CHAPTER 18

SOILS AND FOUNDATIONS

SECTION BC 1801
GENERAL

1801.1 Scope. The provisions of this chapter shall apply to building and foundation systems in those areas not subject to scour or water pressure by wind and wave action. Buildings and foundations subject to such scour or water pressure loads shall be designed in accordance with Chapter 16 and Appendix G of this code.

1801.2 Design. Allowable bearing pressures, allowable stresses and design formulas provided in this chapter shall be used with the allowable stress design load combinations specified in Section 1605.3.

The quality and design of materials used structurally in excavations, footings and foundations shall 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.

Members shall have adequate capacity to resist all applicable combinations of the loads listed in Chapter 16, in accordance with the following:

Where the structural design of soil or foundation members is based on allowable working stresses, the load reductions as described in Section 1605.3.1.1 shall be modified to use the following factors and the design shall be based on the resulting load values:

1. For the design of temporary structures, (defined for this chapter as a structure that will be in place 180 days or less) load combinations in Equations 16-8 and‡ 16-9 can be multiplied by a factor of 0.75.
2. For the design of temporary structures, the Equations 16-10, 16-11 and‡ 16-12 can be multiplied by a factor of 0.67.
3. For any combination of dead loads with three or more variable loads, these variable loads can be multiplied by a factor of 0.67.
4. For the combinations of loads to be used in the design of permanent structures, the load due to lateral earth and ground-water pressure shall be multiplied by a factor of 1.

1801.2.1 Foundation design for seismic overturning. Where the foundation is proportioned using the strength design load combinations of Section 1605.2, the seismic overturning moment need not exceed 75 percent of the value computed from Section 9.5.5.6 of ASCE 7 for the equivalent lateral force method, or Section 1618 for the modal analysis method.

SECTION BC 1802
FOUNDATION AND SOILS INVESTIGATIONS

1802.1 General. Foundation and soils investigations shall be subject to special inspections in accordance with Sections 1704.7, 1704.8 and 1704.9 and be conducted in conformance with Sections 1802.2 through 1802.6. An engineer shall scope, supervise and approve the classification and subsurface investigation of soil.

1802.2 Where required. The owner or applicant shall submit a foundation and soils investigation to the commissioner where required in Sections 1802.2.1 through 1802.2.3.

1802.2.1 Questionable soil. Where the safe load-bearing capacity of the soil is in doubt, or where a load-bearing value superior to that specified in this code is claimed, the commissioner shall require that the necessary investigation be made. Such investigation shall comply with the provisions of Sections 1802.4 through 1802.6.

1802.2.2 Seismic Design Category C. Where a structure is determined to be in Seismic Design Category C in accordance with Section 1616, an investigation shall be conducted, and shall include an evaluation of the following potential hazards resulting from earthquake motions: slope instability, liquefaction and surface rupture due to faulting or lateral spreading.

1802.2.3 Seismic Design Category D. Where the structure is determined to be in Seismic Design Category D in accordance with Section 1616, the soils investigation requirements for Seismic Design Category C, given in Section 1802.2.6, shall be met, in addition to the following:

1. A site-specific analysis in accordance with Sections 1813.2, 1813.3, and 1813.4. Site-specific response shall be evaluated for site peak ground acceleration magnitudes and source characteristics consistent with the design earthquake ground motions.
2. A determination of lateral pressures on basement, cellar, and retaining walls due to earthquake motions.
3. An assessment of potential consequences of any liquefaction and soil strength loss, including estimation of differential settlement, lateral movement or reduction in foundation soil-bearing capacity, and shall address mitigation measures. Such measures shall be given consideration in the design of the structure and shall include, but are not limited to, ground stabilization, selection of appropriate foundation type and depths, selection of appropriate structural systems to accommodate anticipated displacements or any combination of these measures. Peak ground acceleration shall be determined from a site-specific study taking into account soil amplification effects, as specified in Section 1615.2.

Exception: A site-specific study need not be performed provided that peak ground acceleration equal to SDS/2.5 is used, where SDS is determined in accordance with Section 1615.

1802.3 Material classification. Soil and rock classification shall be based on materials disclosed by borings, test pits or other subsurface exploration methods and shall be determined in accordance with Tables 1804.1 and 1804.2 and Section 1804.2. Additional laboratory tests shall be conducted to ascertain these classifications where deemed necessary by the engineer responsible for the investigation or the commissioner.

1802.3.1 General. For the purposes of this chapter, the definition and classification of soil materials for use in Table 1804.2 and Section 1804.2 shall be in accordance with ASTM D 2487.

1802.4 Investigation. Soil classification shall be based on observation and any necessary tests of the materials disclosed by borings, test pits or other subsurface exploration made in appropriate locations. Additional studies shall be made as necessary to evaluate stratigraphy, slope stability, soil strength, adequacy of load-bearing soils, the effect of moisture variation on soil-bearing capacity, compressibility, liquefaction and expansiveness.

1802.4.1 Scope of investigation. The scope of the soil investigation, including the number, types and depths of borings or test pits; the equipment used to drill and sample; the in-situ testing; and the laboratory testing program shall be determined by the engineer responsible for the investigation. Borings shall be uniformly distributed under the structure or distributed in accordance with load patterns imposed by the structure. As a minimum, investigations shall include two exploratory borings for built-over areas up to 5,000 square feet (465 m^2), and at least one additional boring for each additional 2,500 square feet (233 m^2), or part thereof, of built-over areas up to 20,000 square feet (1860 m^2). For built-over areas in excess of 20,000 square feet (1860 m^2), there shall be at least one boring for each additional 5,000 square feet (465 m^2), or part thereof. Borings shall be taken into bedrock, or to an adequate depth below the top of the load-bearing strata to demonstrate that the foundation loads have been sufficiently dissipated. For structures having an average area load (dead plus live) of 1,000 pounds per square foot (47.9 kN/m^2) or more, at least one boring for every 10,000 square feet (930 m^2) of footprint area shall penetrate at least 100 feet (30 480 mm) below the curb grade or 5 feet (1524 mm) into bedrock of Class 1c or better, whichever is less. At least one-half of the borings satisfying this requirement shall be located within the limits of the built-up area and the remainder shall be within 25 feet (7620 mm) of the built-up area limits. For structures to be supported on pile foundations, the required number of borings shall be increased by 30 percent.

The engineer responsible for the investigation shall have a qualified representative on the site inspecting all boring, sampling, and in-situ testing operations.

Exception: Test pits may be substituted for borings for one- and two-story structures, and may be used to establish the top of rock, where practical, for taller structures. In such case, there shall be prepared a test pit observation report that shall be submitted to the commissioner.

1802.4.2 Existing data. Suitable borings, test pits, probings, and the logs and records that were obtained as part of earlier exploration programs and that meet the requirements of this section may be used as partial fulfillment of the requirements of this section, subject to the approval of the commissioner. Additional borings shall be made at the direction of the engineer responsible for the investigation when uncertainty exists as to the accuracy of the available information or specific new project or loading conditions indicate the need for additional information.

1802.4.3 Ground-water table. The subsurface soil investigation shall determine the existing ground-water table.

1802.4.4 Compressible soils. In areas that have compressible soils, the investigation shall determine the extent of these soils in the plan area of the building and shall determine the preconsolidation pressure and consolidation parameters of the deposit using appropriate laboratory tests. The information shall be used in the building's foundation design.

1802.5 Soil and rock sampling. The soil boring and sampling procedures and apparatus shall be in accordance with ASTM D 1586 and ASTM D 1587 and generally accepted engineering practice. Where liquefaction assessment is performed, the investigation shall be in accordance with ASTM D 6066. The rock coring, sampling procedure and apparatus shall be in accordance with ASTM D 2113 and generally accepted engineering practice. Rock cores shall be obtained with a double-tube core barrel with a minimum outside diameter of $2^7/_8$ inches (73 mm). With the approval of the engineer responsible for the investigation, smaller-diameter double-tube core barrels may be used under special circumstances such as telescoping casing to penetrate boulders, or space limitations requiring the use of drill rigs incapable of obtaining large-diameter cores.

1802.5.1 Bedrock support. Where the foundation design relies on rock to support footings, piles or caisson sockets, a sufficient number of rock corings shall extend at least 10 feet (3048 mm) below the lowest level of bearing to provide assurance of the rock soundness. Where foundations are to rest on bedrock and such rock is exposed over a part or all of the area of the building, borings are not required in those areas where rock is exposed, provided the following requirements are met:

1. The presence of defects or the inclination of bedding planes in the rock are of such size and location so as not to affect stability of the foundation.
2. The foundation is not designed for bearing pressures exceeding those permitted in Table 1804.2.

1802.5.2 Alternative investigative methods. The engineer responsible for the investigation may engage specialized technicians to conduct alternative investigative methods such as cone penetrometer probing. Data from these investigations may be used to (1) supplement soil boring and rock coring information, provided there is a demonstrated correlation between the findings, and (2) determine material properties for static and seismic or liquefaction analyses. Subject to the approval of the commissioner, alternate exploration methods may replace borings on a two for one basis, but in no case shall there be less than two standard borings for every 10,000 square feet (930 m^2) of footprint area. All the borings shall penetrate at least 100 feet (30 480 mm) below the curb grade or 5 feet (1524 mm) into rock when the average area load equals or exceeds 1,000 pounds per square foot (48 kPa).

Other in-situ testing methods, such as geophysical, vane shear, and pressure meter, may be used to determine engineering design parameters, but may not be used as a substitute for the required number of borings.

1802.5.3 Material disposition. Soil and rock samples shall be maintained in an accessible location, by the permit holder or owner and made available to the engineer responsible for the investigation and to the department, until the foundation work has been completed and accepted, or until 1 year after the investigation is complete, whichever is longer.

1802.6 Reports. The soil classification and design load-bearing capacity shall be shown on the construction documents. Where required by the commissioner, the engineer responsible for the investigation shall sign, seal and submit a written report of the investigation that includes, but need not be limited to, the following information:

1. A description of the planned structure.
2. A plot showing the location of test borings and/or excavations.
3. A complete record of the soil sample descriptions.
4. A record of the soil profile.
5. Elevation of the water table, if encountered.
6. Results of in-situ or geophysical testing.
7. Results of laboratory testing.
8. Recommendations for foundation type and design criteria, including but not limited to bearing capacity of natural or compacted soil; mitigation of the effects of liquefaction (if applicable); differential settlement and varying soil strength; and the effects of adjacent loads.
9. Expected total and differential settlement.
10. Pile and pier foundation recommendations and installed capacities.
11. Special design and construction provisions for footings or foundations founded on expansive soils, as necessary.
12. Compacted fill material properties and testing in accordance with Section 1803.5.

For pile or pier foundations, the report shall also include:

1. Special installation procedures.
2. Pier and pile load test requirements.

SECTION BC 1803 EXCAVATION, GRADING AND FILL

1803.1 Excavations near footings or foundations. Excavations for any purpose shall not remove 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 backfill. The excavation outside the foundation shall be backfilled with soil that is free of organic material, construction debris, cobbles and boulders or a controlled low-strength material (CLSM). The backfill shall be placed in lifts and compacted, in a manner that does not damage the foundation or the waterproofing or damp proofing material.

Exception: Controlled low-strength material need not be compacted.

