles**:** The allowable design compressive stress in the concrete shall not exceed 25% of the 28-day compressive strength of the concrete or 1,100 psi (7.6 MPa), whichever is smaller. The maximum allowable compressive stress in the steel shall not exceed 9,000 psi (62 MPa).

1810.6.2.2 Mandrel driven piles: For piles installed with mandrels which transmit driving stresses directly to the bottom of the steel pipe, the allowable design compressive stress in the concrete shall not exceed 33% of the 28-day specified compressive strength. The allowable design compressive stress in the steel shall not exceed 35% of the minimum specified yield strength of the steel.

1810.6.2.3 The maximum allowable design stress in the steel shall be limited to 50% of the minimum specified yield strength where higher stresses are substantiated in accordance with Section 1808.2.10.

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 0.1 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 or D in accordance with Section 1616, 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 0.188 inch (4.75 mm).

1810.6.5 Placing concrete. The placement of concrete shall conform to Section 1810.1.3.

1810.7 Caisson piles. Caisson piles shall conform to the requirements of Sections 1810.7.1 through 1810.7.8.

1810.7.1 Construction. Caisson piles shall consist of a shaft section of concrete-filled pipe extending to bedrock of Classes 1 or 2 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 1808.3. Pipes shall have a minimum wall thickness of 0.375 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 (102 mm) to 6 inches (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 socket design stress shall be determined by a registered design professional based upon foundation investigation study in accordance with Section 1802, but in no case will the design bond stress on the perimeter of the socket exceed 200 psi. Load tests, in accordance with Section 1808.2.8.2.4, may be required if foundation investigation data are judged insufficient by the registered design professional to verify the selected bond stress.

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.

1810.7.7 Seismic reinforcement. Caisson piles shall meet the seismic reinforcement requirements of Section 1810.6.4.1.

1810.7.8 Spacing. The minimum center to center spacing shall be not less than 2.5 times the outside diameter of the steel shell.

1810.8 Small diameter grouted piles.

1810.8.1 General. Section 1810.8 covers grouted cast-in-place piles which are less than 12 inches (305 mm) in diameter and in which all or a portion of the pile is cast directly against the soil without permanent casing.

1810.8.2 Materials. Concrete or sand-cement grout shall satisfy the provisions of Section 1810.1.1.

1810.8.3 Allowable stresses. The design stresses shall not exceed the following values:

1. For compression loads: The maximum allowable design stress on the cement grout or concrete shall be 33% of the specified 28- day unconfined compressive strength, but not exceeding 1,600 psi (11.0 MPa). The maximum allowable design stress on the steel reinforcing, including permanent steel casing, shall be 40% of the minimum specified yield strength. Where the design of the pile cross section includes compression on grout that is not confined within a permanent steel casing or bedrock, the allowable stress on the steel shall not exceed 24,000 psi (165 MPa).
2. For tension loads: The maximum allowable design tensile stress on the steel reinforcing shall be 60% of the minimum specified yield strength. The allowable design tensile stress on the cement grout shall be zero.

1810.8.4 Minimum reinforcing. The steel reinforcing shall be designed to carry the following minimum percentage of the design compression load:

1. For a pile or a portion of a pile that is grouted inside a drilled temporary casing or a permanent casing, grouted inside a hole drilled into stable rock, or grouted with a hollow-stem auger, the reinforcing steel shall be designed to carry not less than 40% of the design compression load.
2. For a pile or a portion of a pile that is grouted in an uncased drill hole in materials other than stable rock, or grouted in a driven temporary casing, the pile shall be designed to carry the entire design compression load on the reinforcing steel.

   Exception: Where a portion of the cement grout is placed and permanently enclosed within permanent steel casing or reinforcing consisting of steel pipe, that portion may be included at the allowable stress for the grout.

1810.8.5 Load test. For all design loads, the allowable load shall be determined by load tests in accordance with Section 1808.2.8.2.4. Load tests may be waived by the code official based on substantiating data and analyses prepared by a registered design professional.

1810.8.5.1 Alternative load test procedure for friction piles. For piles designed as friction piles, the friction capacity in compression may be verified by load testing in tension. The tension load test shall be performed in accordance with Section 1808.2.8.2.6, with the following exceptions:

1. The test pile must be cased or left ungrouted down to the top of the bearing stratum in a manner which will ensure that no friction resistance is developed above the bearing stratum.
2. The maximum design load shall be taken as 50% of the applied test load which results in a movement under load of 0.5 inch (13 mm) at the pile tip. The movement at the pile tip shall be a.) measured directly by a tell-tale or b.) computed by deducting the theoretical elastic elongation of the pile from the displacement measured at the top of the pile.

1810.8.6 Installation. The pile may be formed in a hole advanced by rotary or rotary percussive drilling methods (with or without temporary casing), by a hollow-stem auger, or by driving a temporary 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. Piles grouted with temporary casing: For piles grouted inside a temporary casing, the reinforcing steel 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. Piles grouted without temporary casing: 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 immediately prior to grouting. The reinforcing steel shall be inserted prior to grouting.
3. Piles grouted with hollow-stem augers: For piles installed with a hollow-stem auger, the grout shall be pumped under continuous pressure, and the rate of withdrawal of the auger shall be carefully controlled to ensure that the hole is completely filled with grout as the auger is withdrawn. The actual volume of grout pumped for each 1.0 foot (305 mm) of withdrawal of the auger shall be recorded and must be equal to or greater than the theoretical volume. The reinforcing steel shall be inserted prior to withdrawal of the auger.
4. For piles designed for end bearing, a suitable means shall be employed to verify that the bearing surface is properly cleaned prior to grouting.
5. Subsequent piles shall not be drilled or driven near piles that have been grouted until the grout has had sufficient time to harden.