1803.3 Site grading. The ground immediately adjacent to the foundation shall be sloped away from the building as needed, or an approved alternate method of diverting water away from the foundation shall be used, where surface water would detrimentally affect the foundation bearing soils. Site grading shall also comply with Section 1101.11 of the *New York City Plumbing Code*.

1803.3.1 Seepage. In an excavation where soil and groundwater conditions are such that an inward or upward seepage might be produced in materials intended to provide vertical or lateral support for foundation elements or for adjacent foundations, excavating methods shall control or prevent the inflow of ground water to prevent disturbance of the soil material in the excavation or beneath existing buildings. No foundation shall be placed on soil that has been disturbed by seepage unless remedial measures have been taken.

1803.4 Grading and fill in floodways. Any floodway encroachment in areas of special flood hazard shall comply with Appendix G.

1803.5 Compacted fill material. Where footings will bear on compacted fill material, the compacted fill shall comply with the provisions of a geotechnical report, prepared, signed and sealed by the engineer, which 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. Test method to be used to determine the maximum dry density and optimum moisture content of the material to be used as compacted fill.
4. Maximum allowable thickness of each lift of compacted fill material.
5. Field test method for determining the in-place dry density of the compacted fill.
6. Minimum acceptable in-place dry density expressed as a percentage of the maximum dry density determined in accordance with Item 3.
7. Number and frequency of field tests required to determine compliance with Item 6.
8. Acceptable types of compaction equipment for the specified fill materials.

1803.6 Controlled low-strength material (CLSM). Where footings will bear on controlled low-strength material (CLSM), the CLSM shall comply with the provisions of a geotechnical report prepared, signed and sealed by the engineer, 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.

1803.7 Artificially treated soils. After the treatment procedure, a minimum of one boring shall be made for every 1,600 square feet (149 m^2) of that portion of the building that is supported on treated soil, and a sufficient number of samples shall be recovered from the treated soil to demonstrate the efficacy of the treatment.

SECTION BC 1804 ALLOWABLE LOAD-BEARING VALUES OF SOILS

1804.1 Design. The presumptive load-bearing values provided in Table 1804.1 shall be used with the allowable stress design load combinations specified in Sections 1605.3 and 1801.2.

1804.2 Allowable foundation pressure. The allowable foundation pressure for supporting soils at or near the surface shall not exceed the values specified in Table 1804.1, unless data to substantiate the use of a higher value are developed and contained in the engineer's geotechnical report and subject to the commissioner's approval. Allowable bearing pressure shall be considered to be the pressure at the base of a foundation in excess of the stabilized overburden pressure existing at the same level prior to construction operations.

1804.2.1 Classification of materials. Soil materials shall be classified and identified in accordance with Table 1804.2. In addition, refer to Sections 1804.2.1 through 1804.2.4 for supplementary definitions.

BEDROCK.

a. **Hard sound rock (Class 1a).** Includes crystalline rocks, such as gneiss, granite, diabase and schist. Characteristics are as follows: the rock rings when struck with pick or bar; {the rock} does not disintegrate after exposure to air or water; {the rock} breaks with sharp fresh fracture; cracks are unweathered and less than $^1/_8$-inch (3.2 mm) wide, generally no closer than 3 feet (914 mm) apart; the RQD (rock quality designation) with a double tube, NX-size diamond core barrel is generally 85 percent or greater for each 5-foot (1524 mm) run, or core recovery with BX-size core is generally 85 percent or greater for each 5-foot (1524 mm) run.

b. **Medium hard rock (Class 1b).** Includes crystalline rocks of paragraph (a) of this subdivision, plus marble and serpentinite. Characteristics are as follows: all those listed in paragraph (a) of this subdivision, except that cracks may be $^1/_4$-inch (6.4 mm) wide and slightly weathered, generally spaced no closer than 2 feet (610 mm) apart; the RQD with a double tube, NX-size diamond core barrel is generally between 50 and 85 percent for each 5-foot (1524 mm) run, or core recovery with BX-size core is generally 50 to 85 percent for each 5-foot (1524 mm) run.

c. **Intermediate rock (Class 1c).** Includes rocks described in paragraphs (a) and (b) of this subdivision, plus cemented shales and sandstone. Characteristics are as follows: the rock gives dull sound when struck with pick or bar; does not disintegrate after exposure to air or water; broken pieces may show weathered surfaces; may contain fracture and weathered zones up to 1 inch (25 mm) wide spaced as close as 1 foot (305 mm); the RQD with a double tube, NX-size diamond core barrel is generally 35 to 50 percent for each 5-foot (1524 mm) run, or a core recovery with BX-size core of generally 35 to 50 percent for each 5-foot (1524 mm) run.

d. **Soft rock (Class 1d).** Includes rocks described in paragraphs (a), (b) and (c) of this subdivision in partially weathered condition, plus poorly cemented shales and sandstones. Characteristics are: rock may soften on exposure to air or water; may contain thoroughly weathered zones up to 3 inches (76 mm) wide but filled with stiff soil; the RQD with a double tube, NX-size diamond core barrel is less than 35 percent for each 5-foot (1524 mm) run, or core recovery with BX-size core of generally less than 35 percent for each 5-foot (1524 mm) run and a standard penetration resistance more than 50 blows per foot (0.3 meters).

SANDY GRAVEL AND GRAVELS. Consists of coarse-grained material with more than half of the coarse fraction larger than the # 4 size sieve and contains little or no fines (GW and GP). The density of these materials shall be determined in accordance with the following:

Dense (Class 2a). Those materials having a standard penetration test *N*-value greater than 30 blows per 1 foot (0.3 meter).

Medium (Class 2b). Those materials having a standard penetration test *N*-value between 10 and 30 blows per 1 foot (0.3 meter).

Loose (Class 6). Those materials having a standard penetration test *N*-value less than 10 blows per 1 foot (0.3 meter).

GRANULAR SOILS. These materials are coarse-grained soils consisting of gravel and/or sand with appreciable amounts of fines, and gravel (GM, GC, SW, SP, SM, and SC). The density of granular materials shall be determined in accordance with the following:

Dense (Class 3a). Those materials having a standard penetration test *N*-value greater than 30 blows per 1 foot (0.3 meter).

Medium (Class 3b). Those materials having a standard penetration test *N*-value between 10 and 30 blows per 1 foot (0.3 meter).

Loose (Class 6). Those materials having standard penetration test *N*-value less than 10 blows per 1 foot (0.3 meter).

CLAYS. In the absence of sufficient laboratory data, the consistency of clay materials (SC, CL, and CH) shall be determined in accordance with the following:

Hard (Class 4a). Clay requiring picking for removal, a fresh sample of which cannot be molded by pressure of the fingers, or having an unconfined compressive strength in excess of 4 TSF (383 kPa), or standard penetration test *N*-values greater than 30 blows per 1 foot (0.3 meter).

TABLE 1804.1
ALLOWABLE BEARING PRESSURES

CLASS OF MATERIALS (Notes 1 and 3)	MAXIMUM ALLOWABLE FOUNDATION PRESSURE (TSF)	MAXIMUM ALLOWABLE FOUNDATION PRESSURE (kPa)
1. Bedrock (Notes 2 and 7)		
1a Hard sound rock—gneiss, diabase, schist	60	5,746
1b Medium hard rock—marble, serpentine	40	3,830
1c Intermediate rock—shale, sandstone	20	1,915
1d Soft rock—weathered rock	8	766
2. Sandy gravel and gravel (GW, GP) (Notes 3, 4, 8, and 9)		
2a Dense	10	958
2b Medium	6	575
3. Granular soils (GC, GM, SW, SP,SM, and SC) (Notes 4, 5, 8, and 9)		
3a Dense	6	575
3b Medium	3	287
4. Clays (SC, CL, and CH) (Notes 4, 6, 8, and 9)		
4a Hard	5	479
4b Stiff	3	287
4c Medium	2	192
5. Silts and silty soils (ML and MH) (Notes 4, 8, and 9)		
5a Dense	3	287
5b Medium	1.5	144
6. Organic silts, organic clays, peats, soft clays, loose granular soils and varved silts	See 1804.2.1	See 1804.2.1
7. Controlled and uncontrolled fills	See 1804.2.2 or 1804.2.3	See 1804.2.2 or 1804.2.3

Notes:

1. Where there is doubt as to the applicable classification of a soil stratum, the allowable bearing pressure applicable to the lower class of material to which the given stratum might conform shall apply.
2. The tabulated values of allowable bearing pressures apply only for massive rocks or for sedimentary or foliated rocks, where the strata are level or nearly so, and then only if the area has ample lateral support. Tilted strata and their relation to nearby slopes or excavations shall receive special consideration. The tabulated values for Class 1a materials (hard sound rock) may be increased by 25 percent provided the geotechnical engineer performs additional tests and/or analyses substantiating the increase.
3. For intermediate conditions, values of allowable bearing pressure shall be estimated by interpolation between indicated extremes.
4. Footing embedment in soils shall be in accordance with Section 1805.2 and the width of the loaded area not less than 2 feet (610 mm), unless analysis demonstrates that the proposed construction will have a minimum factor of safety of 2.0 against shear failure of the soil.
5. Estimates of settlements shall govern the allowable bearing value, subject to the maximums given in this table, and as provided in Section 1804.2.
6. The bearing capacity of clay soils shall be established on the basis of the strength of such soils as determined by field or laboratory tests and shall provide a factor of safety against failure of the soil of not less than 2.0 computed on the basis of a recognized procedure of soils analysis, shall account for probable settlements of the building and shall not exceed the tabulated maximum values.
7. Increases in allowable bearing pressure due to embedment of the foundation. The allowable bearing values for intermediate to hard rock shall apply where the loaded area is on the surface of sound rock. Where the loaded area is below the adjacent rock surface and is fully confined by the adjacent rock mass and provided that the rock mass has not been shattered by blasting or otherwise is or has been rendered unsound, these values may be increased 10 percent of the base value for each 1 foot (0.3 meters) of embedment below the surface of the adjacent rock surface in excess of 1 foot (0.3 meters), but shall not exceed 200 percent of the values.
8. The allowable bearing values for soils of Classes 2, 3, 4, and 5 determined in accordance with Notes three, four and five above, shall apply where the loaded area is embedded 4 feet (1219 mm) or less in the bearing stratum. Where the loaded area is embedded more than 4 feet (1219 mm) below the adjacent surface of the bearing stratum, and is fully confined by the weight of the adjacent soil, these values may be increased 5 percent of the base value for each 1-foot (305 mm) additional embedment, but shall not exceed twice the values. Increases in allowable bearing pressure due to embedment shall not apply to soft rock, clays, silts and soils of Classes 6 and 7.
9. The allowable bearing values for soils of Classes 2, 3, 4, and 5 determined in accordance with this table and the notes thereto, may be increased up to one-third where the density of the bearing stratum below the bottom of the footings increases with depth and is not underlain by materials of a lower allowable bearing pressure. Such allowable bearing values shall be demonstrated by a recognized means of analysis that the probable settlement of the foundation due to compression, and/or consolidation does not exceed acceptable limits for the proposed building.
10. The maximum toe pressure for eccentrically loaded footings may exceed the allowable bearing value by up to 25 percent if it is demonstrated that the heel of the footing is not subjected to tension.

Stiff (Class 4b). Clay that can be removed by spading, a fresh sample of which requires substantial pressure of the fingers to create an indentation, or having an unconfined compressive strength between 1 TSF (96 kPa) and 4 TSF (383 kPa), or standard penetration test *N*-values between 8 and 30 blows per 1 foot (0.3 meter).

Medium (Class 4c). Clay that can be removed by spading, a fresh sample of which can be molded by substantial pressure of the fingers, or having an unconfined compressive strength between 0.5 TSF (48 kPa) and 1 TSF (96 kPa), or standard penetration test *N*-values between 4 and 8 blows per 1 foot (0.3 meter).

Soft (Class 6). Clay, a fresh sample of which can be molded with slight pressure of the fingers, or having an unconfined compressive strength less than 0.5 TSF (48 kPa), or standard penetration test *N*-values less than 4 blows per 1 foot (0.3 meter).

SILTS AND CLAYEY SILTS. In the absence of sufficient laboratory data, the consistency of silt materials (ML and MH) shall be determined in accordance with the following:

Dense (Class 5a). Silt with a standard penetration test *N*-values greater than 30 blows per 1 foot (0.3 meter).

Medium (Class 5b). Silt with a standard penetration test *N*-values between 10 and 30 blows per 1 foot (0.3 meter).

Loose (Class 6). Silt with a standard penetration test *N*-values less than 10 blows per 1 foot (0.3 meters).

Organic silts, organic clays, peats, soft clays, loose granular soils and varved silts. The allowable bearing pressure shall be determined independently of Table 1804.1 subject to the following:

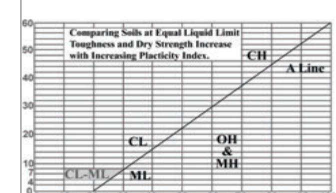
1. For varved silts, the soil bearing pressure produced by the proposed building shall not exceed 2 tons per square foot (192 kPa), except that for desiccated or over consolidated soils, higher bearing pressures are allowed subject to approval by the commissioner.
2. For organic silts or clays, soft clays, or for loose granular soils, the engineer responsible for the investigation shall establish the allowable soil bearing pressure based upon the soil's specific engineering properties. This may require that the soils be preconsolidated, artificially treated, or compacted.
3. A report prepared, signed and sealed by the engineer is required to be filed to substantiate the design soil pressures to be used on soil materials and it shall contain, at a minimum:
   - 3.1. Sufficient laboratory test data on the compressible material to indicate the soil strength and the preconsolidation pressure, coefficient of consolidation, coefficient of compressibility, permeability, secondary compression characteristics, and Atterberg limits.
   - 3.2. Where the design contemplates improvement of the natural bearing capacity and/or reduction in settlements by virtue of preloading: cross sections showing the amount of fill and surcharge to be placed, design details showing the required time for surcharging, and computations showing the amount of settlement to be expected during surcharging and the estimated amount and rate of settlement expected to occur after the structure has been completed, including the influence of dead and live loads of the structure.
   - 3.3. A detailed analysis showing that the anticipated future settlement will not adversely affect the performance of the structure.
   - 3.4. Where strip drains, sand drains, or stone columns are to be used, computations showing the diameter, spacing, and anticipated method of installation of such drains.
   - 3.5. Records of settlement plate elevations and pore pressure readings, before, during, and after surcharging.

1804.2.2 Controlled fills. Fills shall be considered as satisfactory bearing material of the applicable class when placed in accordance with the following procedures and subject to the special inspection provisions of Chapter 17:

1. Area to be filled shall be stripped of all organic materials, rubbish and debris.
2. Fill shall not be placed when frozen or on frozen or saturated subgrade.
3. The engineer, or the engineer's representative, shall approve the subgrade prior to fill placement.
4. Fill material shall consist of well graded sand, gravel, crushed rock, recycled concrete aggregate, or a mixture of these, or equivalent materials with a maximum of 10 percent passing the #200 sieve, as determined from the percent passing the #4 sieve.
5. Fill shall be placed and compacted in lifts, not exceeding 12 inches (305 mm), at its optimum moisture content, plus or minus 2 percent, and to not less than a density of 95 percent of the optimum density as determined by ASTM D 1557.
6. Fill density shall be verified by in-place tests made on each lift.
7. The allowable bearing value of controlled fill shall be limited to 3 tons per square foot (383 kPa) providing the underlying soil is not weaker than the controlled fill.

TABLE 1804.2 UNIFIED SOIL CLASSIFICATION (Including Identification and Description)

MAJOR DIVISIONS 1	MAJOR DIVISIONS 2		GROUP SYMBOLS 3	TYPICAL NAMES 4	FIELD IDENTIFICATION PROCEDURES (EXCLUDING PARTICLES LARGER THAN 3 IN. AND BASING FRACTIONS ON ESTIMATED WEIGHTS) 5			INFORMATION REQUIRED FOR DESCRIBING SOILS 6	LABORATORY CLASSIFICATION CRITERIA 7	
Coarse-grained Soils More than half of material is larger than No. 200 sieve size. The No. 200 sieve size is about the smallest visible to the naked eye.	Gravels More than half of coarse fraction is larger than No. 4 sieve size. (For visual classification. the $^1/_4$-in. size may be used as equivalent to the No. 4 sieve size.)	Clean Gravels (Little or no fines)	GW	Well-graded gravels, gravel-sand mixture, little or no fines.	Wide range in grain size and substantial amounts of all intermediate particle sizes.			For undisturbed soils add information on stratification, degree of compactness, cementation, moisture condition, and drainage characteristics.	Use grain-size curve in identifying the fractions as given under field identification. Determine percentage of gravel and sand from grain-size curve. Depending on percentage of fine (fraction smaller than No. 200 sieve size) coarse-grained soils are classified as follows: Less than 5% GW, GP, SW, SP. More than 12% GM, GC, SM, SC. 5% to 12% Borderline cases requiring use of dual symbols.	$C_u = \frac{D_{60}}{D_{10}}$ Greater than 4; $C_e = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3
			GP	Poorly graded gravels or gravel-sand mixture, little or no fines.	Predominantly one size or a range of sizes with some intermediate sizes missing.					Not meeting all gradation requirements for GW
		Gravels with Fines (Appreciable amount of fines)	GM	Silty gravels, gravel and silt mixtures.	Nonplastic fines or fines with low plasticity (for identification procedures see ML below).			Give typical name; indicate approximate percentages of sand and gravel, maximum size; angularity, surface condition, and hardness of the coarse grains; local or geologic name and other pertinent descriptive information; and symbol in parentheses.		Atterberg limits below "A" line or P1 less than 4 — Above "A" line with P1 between 4 and 7 are borderline cases requiring use of dual symbols.
			GC	Clayey gravels, gravel and clay mixtures.	Plastic fines (for identification procedures see CL below).					Atterberg limits above "A" line with P1 greater than 7
	Sands More than half of coarse fraction is smaller than No .4 sieve size.	Clean Sand (Little or no fines)	SW	Well-graded sands, gravelly sands, little or no fines.	Wide range in grain size and substantial amounts of all intermediate particle sizes.					$Cu = \frac{D_{60}}{D_{10}}$ Greater than 6; $C_e = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3
			SP	Poorly graded sands or gravelly sands, little or no fines.	Predominantly one size or a range of sizes with some intermediate sizes missing.			Example: Silty sand, gravelly; about 20% hard, angular gravel particles $^1/_2$-in. maximum size; rounded and subangular sand grains, coarse to fine; about 15% nonplastic fines with low dry strength; well compacted and moist in place; alluvial sand; (SM).		Not meeting all gradation requirements for SW
		Sands with Fines (Appreciable amount of fines)	SM	Silty sands, sand-silt mixtures.	Nonplastic fines or fines with low plasticity (for identification procedures see ML below).					Atterberg limits above "A" line or P1 less than 4 — Limits plotting in hatched zone with P1 between 4 and 7 are borderline cases requiring use of dual symbols.
			SC	Clayey sands, sand-clay mixtures.	Plastic fines (for identification procedures see CL below).					Atterberg limits above "A" line with Pl greater than 7
Fine-grained Soils More than half of material is smaller than No. 200 sieve size.					Identification Procedure on Fraction Smaller than No. 40 Sieve Size.					
					Dry Strength (Crushing Characteristics)	Dilatancy (Reaction to shaking)	Toughness (Consistency near PL)			
	Silts and Clays Liquid limit is less than 50		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.	None to slight	Quick to slow	None	For undisturbed soils add information on structure, stratification, consistency in undisturbed and remolded states, moisture and drainage conditions		LIQUID LIMIT PLASTICITY CHART For laboratory classification of fine-grained soils
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.	Medium to high	None to very slow	Medium			
	Silts and Clays Liquid limit is greater than 50		OL	Organic silts and organic silty clays of low plasticity.	Slight to medium	Slow	Slight	Give typical name; indicate degree and character of plasticity; amount and maximum size of coarse grains; color in wet condition; odor, if any; local or geologic name and other pertinent descriptive information; and symbol in parentheses.		
			MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.	Slight to medium	Slow to none	Slight to medium			
			CH	Inorganic clays of high plasticity, fat clays.	High to very high	None	High			
			OH	Organic clays of medium to high plasticity, organic silts.	Medium to high	None to very slow	Slight to medium	Example: Clayey silt, brown; slightly plastic; small percentage of fine sand; numerous vertical root holes; firm and dry in place; loess; (ML)		
Highly Organic Soils			Pt	Peat and other highly organic soils.	Readily identified by color, odor, spongy feel and frequently by fibrous texture					



1. Boundary classifications: Soils possessing characteristics of two groups are designed by combinations of group symbols. For example GM-GC, well-graded gravel-sand mixture with clay binder.
2. All sieve sizes on this chart are U.S. standard.
3. Adopted by Corps of Engineers and Bureau of Reclamation, January 1952.

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1804.2.3 Uncontrolled fills. Fills other than controlled fill may be considered as satisfactory bearing material of applicable class, subject to the following:

1. The soil within the building area shall be explored using test pits at every column. All test pits shall extend to depths equal to the smaller width of the footing and at least one test pit shall penetrate at least 8 feet (2438 mm) below the level of the bottom of the proposed footings. All test pits shall be backfilled with properly compacted fill. Borings may be used in lieu of test pits, provided that continuous samples of at least 3 inches (76 mm) in diameter are recovered.
2. The building area shall be additionally explored using one standard boring for every 2,500 square foot (232.3 m^2) of building footprint area. These borings shall be carried to a depth sufficient to penetrate into natural ground, but not less than 20 feet (6096 mm) below grade.
3. The fill shall be composed of material that is free of voids and free of extensive inclusions of mud, organic materials, such as paper, wood, garbage, cans, or metallic objects and debris.
4. The allowable soil bearing pressure on satisfactory uncontrolled fill material shall not exceed 2 tons per square foot (192 kPa). One- and two-family dwellings may be founded on satisfactory uncontrolled fill provided the dwelling site has been explored using at least one test pit, penetrating at least 8 feet (2438 mm) below the level of the bottom of the proposed footings, and that the fill has been found to be composed of material that is free of voids and generally free of mud and‡ organic materials, such as paper, garbage, cans, or metallic objects, and debris. Test pits shall be backfilled with properly compacted fill.