1810.8.7 Pile diameter. The design pile diameter shall be taken as:

1. The outside diameter of the temporary casing; or
2. The diameter of a full circumferential drill bit attached to the bottom of the temporary casing; or
3. The outside diameter of the hollow-stem auger; or
4. The borehole diameter verified by suitable measurements made immediately prior to grouting.

1810.8.8 Corrosion protection.

1. Where steel reinforcing is not enclosed inside a permanent casing, centralizers shall be provided on the reinforcing to ensure a minimum grout cover of 1.0 inch (25 mm) in soil and 0.5 inch (13 mm) in rock. Grout cover requirements may be reduced when the reinforcing steel is provided with a suitable protective coating.
2. Permanent steel casing that is used as structural reinforcing shall be protected in accordance with the provisions of Section 1808.2.17.
3. For piles subjected to sustained tension loading in corrosive environments, the reinforcing steel shall be protected by a suitable protective coating or encapsulation method.

1810.8.9 Buckling. For piles that extend through more than 5 feet of soft soil of Material Classes 10 or 11 using a solid bar section for the steel reinforcing, the lateral support assumptions of Section 1808.2.9.1 shall not apply if any of the following conditions exist:

1. Steel designed for an allowable stress greater than 24,000 psi.
2. Outside diameter of the grout less than four (4) times the diameter of the steel reinforcing bar
3. Soil shear strength less than 200 psf

For these conditions, the potential for buckling in the soft soil zone shall be investigated.

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 shall be designed to meet the conditions of installation.

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 or D in accordance with Section 1616, 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 or 1810.3.5.

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 Design. Foundation piers may be designed as concrete columns with continuous lateral support below the soil level. The unit compressive stress in the concrete shall not exceed 33% of the 28-day strength of the concrete or 1,600 psi (11.0 MPa), whichever is less. The unit compressive stress in the steel reinforcement or the permanent steel casing shall not exceed 40% of the yield strength of the steel or 24,000 psi (165 MPa), whichever is less. Permanent steel casing which is used as structural reinforcement shall be protected against corrosion in accordance with Section 1808.2.17.

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.

1812.4 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 2.5 inch (64 mm) concrete cover requirement be reduced to 2 inches (51 mm) for Group R, Division 3 and Group U, Division 1 occupancies not exceeding two stories of light-frame construction, provided the construction method can be demonstrated to the satisfaction of the building official.

1812.4.1 Seismic Reinforcement. Where a structure is assigned to Seismic Design Category C or D in accordance with Section 1616, the corresponding requirements of Sections 1810.1.2.1 and 1810.1.2.2 shall be met. The minimum longitudinal reinforcement ratio shall be applied to the minimum design cross-sectional area determined in accordance with Section 1812.2.

Exceptions:

1. Isolated piers supporting posts of Group R, Division 3 and Group U, Division 1 occupancies not exceeding two stories of light-frame construction is 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, Division 3 and Group U, Division 1 occupancies not exceeding two stories of light-frame construction may 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, Division 3 and Group U, Division 1 occupancies not exceeding two stories of light-frame construction is 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, 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, Division 3 and Group U, Division 1 occupancies of Seismic Design Category D, not exceeding two stories of light-frame construction.

1812.5 Concrete placement. Concrete may be dropped into the pier from the ground surface provided no more than 3.0 inches (76 mm) of water remains in the bottom and the concrete will free-fall vertically without obstruction. The concrete shall be placed in a rapid, continuous operation and controlled such that the concrete does not segregate.

1. No piers shall be installed near a concreted pier until the concrete has set sufficiently to avoid damage to the concreted pier.
2. For piers without belled bases, concrete or grout may be placed through still water or slurry. A properly operated tremie or pumping method shall be used. Samples of the slurry shall be tested to determine the properties prior to placing concrete in each pier. The quality, consistency, and density of the slurry shall be controlled to ensure that there will be free-flow of concrete from the tremie pipe. The concrete must be placed such that all water, slurry and contaminated concrete below design cutoff level is displaced.
3. For piers with belled bases, the concrete may be placed under slurry, based upon the recommendations of a registered design professional and with the approval of the building official. The specific soil or rock conditions, equipment and procedures used shall be taken into account.
4. A suitable method shall be employed to verify that the entire length of the shaft is completely filled with concrete. Such means shall include the ability to determine the incremental volumes of concrete installed in relation to calculated shaft volume.

1812.6 Belled bases. Where pier foundations are belled at the bottom, the edge thickness of the bell shall not be less than four inches (102 mm). 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 Laterally unsupported piers. 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. 8 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. 9 Installation. Where piers are carried to depths below water level, the piers shall be constructed by a method that will ensure accurate preparation and inspection of the bottom, and the depositing or construction of sound concrete. In unstable soils, temporary casing or slurry shall be used to stabilize the excavation. When slurry is used to stabilize the excavation, the level and quality of the slurry shall be monitored and controlled to maintain stability of the shaft and the bearing surface.

1812.10 Alignment. When the center of the cross section of a foundation pier at any level deviates from the resultant of all forces more than 2 % of its height, or more than 10 % of its diameter, it shall be reinforced as provided in ACI 336. The restraining effect of the surrounding soil may be taken into account.

1812.11 Minimum spacing. The minimum center-to-center spacing between adjacent piers designed for friction support shall be not less than 2.0 times the shaft diameter.

1812.12 Special provisions. For piers with shaft diameter less than 24 inches (610 mm), the following special provisions shall apply:

1. For piers with temporary casing extending to the bottom, the concrete may be poured from the top in accordance with Section 1812.5.
2. For all other cases, piers shall be filled from the bottom upward through a tremie or concrete pump tube in accordance with Section 1812.5 (2).