1804.2.4 Artificially treated soils. Nominally unsatisfactory soil materials that are artificially compacted, cemented, or preconsolidated may be used for the support of buildings, and nominally satisfactory soil materials that are similarly treated may be used to resist soil bearing pressures in excess of those indicated in Table 1804.1. The engineer shall develop treatment plans and procedures and post-treatment performance and testing requirements and submit such plans, procedures, and requirements to the commissioner for approval. After treatment, a sufficient amount of sampling and/or in-situ tests shall be performed in the treated soil to demonstrate the efficacy of the treatment for the increased bearing pressure.

1804.3 Reserved.

SECTION BC 1805
FOOTINGS AND FOUNDATIONS

1805.1 General. Footings and foundations shall be designed and constructed in accordance with Sections 1805.1 through 1805.9. Footings and foundations shall be built on undisturbed soil, compacted fill material or CLSM. Compacted fill material shall be placed in accordance with Section 1803.5. CLSM shall be placed in accordance with Section 1803.6.

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

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

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

1. Extending a minimum of 4 feet (1219 mm) below grade;
2. Constructing in accordance with ASCE-32; or
3. Erecting on solid rock.

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

1. Classified in Structural Occupancy Category I (see Table 1604.5);
2. Area of 400 square feet (37 m^2) or less; and
3. Eave height of 10 feet (3048 mm) or less.

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

1805.3 Foundations at different levels. Where footings are supported at different levels, or are at different levels from the footings of adjacent structures, the influence of the pressures under the higher footings on the stability of the lower footings shall be considered in the design. The design shall consider the requirements for lateral support of the material supporting the higher footing, the additional load imposed on the lower footings, and assessment of the effects of dragdown on adjacent pile-supported buildings.

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

1805.4.1 Design. Footings shall be so designed that the allowable bearing capacity of the soil is not exceeded, and that differential settlement is minimized. The minimum width of footings shall be 18 inches (457 mm).

1805.4.1.1 Design loads. Footings shall be designed for the most unfavorable effects due to the combinations of loads specified in Sections 1605.3 and‡ 1801.2. The dead load shall include the weight of foundations, footings and overlying fill. Reduced live loads, as specified in Section 1607.9, are permitted to be used in the design of footings.

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

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

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

1805.4.2.2 Footing seismic ties. Where a structure is assigned to Seismic Design Category D 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 SDS divided by 10 unless it is demonstrated that equivalent restraint is provided by reinforced concrete beams within slabs on grade or reinforced concrete slabs on grade.

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

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

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

1805.4.2.5 Protection of concrete. No foundation shall be placed on frozen soil. No foundation shall be placed in freezing weather unless provision is made to maintain the underlying soil free of frost. Concrete footings shall be protected from freezing during depositing and for a period of not less than five days thereafter. Water shall not be allowed to flow through the deposited concrete.

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

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

1805.4.5 Timber footings. Refer to Chapter 23.

1805.4.6 Wood foundations. Refer to Chapter 23.

1805.5 Foundation walls. Concrete and masonry foundation walls shall be designed in accordance with Chapter 19 or 21, respectively.

1805.5.1 Foundation wall thickness. The minimum thickness of concrete and masonry foundation walls shall comply with Section 1805.5.1.1.

1805.5.1.1 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.

1805.5.2 Foundation wall drainage. Foundation walls shall be designed to support the weight of the full hydrostatic pressure of undrained backfill unless a drainage system is installed in accordance with Sections 1807.4.2 and 1807.4.3. Foundation walls shall be designed to support the earth pressures due to the backfill including full hydrostatic pressure of undrained backfill unless a drainage system is installed in accordance with Sections 1807.4.2 and 1807.4.3.

1805.5.3 Pier and curtain wall foundations. Except in Seismic Design Category D, pier and curtain wall foundations are permitted to be used to support light-frame construction not more than two stories in height, provided the following requirements are met:

1. All load-bearing walls shall be placed on continuous concrete footings bonded integrally with the exterior wall footings.
2. The minimum actual thickness of a load-bearing masonry wall shall not be less than 4 inches (102 mm) nominal or $3^5/_8$ inches (92 mm) actual thickness, and shall be bonded integrally with piers spaced 6 feet (1829 mm) on center (o.c.).
3. Piers shall be constructed in accordance with Chapter 21 and the following:
   3.1. The unsupported height of the masonry piers shall not exceed 10 times their least dimension.
   3.2. Where structural clay tile or hollow concrete masonry units are used for piers supporting beams and girders, the cellular spaces shall be filled solidly with concrete or Type M or S mortar.

   Exception: Unfilled hollow piers are permitted where the unsupported height of the pier is not more than four times its least dimension.
4. The maximum height of a 4-inch (102mm) load-bearing masonry foundation wall supporting wood frame walls and floors shall not be more than 4 feet (1219 mm) in height.
5. The unbalanced fill for 4-inch (102 mm) foundation walls shall not exceed 24 inches (610 mm) for solid masonry, nor 12 inches (305mm) for hollow masonry.

1805.6 Reserved.

1805.7 Reserved.

1805.8 Reserved.

1805.9 Seismic requirements. See Section 1910 for additional requirements for footings and foundations of structures assigned to Seismic Design Category C or D.

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

Exceptions:

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

SECTION BC 1806 RETAINING WALLS AND OTHER RETAINING STRUCTURES

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

1806.2 Seismic loads on retaining walls and other retaining structures. Seismic foundation design shall comply with the requirements of ASCE 7, Section 9.7. The geotechnical analysis and design shall take into consideration the yielding characteristics of the retaining walls or other retaining structures.

SECTION BC 1807 DAMPPROOFING AND WATERPROOFING

1807.1 Where required. Walls or portions thereof that retain earth and enclose interior spaces and floors below grade shall be waterproofed and dampproofed in accordance with this section, with the exception of those spaces containing occupancy groups other than residential and institutional where such omission is not detrimental to the building or occupancy. Ventilation for crawl spaces shall comply with Section 1203.3.

1807.1.1 Story above grade. Where a basement or cellar is considered a story above grade and the finished ground level adjacent to the basement or cellar wall is below the basement or cellar 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 or cellar floor is below ground level. The provisions of Sections 1807.3 and 1807.4.1 shall not apply in this case.

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

1807.1.2.1 Flood hazard areas. For buildings and structures in areas of special flood hazard, the finished ground level of an under-floor space such as a crawl space shall comply with Appendix G.

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 operate and the rated capacity of the disposal area of the system.

1807.2 Dampproofing required. Where hydrostatic pressure will not occur as determined by Section 1802, 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 investigation required by Section 1802 indicates that a hydro-static 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 water-tight utilizing approved methods and materials.

1807.4 Subsoil drainage system. Where a hydrostatic pressure is to be controlled by a subsoil drainage system, dampproofing, a floor base course, and subdrains around the foundation perimeter and under the floor shall be provided. 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 or cellars, 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 that consists of gravel or crushed stone containing not more than 10-percent material that passes through a No. 4 (4.75 mm) sieve shall be placed around the perimeter of a foundation. 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 *New York City Plumbing Code*.

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

1807.5 In-situ walls. Applied waterproofing or dampproofing need not to be applied to slurry walls or walls constructed below the water table. In-situ walls shall be constructed with water-tight joints between individual elements and sealed by grouting to achieve a uniform water-tight surface. External drains and waterproofing are not required. In-situ walls constructed using tangent pile or secant pile may require waterproofing systems because their joints cannot be sealed. For both instances, an under floor seepage collection system or pressure slab shall be provided.

SECTION BC 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.

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

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

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

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

Augered-cast-in-place piles. Augered-cast-in-place piles are constructed by pumping grout into an augered hole during the withdrawal of the auger. The pile is reinforced with a single reinforcing bar, a reinforcing steel cage or a structural steel section.

Caisson piles. Steel cased piles are constructed by driving a steel shell to a water-tight seal at the top of rock and drilling of an uncased socket within the rock. The shell and socket is filled with a steel core section and concrete or grout.

Compacted concrete piles. Compacted concrete piles are constructed by filling a shaft with low-strength concrete as the casing is withdrawn.

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

Driven uncased piles. Driven uncased piles are constructed by driving a steel shell into the soil to shore an unexcavated hole that is later filled with concrete. The steel casing is lifted out of the hole during the deposition of the concrete. These piles are allowed only if the concrete is placed under pressure.

Enlarged base piles. Enlarged base piles are cast-in-place concrete piles constructed with a base that is larger than the diameter of the remainder of the pile. The enlarged base is designed to increase the load-bearing area of the pile in end bearing. Enlarged base piles include piles installed by driving a precast concrete tip or by compacting concrete into the base of the pile to form an enlarged base.

H-piles. Steel H-piles are constructed by driving a steel H-shaped section into the ground.

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

Jacked piles. Steel pipe piles installed by hydraulically jacking the pile into the ground against a dead-weight reaction. Piles installed by other static forces shall be considered in this category.

Micropiles/minipiles. Small-diameter drilled steel cased piles, driven uncased piles or caisson piles.

Open-end pipe pile. Steel pipe driven open ended that may or may not be filled with concrete or soil.

1808.2 Piers and piles—general requirements.

1808.2.1 Design. Piles are permitted to be designed in accordance with provisions for piers in Section 1808 and Sections 1812.3 through 1812.11 where either of the following conditions exists, subject to the approval of the commissioner:

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

1808.2.2 General. Pier and pile foundations shall be designed and installed on the basis of a foundation investigation as defined in Section 1802.

The investigation and report provisions of Section 1802 shall be expanded to include, but not be limited to, the following:

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

Pier and pile foundations shall be designed and installed under the direct control of an engineer knowledgeable in the field of geotechnical engineering and pier and pile foundations and shall be subject to special inspection performed under the direct control of such engineer. The engineer shall certify to the commissioner that the piers or piles as installed satisfy the design criteria.

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

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

1808.2.5 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 commissioner.

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.

1808.2.6 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.

1808.2.6.1 Minimum spacing of piles. Piles shall be spaced to meet the following requirements:

1. Spacing of' piles shall provide for adequate distribution of' the load on the pile group to the supporting soil. In no case shall the minimum center-to-center spacing of piles be less than 24 inches (610 mm) nor less than the following for the specific types of piling indicated in this chapter.
2. Minimum spacing between enlarged base concrete piles shall be 4 feet 6 inches (1372 mm), center to center except that where the shafts of such piles are cased for their full length, this spacing may be reduced to 3 feet 6 inches (1067 mm). Where a question exists as to possible damage to adjacent previously driven piles, these minimums shall be increased. Minimum center-to-center spacing of piles at the bearing level shall be at least two and one half times the outside diameter of the shell.
3. Unless special measures are taken to assure that piles will penetrate sufficiently to meet the requirements of Section‡1808.2.8 without interfering with or intersecting each other, the minimum center-to-center spacing of piles shall be twice the average diameter of the butt for round piles, one and three-quarters times the diagonal for rectangular piles; or, for taper piles, twice the diameter at a level two-thirds of the pile length measured up from the tip.
4. In cases of practical difficulty, the spacing of new piles from existing piles under an adjacent building may be less than the above values provided that the requirements relating to minimum embedment and pile interference are satisfied and that the soil under the proposed and existing buildings is not overloaded by the closer pile grouping.

1808.2.6.2 Piles located near a lot line. Piles located near a lot line shall be designed on the assumption that the adjacent lot will be excavated to a depth of 10 feet (3048 mm) below the nearest legally established curb level. Where such excavation would reduce the embedded length of the pile, the portion of the pile exposed shall be deemed to provide no lateral or vertical support, and the load-carrying determination shall be made the resistance offered by the soil that is subject to potential excavation has been discounted.

1808.2.7 Splices. Splices shall be constructed so as to provide and maintain true alignment and position of the component parts of the pile during installation and subsequent thereto and shall be of adequate strength to transmit the vertical and lateral loads (including tensions) and the moments occurring in the pile section at the location of the splice without exceeding the allowable stresses for such materials as established in Section 1809. In all cases splices shall develop at least 50 percent of the capacity of the pile in bending. In all cases pile splices situated in the upper 10 feet (3048 mm) of the pile shall be capable of resisting (at allowable working stresses) the applied moments and shears or the moment and shear that would result from an assumed eccentricity of the pile load of 3 inches (76 mm), whichever is greater. For piles located near a lot line, the embedded length of such piles shall be determined on the basis that the adjacent site will be excavated to a depth of 10 feet (3048 mm) below the nearest established curb level as required in Section 1808.2.6.2.

1808.2.8 Allowable pier or pile loads. Allowable pier or pile loads shall be determined in accordance with Sections 1808.2.8.1 through 1808.2.8.9.

1808.2.8.1 Determination of allowable loads. The allowable axial and lateral loads on piers or piles shall be determined by load tests or a recognized method of analysis. The allowable load shall be determined by a licensed engineer experienced in geotechnical engineering and shall be approved by the commissioner.

The allowable axial load on a pile shall be the least value permitted by consideration of the following factors (for battered piles, the axial load shall be computed from the resultant of all vertical loads and lateral forces occurring simultaneously):

1. The capacity of the pile as a structural member.
2. The allowable bearing pressure on soil strata underlying the pile tips.
3. The resistance to penetration of the piles, including resistance to driving, resistance to jacking, the rate of penetration, or other equivalent criteria.
4. The capacity as indicated by load test, where load tests are required.

5. The maximum loads prescribed in Section 1808.2.8.1.3.

1808.2.8.1.1 Bearing capacity. The allowable pile load shall be limited by the provision that the pressures in materials at and below the pile tips, produced by the loads on individual piles and by the aggregate of all piles in a group or foundation, shall not exceed the allowable bearing values established in Table 1804.1. The transfer of load from piles to soil shall be determined by a recognized method of analysis. As an alternative, for purposes of this section, piles or pile groups may be assumed to transfer their loads to the underlying materials by spreading the load uniformly at an angle of 60 degrees (1.04 rad)‡ with the horizontal, starting at a polygon circumscribing the piles, located as follows:

1. For piles embedded entirely in decomposed rock or granular soils, or in controlled fill materials, the polygon shall be circumscribed at a level located two-thirds of the embedded length of the pile, measured up from the tip.
2. For piles penetrating through silts and clays into bearing in decomposed rock or granular soils, the polygon shall be circumscribed at the bottom of the strata of silts or clays.

1808.2.8.1.2 Bearing stratum. The plans for the proposed work shall establish, in accordance with the requirements relating to allowable bearing pressure, the bearing stratum to which the piles in the various sections of the building must penetrate and the approximate elevations of the top of such bearing stratum. Where penetration of a given distance into the bearing strata is required for adequate distribution of the loads, such penetration shall be shown on the plans. The indicated elevations of the top of the bearing strata shall be modified by such additional data as may be obtained during construction. All piles shall penetrate to or into the designated bearing stratum.

1808.2.8.1.3 Maximum loads. Except as permitted by the provisions of Section 1808.2.8.3.1.6, the maximum allowable pile load shall not exceed the values specified in Table 1808.2.8.1.3.

1808.2.8.1.4 Minimum pile penetrations. Piles shall penetrate the minimum distance required to develop the required load capacity of the pile as established by the required penetration resistance and load tests as applicable.

1808.2.8.1.5 Capacity as indicated by resistance to penetration. Where subsurface investigation and general experience in the area indicate that the soil that must be penetrated by the pile consists of glacial deposits containing boulders, or fills containing riprap, excavated detritus, masonry, concrete or other obstructions in sufficient numbers to present a hazard to the installation of the piles, the selection of type of pile and penetration criteria shall be subject to the approval of the commissioner but in no case shall the minimum penetration resistance be less than that stated in Tables 1808.2.8.1.5A and 1802.8.2.8.1.5B.

1808.2.8.1.5.1 Piles installed by use of steam-powered, air-powered, diesel-powered or hydraulic impact hammers.

1. The minimum required driving resistance and the requirements for hammer energies for various types and capacities of piles are given in Tables 1808.2.8.1.5A and 1808.2.8.1.5B. To obtain the required total driving resistance, the indicated driving resistances shall be added to any driving resistance experienced by the pile during installation, but which will be dissipated

TABLE 1808.2.8.1.3
MAXIMUM ALLOWABLE PILE LOADS

TYPE OF PILE	MAXIMUM ALLOWABLE PILE LOAD (TONS)	
Caisson Piles	**No upper limit**	
Open-end pipe (or tube) piles bearing on rock of Class 1a, 1b, or 1c	18-in O.D. and greater 14-in to 18-in O.D. 12-in to 14-in 10-in to 12-in 8-in to 10-in	250 200 150 100 60
Closed-end pipe (or tube) piles, H-piles, cast-in-place concrete, enlarged base piles, and precast concrete piles bearing on rock of Class 1a, 1b, or 1c	150	
Piles (other than timber piles) bearing on soft rock of Class 1d	80	
Piles (other than timber piles) that receive their principal support other than by direct bearing on rock of Class 1a through 1d	75	
Timber piles bearing on rock of Class 1a through 1d	25	
Timber piles bearing in suitable soils	40 tons maximum permissible with load test, 30 tons maximum without load test.	

For SI: 1 ton = 1000 kg.

with time (resistance exerted by nonbearing materials or by materials which are to be excavated). For the purposes of this section, the resistance exerted by nonbearing materials may be approximated as the resistance to penetration of the pile recorded when the pile has penetrated to the bottom of the lowest stratum of nominally unsatisfactory bearing material (Class 6 and uncontrolled fills, but not controlled fill) or to the bottom of the lowest stratum of soft or loose deposits of soils of Class 4 or 5 but only where such strata are completely penetrated by the pile.

2. Alternate for similitude method—The requirement for installation of piling to the penetration resistances given in Tables 1808.2.8.1.5A and 1808.2.8.1.5B may be

TABLE 1808.2.8.1.5A
MINIMUM DRIVING RESISTANCE AND MINIMUM HAMMER ENERGY FOR STEEL H-PILES, PIPE PILES, PRECAST AND CAST-IN-PLACE CONCRETE PILES AND COMPOSITE PILES (other than timber)

MINIMUM DRIVING RESISTANCE [a, c, d, e]				
Pile Capacity (tons)	**Hammer Energy[b] (ft. lbs.)**	**Friction Piles (blows/ft.)**	**Piles Bearing on Soft Rock (Class 1d) (blows/ft.)**	**Piles Bearing on Rock (Class 1a, 1b, and 1c)**
Up to 20	15,000	19	48	5 Blows per 1/4 inch (Minimum hammer energy of 15,000 ft. lbs.)
	19,000	15	27	
	24,000	11	16	
30	15,000	30	72	
	19,000	23	40	
	24,000	18	26	
40	15,000	44	96	
	19,000	32	53	
	24,000	24	34	
50	15,000	72	120	
	19,000	49	80	
	24,000	35	60	
	32,000	24	40	
60	15,000	96	240	
	19,000	63	150	
	24,000	44	100	
	32,000	30	50	
70 & 80	19,000		5 Blows per 1/4 inch (Minimum hammer energy of 19,000 ft. lbs.)	
	24,000			
	32,000			

For SI: 1 foot = 304.8 mm, 1 ton = 1000 kg.

a. Final driving resistance shall be the sum of tabulated values plus resistance exerted by nonbearing materials. The driving resistance of nonbearing materials shall be taken as the resistance experienced by the pile during driving, but which will be dissipated with time and may be approximated as described in Section 1808.2.8.1.5.1.

b. The hammer energy indicated is the rated energy.

c. Sustained driving resistance. Where piles are to bear in soft rock, the minimum driving resistance shall be maintained for the last 6 inches, unless a higher sustained driving resistance requirement is established by load test. Where piles are to bear in soil Classes 2 through 5, the minimum driving resistance shall be maintained for the last that inches unless load testing demonstrates a requirement for higher sustained driving resistance. No pile needs to be driven to a resistance that penetrates in blows per inch (blows per 25 mm) more than twice the resistance indicated in this table, nor beyond the point at which there is no measurable net penetration under the hammer blow.

d. The tabulated values assume that the ratio of total weight of pile to weight of striking part of the hammer does not exceed 3.5. If a larger ratio is to be used, or for other conditions for which no values are tabulated, the driving resistance shall be as approved by the commissioner.

e. For intermediate values of pile capacity, minimum requirements for driving resistance may be determined by straight line interpolation.

waived where the following six conditions are satisfied:

2.1. The piles bear on, or in, soil of Classes 2 through 5.

2.2. The stratigraphy, as defined by not less than one boring for every 1600 square feet (149 m^2) of building area, shall be reasonably uniform or divisible into areas of uniform conditions.

2.3. Regardless of pile type or capacity, one load test, as described in Section 1808.2.8.1.5.3, shall be conducted in each area of uniform conditions, but not less than two typical piles for the entire foundation installation of the building or group of buildings on the site, nor less than one pile for every 15,000 square feet (1394 m^2) of pile foundation area shall be load tested.

2.4. Except as permitted by the provisions of paragraph 2.6 below, all building piles within the area of influence of a given load-tested pile of satisfactory performance shall be installed to the same or greater driving resistance as the successful load-tested pile. The same or heavier equipment of the same type that was used to install the load-tested pile shall be used to install all other building piles, and the equipment shall be operated identically. Also, all other piles shall be of the same type, shape, external dimension, and equal or greater cross-section as the load-tested pile. All building piles within the area of influence represented by a given satisfactory load-tested pile shall bear in, or on the same bearing stratum as the load test pile.

2.5. A report by an engineer shall be submitted to the commissioner for review and approval establishing that the soil-bearing pressures do not exceed the values permitted by Table 1804.1 and that the probable differential settlements will not cause stress conditions in the building in excess of those permitted by the provisions of this code.

TABLE 1808.2.8.1.5B
MINIMUM DRIVING RESISTANCE AND HAMMER ENERGY FOR TIMBER PILES

PILE CAPACITY (TONS)	MINIMUM DRIVING RESISTANCE (BLOWS/IN.) TO BE ADDED TO DRIVING RESISTANCE EXERTED BY NONBEARING MATERIALS (NOTES 1,3,4)	HAMMER ENERGY (ft./lbs.) (Note 2)
Up to 20	Formula in Note 4 shall apply	7,500-12,000
Over 20 to 25		9,000-12,000
		14,000-16,000
Over 25 to 30		12,000-16,000 (single-acting hammers)
Greater than 30		15,000-20,000 (double-acting hammers)

For SI: 1 ton = 1000 kg, 1 inch = 25.4 mm.

Notes:

1. The driving resistance exerted by nonbearing materials is the resistance experienced by the pile during driving, but which will be dissipated with time and may be approximated as described in Section 1808.2.8.1.5.1.
2. The hammer energy indicated is the rated energy.
3. Sustained driving resistance. Where piles are to bear in soil classes, Soft rock, the minimum driving resistance shall be maintained for the last 6 inches, unless a higher sustained driving resistance requirement is established by load test. Where piles are to bear in soil Classes 2 through 5, the minimum driving resistance measured in blows per inch (blows per 25 mm) shall be maintained for the last 12 inches unless load testing demonstrates a requirement for higher sustained driving resistance. No pile need be driven to a resistance that penetrates in blows per inch (blows per 25 mm) more than twice the resistance indicated in this table nor beyond the point at which there is no measurable net penetration under the hammer blow.
4. The minimum driving resistance shall be determined by the following formula:

$$P = \frac{2W_hH}{(S + 0.1)} \quad \text{or} \quad P = \frac{2E}{(S+0.1)}$$

where:

P = Allowable pile load in pounds.

W_p = Weight of pile in pounds.

W_h = Weight of striking part of hammer in pounds.

H = Actual height of fall of striking part of hammer in feet.

E = Rated energy delivered by the hammer per blow in foot/lbs.

S = Penetration of pile per blow, in inches, after the pile has been driven to a depth where successive blows produce approximately equal net penetration.

The value W_p shall not exceed three times W_h.

2.6. Where the structure of the building or the spacing and length of the piling is such as to cause the building and its foundation to act as an essentially rigid body, the building piles may be driven to length and/or penetration into the bearing stratum without regard to penetration resistance, subject to the requirement of paragraph 2.5 above, relating to the submission and approval of a report.

1808.2.8.1.5.2 Piles installed by jacking or other static forces. The carrying capacity of a pile installed by jacking or other static forces shall be not more than 50 percent of the load or force used to install the pile to the required penetration, except for piles jacked into position for underpinning. The working load of each permanent underpinning pile shall not exceed the larger of the following values: two-thirds of the total jacking force used to obtain the required penetration if the load is held constant for 7 hours without measurable settlement; or one-half of the total jacking force at final penetration if the load is held for a period of 1 hour without measurable settlement. The jacking resistance used to determine the working load shall not include the resistance offered by nonbearing materials which will be dissipated with time.

1808.2.8.1.5.3 Piles installed by use of vibratory hammer. The capacity of piles installed by vibratory hammer shall not exceed the value established on the principle of similitude, as follows:

1. Comparison piles, as required by the provisions of Item 4 below, shall be installed using an impact hammer and driving resistances corresponding to the proposed pile capacities as determined in paragraph 3‡ below or to tip elevations and driving resistances as determined by the engineer.
2. For each comparison pile, an identical index pile shall be installed by use of the vibratory hammer at a location at least 4 feet (1219 mm), but not more than 6 feet (1829 mm), from each comparison pile. The index piles shall be installed to the same tip elevation as the comparison pile, except that where the comparison piles bear on rock, the index piles shall bear in or on similar material. All driving data for the index pile shall be recorded.
3. The index piles shall be load tested in accordance with the provisions of Item 4 of this section. Should the specified load test criteria indicate inadequate capacity of the index piles, steps 1‡, 2‡, and 3‡ shall be repeated using longer, larger or other types of piles.
4. All building piles within the area of influence of a given, satisfactorily tested index pile shall be installed to the same or lesser rate of penetration (inches per minute mm per minute) as the successful index pile. The same equipment that was used to install the index pile, identically operated in all aspects, shall be used to install the building piles. All building piles shall be of the same type size and shape as the index pile. All building piles within the area of influence as represented by a given satisfactorily tested index pile shall bear in, or on, the same bearing stratum as the index pile.

1808.2.8.2 Driving criteria. The allowable compressive load on steel and concrete piles, where determined solely by the application of an approved wave equation analyses, shall not exceed 40 tons (356 kN). The allowable compressive loads on timber piles, where determined soley by the wave equation analysis, shall not exceed 30 tons (267 kN)‡. For allowable loads greater than these values, the wave equation method of analysis may be used to establish initial driving criteria, but final driving criteria and the allowable load shall be verified by load tests. The delivered energy of the hammer to be used shall be the maximum consistent with the size, strength and weight of the driven piles. The use of a follower is permitted only with the approval of the engineer of record. The introduction of fresh hammer cushion or pile cushion material just prior to final penetration is not permitted.

1808.2.8.3 Load tests. Where design compressive loads per pier or pile are greater than 40 tons (356 kN), [30 tons (267 kN)‡ for timber piles] or where drilled or jacked piles are not installed in Class 1a, 1b or 1c material, or where final penetration is by a vibratory hammer, or where the design load for any pier or pile foundation is in doubt, piers or piles shall be load tested in accordance with this section. At least one pier or pile shall be test loaded in each area of uniform subsoil conditions.

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

1. Davisson Offset Limit.
2. Brinch-Hansen 90 Percent Criterion.
3. Chin-Konder Extrapolation.
4. Other methods approved by the commissioner.

1808.2.8.3.1.1 Additional load tests. Where required by the commissioner, additional piers or piles shall be load tested where necessary to establish the allowable capacity. The load capacity shall be determined by an engineer in accordance with Section 1808.2.8.3. For friction piles where the actual production pile lengths vary more than 25 percent from that of the test pile, the engineer shall require an investigation to determine the adequacy of the piles.

1808.2.8.3.1.2 Load test requirements. Before any load test is made, the proposed apparatus and structure to be used in making the load test shall be satisfactory to the commissioner. The test shall be made under the responsible engineer's surveillance. A complete record of such tests shall be filed with the commissioner. Areas of the foundation site within which the subsurface soil conditions are substantially similar in character shall be established by the engineer. For piles installed by impact hammer, one load test shall be conducted in each area of substantially similar conditions, but not less than one typical pile for the entire foundation installation of the building or group of buildings on the site occupying a total area of 5,000 square feet (465 m^2) or less; and not less than two load tests for a site having a footprint between 5,000 (465 m^2) and 30,000 (2787 m^2) square feet and one additional load test for each 20,000 (1860 m^2) square feet of added footprint area.

1808.2.8.3.1.3 Load test procedures. Load tests shall be conducted in accordance with ASTM D 1143 standard procedures and the following conditions:

1. Dial extensometer gages shall provide readings to the nearest 0.001 inch (0.025 mm). Electrical transducers may be used to make settlement observations provided that backup measurements are made utilizing dial extensometers as described herein at sufficient times to validate the transducer readings. The total test load shall remain in place until the rate of settlement does not exceed 0.012 inches (0.305 mm) over a time period of 12 hours. The total load shall be removed in decrements not exceeding 25 percent of the total load at 1 hour intervals or longer. In addition to observations required by ASTM D 1143, settlement observations shall be performed at $^1/_2$ minute, 1-minute, 2-minute and 4-minute intervals after application of each load increment, and 24 hours after the entire test load has been removed.
2. Any temporary supporting capacity that the soil might provide to the pile during a load test, but which would be dissipated with time, shall be eliminated by casing off or by other suitable means, such as increasing the total test load to account for such temporary capacity.

1808.2.8.3.1.4 Alternative test methods. Load test methods other than those described in Section 1808.2.8.3 may be used subject to the approval of the commissioner where three or more load tests are required. In such case, at least one alternative test shall be performed as a calibration on a static load tested pile or nearby pile driven to comparable resistance. No more than one-half the required number of load tests may be performed by alternative methods. Alternative tests shall be performed under the supervision of an engineer experienced in the methods used. The number of alternative tests shall be at least twice the number of replaced static load tests.

1808.2.8.3.1.5 Acceptance criteria. The allowable pile load shall be the lesser of the two values computed as follows:

1. Fifty-percent of the applied load causing a net settlement of the pile of not more than one 1/100 of 1 inch per ton (0.25 mm per 8.9 kN) of applied load. Net settlement in this paragraph is defined as gross settlement due to the total test load minus the rebound after removing 100 percent of the test load.
2. Fifty-percent of the applied load causing a net settlement of the pile of $^3/_4$ inch (19 mm). Net settlement in this paragraph is defined as the gross settlement due to the total test load less the amount of elastic shortening in the pile section due to total test load. The elastic shortening shall be calculated as if the pile is designed as an end-bearing pile or as a friction pile. Alternatively, the net settlement may be measured directly using a telltale or other suitable instrumentation.

1808.2.8.3.1.6 Substantiation of higher allowable loads. The pile capacities tabulated in Table 1808.2.8.1.3 may be exceeded where a higher value can be substantiated on the basis of load tests and analysis. The provisions of Sections‡ 1808.2.8.3.1.2 and 1808.2.8.3.1.3 shall be supplemented, as follows:

The final load increment shall remain in place for a total of not less than 24 hours; single test piles shall be subjected to cyclical loading or suitably instrumented with telltales and strain gauges so that the movements of the pile tip and butt may be independently determined and load transfer to the soil evaluated. A complete record demonstrating satisfactory performance of the test shall be submitted to the commissioner.

1808.2.8.4 Allowable frictional resistance. The assumed frictional resistance developed by any pier or uncased cast-in-place pile shall not exceed one-sixth of the bearing value of the soil material at minimum depth as set forth in Table 1804.1, up to a maximum of 500 psf (24 kPa), unless a greater value is allowed by the commissioner after a soil investigation as specified in Section 1802 is submitted. Frictional resistance and bearing resistance shall not be assumed to act simultaneously unless recommended by a soil investigation as specified in Section 1802.

1808.2.8.5 Uplift capacity. Where required by the design, the uplift capacity of a single pier or pile shall be determined in accordance with accepted engineering practice based on a minimum factor of safety of three or by uplift load tests conducted in accordance with ASTM D 3689. The maximum allowable uplift load shall not exceed the ultimate load capacity divided by a factor of safety of two. The design uplift capacities of a pile group shall not exceed the sum of the design uplift capacities of the individual piles in the group, nor the uplift capacity calculating the group action of the pile in accordance with accepted engineering practice where the calculated ultimate group capacity is divided by a safety factor of 2.5.

1808.2.8.6 Load-bearing capacity. Piers, individual piles and groups of piles shall develop ultimate load capacities of at least twice the design working loads in the designated load-bearing layers. Analysis shall show that no soil layer underlying the designated load-bearing layers causes the load-bearing capacity safety factor to be less than two and that settlements of strata within the influence of the pile groups and the average area load of the supported structure are tolerable for the structure.

1808.2.8.7 Bent piers or piles. The load-bearing capacity of piers or piles discovered to have a sharp or sweeping bend shall be determined by an approved method of analysis in accordance with accepted engineering practice or by load testing a representative pier or pile.

1808.2.8.8 Overloads on piers or piles. The maximum compressive load on any pier or pile due to mislocation shall not exceed 110 percent of the allowable design load. If the total load on any pile, so determined, is in excess of 110 percent of the allowable load-bearing capacity, correction shall be made by installing additional piles or by other methods of load distribution as required to reduce the maximum pile load to 110 percent of the capacity.

1808.2.8.9 More than one pile type, pile capacity or method of pile installation. In the conditions described below, the several parts of the building supported on the different types, capacities, or modes of piling shall be separated by suitable joints providing for differential movement, or analysis shall be prepared by the engineer, establishing to the satisfaction of the commissioner that the proposed construction is adequate and safe, and showing that the probable settlements and differential settlements to be expected will be tolerable to the structure and not result in instability of the building. The load test requirements of Sections 1808.2.8.3 shall apply separately and distinctly to each different type or capacity of piling, method of installation, or type or capacity of equipment used, except where analysis of the probable, comparative behavior of the different types or capacities of the piles or the methods of installation indicates that data on one type or capacity of pile permit a reliable extrapolation of the probable behavior of the piles of other types and capacities. The requirements of this section apply to the following proposed conditions:

1. Construction of a foundation for a building utilizing piles of more than one type or capacity;
2. Modification of an existing foundation by the addition of piles of a type or capacity other than those of the existing piling;
3. Construction or modification of a foundation utilizing different methods or more than one method of installation, or using different types or capacities of equipment (such as different types of hammers having markedly different striking energies or speeds); or
4. Support of part of a building on piles and part on footings.

1808.2.9 Lateral support.

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

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

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 in accordance with accepted engineering practice or by lateral load tests in accordance with ASTM D 3966 to at least twice the proposed design working load. The resulting allowable load shall not be more than one-half of that test load that produces a gross lateral movement of 1 inch (25mm) at the ground surface.

In the absence of specific project requirements as determined by the engineer, 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. The maximum allowable lateral load of a pile shall be 1 ton (8.9 kN), unless verified by load test. Lateral capacities for pile groups shall be modified to account for group effects in accordance with accepted engineering practice.

1808.2.10 Allowable stresses in piles and use of higher allowable pier or pile stresses. Allowable stresses for designing piles shall be as specified in Sections 1809, 1810, and 1811. Allowable stresses greater than those specified for piers or for each pile type in Sections 1809 and 1810 are permitted where supporting data justifying such higher stresses are filed and approved by the commissioner. Such substantiating data shall include:

1. A soils investigation in accordance with Section 1802.
2. Pier or pile load tests in accordance with Section 1808.2.8.3, regardless of the load supported by the pier or pile. The design and installation of the pier or pile foundation shall be under the direct supervision of an engineer knowledgeable in the field of soil mechanics and pier or pile foundations who shall certify to the commissioner that the piers or piles as installed satisfy the design criteria.

1808.2.11 Piles in subsiding areas. Where piles are driven through subsiding fills or other subsiding strata and derive support from underlying firmer materials, consideration shall be given to the downward frictional forces that may be imposed on the piles by the subsiding upper strata.

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

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

1808.2.14 Installation sequence. Piles shall be installed in such sequence so 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.

1808.2.14.1 Protection of adjacent property. Piles and piers shall be installed with adequate provision for the protection of adjacent buildings and property.

1808.2.15 Use of vibratory drivers. Vibratory drivers shall only be used to install piles where the pile is subsequently seated by an impact hammer to the final driving criteria established in accordance with Section 1808.2.8.2.

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

1808.2.17 Protection of pile materials. Where boring records or site conditions indicate possible deleterious action on pier or pile materials because of soil constituents, changing water levels or other factors, the pier or pile materials shall be adequately protected by materials, methods or processes approved by the engineer. 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. The following specific provisions shall apply:

1. Untreated timber piles shall not be used unless the top level of the pile is below the permanent water table. The permanent water table level shall not be assumed higher than the invert level of sewer, drain, or subsurface structure in the adjacent streets, nor higher than the water level at the site resulting from the lowest drawdown of wells or sumps, but in no case shall untreated timber piles be used where the cut-off level is less than 10 feet (3048 mm) below the adjacent legal grade. Where treated piles are required, preservative treatment shall consist of impregnation with creosote or a creosote solution or CCA treatment. For piles entirely embedded below grade, a pentachlorophenal solution may be used. Treatment shall be in accordance with all requirements of the AWPA standards and as specified in Section 1809.1.2.
2. Piles installed in ash or garbage fills, cinder fills, and piles that are free-standing in or near a seawater environment, or that are used for the support of chemical plants, coal piles and piles under similar conditions of chemical seepage or aggressive action, or that are used for support of electrical generating plants, shall be investigated regarding the need for special protective treatment. Where special protective treatment is indicated by the engineer, such piles shall be protected against deterioration by encasement, coating or other device acceptable to the engineer.

1808.2.17.1 Protection of piles during installation. Piling shall be handled and installed to the required penetration and resistance by methods that leave the piles' strength unimpaired and that develop and retain their required load-bearing resistance. Any damaged pile shall be satisfactorily repaired or the pile shall be rejected. As an alternative and subject to the approval by the commissioner, damaged or misaligned piles or piles not reaching design tip elevation may be used at a reduced fraction of the design load based on an analysis by the engineer.

1808.2.17.2 Equipment. Equipment and methods of installation shall be such that piles are installed in their proper position and alignment, without damage. Equipment shall be maintained in good working order. The pile-driving hammer shall travel freely in the leads. The hammer shall deliver its rated energy and measurements shall be made of the fall of the ram or other suitable data shall be obtained at intervals necessary to verify the actual energy delivered during the final 20 blows of the hammer.

1808.2.17.3 Cushion or cap block. The cushion or cap block shall be a solid block of hardwood with its grains parallel to the axis of the pile and enclosed in a tight-fitting steel housing, or other accepted equivalent assembly. If laminated materials are used, their type and construction shall be such that their strength is equal to or greater than hardwood. Wood chips, pieces of rope, hose, shavings, automobile tires or similar materials shall not be used. Cap block cushions shall be replaced if burned, crushed, or otherwise damaged. Other cushion materials may be used subject to the approval of the engineer.

1808.2.17.4 Followers. Followers shall not be used unless permitted in writing by the engineer responsible for the pile driving operation. The required driving resistance shall account for the losses of driving energy transmitted to the pile because of the follower. The follower shall be a single length section, shall be provided with a socket or hood carefully fitted to the top of the pile to minimize loss of energy and to prevent damage to the pile, and shall have sufficient rigidity to prevent "whip" during driving.

1808.2.18 Use of existing piers or piles. Piers or piles left in place where a structure has been demolished shall not be used for the support of new construction unless the piles are load tested, original installation and testing records are available, or the new loads are no more than half the calculated previous loads on the piles. The engineer shall determine and certify that the piers or piles are sound and meet the requirements of this code. The design load applied to such piers or piles shall be the lowest allowable load as determined by tests, redriving data or calculations.

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

1808.2.20 Identification. All pier or pile materials shipped or delivered to the job site shall be identified for conformity to the specified grade and this identification shall be maintained continuously from the point of manufacture to the point of installation. Such shipment or delivery shall be accompanied by a certification from the material supplier or manufacturer indicating conformance with the construction documents. Such certification shall be made available to the engineer of record and the department. In the absence of adequate data, pier or pile materials shall be tested by an approved agency to determine conformity to the specified grade. The approved agency shall furnish a certification of compliance to the engineer of record, or upon request to the commissioner.

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 commissioner 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.

1808.2.21.1 Tolerance in alignment of the pile axis. If the axis of any pile is installed out of plumb or deviates from the specified batter by more than 4 percent of the pile length, the design of the foundation shall be modified to resist the resulting vertical and lateral forces. In types of piles for which subsurface inspection is not possible, this determination shall be made on the exposed section of the pile, which section, at the time of checking axial alignment, shall not be less than 2 feet (610 mm) in length. In piles that can be checked for axial alignment below the ground surface, the sweep of the pile axis shall not exceed 4 percent of the embedded length.

1808.2.21.2 Tolerance in the location of the head of the pile. A tolerance of 3 inches (76 mm) from the designed location shall be permitted in the installation of each pile, without reduction in load capacity of the pile group. Where piles are installed out of position in excess of this amount, the true loading on such piles shall be analytically determined from a survey that defines the actual location of the piles as driven, and using the actual eccentricity in the pile group with respect to the line of action of the applied load.

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

1808.2.23 Seismic design of piers or piles. Seismic design of piers and piles shall be done in accordance with Sections 1808.2.23.1 through 1808.2.23.2.

1808.2.23.1 Seismic Design Category C. Where a structure is assigned to Seismic Design Category C in accordance with Section 1616, individual pile caps, piers or piles shall be interconnected by ties. Ties shall be capable of carrying, in tension and compression, a force equal to the product of the larger pile cap or column load times the seismic coefficient, SDS, divided by 10 unless it can be demonstrated that equivalent restraint is provided by reinforced concrete beams within slabs on grade or reinforced concrete slabs on grade or confinement by competent rock, hard cohesive soils or very dense granular soils.

Exception: Piers supporting foundation walls, isolated interior posts detailed so the pier is not subject to lateral loads, and lightly loaded exterior decks and patios of Group R-3 and U occupancies not exceeding two stories of light-frame construction, are not subject to interconnection if it can be shown that the soils are of adequate stiffness, subject to the approval of the commissioner.

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

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

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

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

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

1808.2.23.1.2 Design details. Pier or pile moments, shears and lateral deflections used for design shall be established considering the nonlinear interaction of the shaft and soil, as recommended by the engineer. Where the ratio of the depth of embedment of the pile-to-pile diameter or width is less than or equal to six, the pile may be assumed to be rigid.

Pile group effects from soil on lateral pile nominal strength shall be included where pile center-to-center spacing in the direction of lateral force is less than eight pile diameters. Pile group effects on vertical nominal strength shall be included where pile center-to-center spacing is less than three pile diameters. The pile uplift soil nominal strength shall be taken as the pile uplift strength as limited by the frictional force developed between the soil and the pile.

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

1808.2.23.2 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 in Section 1808.2.23.1 shall be met. Provisions of ACI 318, Section 21.10.4, shall also apply when not in conflict with the provisions of Sections 1808 through 1812. Concrete shall have a specified compressive strength of not less than 3,000 psi (20.68 MPa) at 28 days.

Exceptions:

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

1808.2.23.2.1 Design details for piers, piles and grade beams. Piers or piles shall be designed and constructed to withstand maximum imposed curvatures from earthquake ground motions and structure response. Curvatures shall include free-field soil strains modified for soil-pile-structure interaction coupled with pier or pile deformations induced by lateral pier or pile resistance to structure seismic forces. Concrete piers or piles on Site Class E or F sites, as determined in Section 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. For precast prestressed concrete piles, detailing provisions as given in Sections 1809.2.3.2.1 and 1809.2.3.2.2 shall apply.

Grade beams shall be designed as beams in accordance with ACI 318, Chapter 21. When grade beams have the capacity to resist the forces from the load combinations in Section 1605.4, they need not conform to ACI 318, Chapter 21.

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

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

1808.2.23.2.3 Flexural strength. Where the vertical, lateral-force-resisting elements are columns, the grade beam or pile cap flexural strengths shall exceed the column flexural strength.

The connection between batter piles and grade beams or pile caps shall be designed to resist the nominal strength of the pile acting as a short column. Batter piles and their connections shall be capable of resisting forces and moments from the load combinations of Section 1605.4.

SECTION BC 1809
DRIVEN PILE FOUNDATIONS

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

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

1809.1.2 Preservative treatment. Timber piles used to support permanent structures shall be treated in accordance with this section unless it is established that the tops of the untreated timber piles will be below the lowest ground-water level assumed to exist during the life of the structure as specified in Section 1808.2.17. 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 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.

1809.1.4 Sizes of piles. Piles shall be of adequate size to resist the applied loads without creating stresses in the pile material in excess of 1,200 psi (8.27 MPa) for piles of southern pine, Douglas fir, oak, or other wood of comparable strength; or 800 psi (5.52 MPa) for piles of cedar, Norway pine, spruce or other wood of comparable strength. Piles of 25 tons' (222.5 kN) capacity or more shall have a minimum 8-inch tip (203 mm) with uniform taper. Piles of less than 25 tons' (222.5 kN) capacity shall have a minimum 6-inch (152 mm) tip with uniform taper. All piles, regardless of capacity, driven to end bearing on bedrock of Classes 1a to 1d and compact gravels and sands of Class 2a shall have a minimum 8-inch (203 mm) tip and a uniform taper. Any species of wood may be used that conforms to ASTM D 25 and that will stand the driving stresses.

1809.1.5 Lagged or inverted piles. The use of lagged or inverted piles is permitted. Double lagging shall be adequately connected to the basic pile material to transfer the full pile load from the basic pile material to the lagging without exceeding values of allowable stress as established in Chapter 23. The connection for single lagging shall be proportioned for half the pile load. The diameter of any inverted pile at any section shall be adequate to resist the applied load without exceeding the stresses specified in Section 1809.1.4, but in no case shall it be less than 8 inches (203 mm).

1809.2 Precast concrete piles.

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

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

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

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

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

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

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

1809.2.2.2.1 Seismic reinforcement in Seismic Design Category C. Where a structure is assigned to Seismic Design Category C in accordance with Section 1616, longitudinal reinforcement with a minimum steel ratio of 0.01 shall be provided throughout the length of precast concrete piles. Within three pile diameters of the bottom of the pile cap, the longitudinal reinforcement shall be confined with closed ties or spirals of a minimum $^3/_8$-inch (9.5 mm) diameter. Ties or spirals shall be provided at a maximum spacing of eight times the diameter of the smallest longitudinal bar, not to exceed 6 inches (152 mm). Throughout the remainder of the pile, the closed ties or spirals shall have a maximum spacing of 16 times the smallest longitudinal-bar diameter, not to exceed 8 inches (203 mm).

1809.2.2.2.2 Seismic reinforcement in Seismic Design Category D. 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 by this section. Transverse confinement reinforcement consisting of closed ties or equivalent spirals shall be provided in accordance with Sections 21.4.4.1, 21.4.4.2 and 21.4.4.3 of ACI 318 within three pile diameters of the bottom of the pile cap. For other than Site Class E or F, or liquefiable sites and where spirals are used as the transverse reinforcement, 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) 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 (38mm) 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 seawater shall have a concrete cover of not less than 3 inches (76 mm).

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

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

1809.2.3.2 Design. Precast prestressed piles shall be designed to resist stresses induced by handling and driving as well as by loads. The effective prestress in the pile shall not be less than 400 psi (2.76MPa) for piles up to 30 feet (9144 mm) in length, 550 psi (3.79MPa) 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 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.

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

where:

f'_c = Specified compressive strength of concrete, psi (MPa)

f_{yh} = Yield strength of spiral reinforcement $\leq$ 85,000 psi (586 MPa).

ρ_s = Spiral reinforcement index (vol. spiral/vol. core).

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

The pile cap connection by means of dowels as indicated in Section 1808.2.23.1 is permitted. Pile cap connection by means of developing pile reinforcing strand is permitted provided that the pile reinforcing strand results in a ductile connection.

1809.2.3.2.2 Design in Seismic Design Category D. 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, do 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 Sec. 12.14.3 of ACI 318.

5. Where the transverse reinforcement consists of circular spirals, the volumetric ratio of spiral transverse reinforcement in the ductile region shall comply with the following:

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

(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:

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

where:

A_g = Pile cross-sectional area, square inches (mm^2).

A_{ch} = Core area defined by spiral outside diameter, square inches (mm^2).

f'_c = Specified compressive strength of concrete, psi (MPa)

f_{yh} = Yield strength of spiral reinforcement $\leq$ 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).

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} = $\leq$ 70,000 psi (483 MPa).

h_c = Cross-sectional dimension of pile core measured center to center of hoop reinforcement, inch (mm).

s = Spacing of transverse reinforcement measured along length of pile, inch (mm).

A_{sh} = Cross-sectional area of tranverse reinforcement, square inches (mm^2)

f'_c = Specified compressive strength of concrete, psi (MPa)

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

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

1809.2.3.3 Allowable stresses. The maximum allowable design compressive stress, fc, in concrete shall be determined as follows:

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

where:

f'_c = The 28-day specified compressive strength of the concrete.

f_{pc} = The effective prestress stress on the gross section.

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

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

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

1809.3.1 Materials. Structural steel piles, steel pipe and fully welded steel piles fabricated from plates shall conform to ASTM A36, ASTM A252, ASTM A283, ASTM A572, ASTM A 588 or ASTM A 913.

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

Exception: Where justified in accordance with Section 1808.2.10.1, the allowable axial stress is permitted to be increased above $0.35F_y$, but shall not exceed $0.5F_y$.

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

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

1809.3.4 Dimensions of steel pipe piles. Steel pipe piles driven open ended shall have a nominal outside diameter of not less than 8 inches (203 mm). The pipe shall have a minimum of 0.34 square inch (219 mm2) 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 Sections 1810.1.1 through 1810.1.3.

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

1810.1.2 Reinforcement. Except for steel dowels embedded 5 feet (1524 mm) or less in the pile, reinforcement where required shall be placed in accordance with Section 1810.3.4 and shall be assembled and tied together and 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 filled, while the grout is still in a semifluid state.

1810.1.2.1 Reinforcement in Seismic Design Category C. Where a structure is assigned to Seismic Design Category C in accordance with Section 1616, a minimum longitudinal reinforcement ratio of 0.0025 shall be provided for uncased cast-in-place concrete drilled or augered piles, piers or caissons in the top one-third of the pile length, a minimum length of 10 feet (3048 mm) below the ground or that required by analysis, whichever length is greatest. The minimum reinforcement ratio, but no less than that ratio required by rational analysis, shall be continued throughout the flexural length of the pile. There shall be a minimum of four longitudinal bars with closed ties (or equivalent spirals) of a minimum $^3/_8$-inch (9 mm) diameter provided at 16-longitudinal-bar diameter maximum spacing. Transverse confinement reinforcing with a maximum spacing of the lesser of 6 inches (152 mm) or 8-longitudinal-bar diameters shall be provided within a distance equal to three times the least pile dimension of the bottom of the pile cap.

1810.1.2.2 Reinforcement in Seismic Design Category D. 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, a minimum longitudinal reinforcement ratio of 0.005 shall be provided for uncased cast-in-place drilled or augered concrete piles, piers or caissons in the top one-half of the pile length, a minimum length of 10 feet (3048 mm) below ground or throughout the flexural length of the pile, whichever length is greatest. The flexural length shall be taken as the length of the pile to a point where the concrete section cracking moment strength multiplied by 0.4 exceeds the required moment strength at that point. There shall be a minimum of four longitudinal bars with transverse confinement 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 three times the least pile dimension of the bottom of the pile cap. Use of 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 is allowed. 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.

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

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

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

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

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

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

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

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

1810.3.1 Allowable stresses. The allowable design stress in the concrete of drilled uncased piles shall not exceed 33 percent of the 28-day specified compressive strength (f'_c). The allowable design stress in the concrete of augered cast-in-place piles shall not exceed 25 percent of the 28-day specified compressive strength (f'_c). The allowable compressive stress of reinforcement shall not exceed 34 percent of the yield strength of the steel or 25,500 psi (175.8 Mpa).

1810.3.2 Dimensions. The minimum diameter of drilled or augered uncased piles shall be 12 inches (305 mm).

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

Where grout is placed by pumping through a hollow-stem auger, the auger shall be permitted to rotate in a clockwise direction during withdrawal. An initial head of grout shall be established and maintained on the auger flights before withdrawal. The auger shall be withdrawn in a continuous manner in increments of about 12 inches (305 mm) each. Grout pumping pressures shall be measured and maintained high enough at all times to offset hydrostatic and lateral earth pressures. Grout volumes shall be measured to ensure that the volume of grout placed in each pile is equal to or greater than the theoretical volume of the hole created by the auger. Where the installation process of any pile is interrupted or a loss of grout pressure occurs, the pile shall be re-drilled to 5 feet (1524 mm) below the elevation of the tip of the auger when the installation was interrupted or grout pressure was lost and reformed. Augered cast-in-place piles shall not be installed within six pile diameters center to center of a pile filled with concrete or grout less than 12 hours old, unless approved by the engineer. The level at which return of the grout occurs during withdrawal shall be recorded. If the grout level in any completed pile drops during installation of an adjacent pile, the pile shall be replaced. The installation shall be performed under the direct supervision of the engineer. The engineer shall certify to the commissioner that the piles were installed in compliance with the approved construction documents.

1810.3.4 Reinforcement. For piles installed with a hollow-stem auger, where full-length longitudinal steel reinforcement is placed without lateral ties, the reinforcement shall be placed through ducts in the auger prior to filling the pile with concrete. All pile reinforcement shall have a concrete cover of not less than $2^1/_2$ inches (64 mm).

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

1810.3.5 Reinforcement in Seismic Design Category C or D. 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. Driven uncased piles shall conform to Sections 1810.4.1 through 1810.4.4.

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

1810.4.2 Dimensions. The minimum diameter of the driven uncased pile shall be 12 inches (305 mm).

1810.4.3 Installation. Piles shall not be driven within six pile diameters center to center in granular soils or within one-half the pile length in cohesive soils of a pile filled with concrete less than 48 hours old unless approved by the commissioner. If the concrete surface in any completed pile rises or drops, the pile shall be replaced. Piles shall not be installed in soils that could cause pile heave. The installation shall be performed under the direct supervision of the engineer who shall certify to the commissioner that the piles were installed in compliance with the approved design.

1810.4.4 Concrete cover. Pile reinforcement shall have a concrete cover of not less than $2^1/_2$ inches (64 mm), measured from the inside face of the drive casing or mandrel.

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

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

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

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

1810.5.2.2 Shell type. The shell shall be seamless or provided with seams of strength equal to the basic material and be of a configuration that will provide confinement to the cast-in-place concrete.

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

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

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

The steel shells shall be driven in such order and with such pacing as to ensure against distortion of or injury to piles already in place. A pile shall not be driven within four and one-half average pile diameters of a pile filled with concrete less than 24 hours old unless approved by the commissioner. Concrete shall not be placed in steel shells within heave range of driving.

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

1810.5.4.1 Seismic reinforcement. Where a structure is assigned to Seismic Design Category C or D in accordance with Section 1616, the reinforcement requirements for drilled or augered uncased piles in Section 1810.3.5 shall be met.

Exception: A spiral-welded metal casing of a thickness not less than manufacturer's standard gage No. 14 (0.068 inch) is permitted to provide concrete confinement in lieu of the closed ties or equivalent spirals required in an uncased concrete pile. Where used as such, the metal casing shall be protected against possible deleterious action due to soil constituents, changing water levels or other factors indicated by boring records of site conditions.

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

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

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

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

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

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

1810.6.4.1 Seismic reinforcement. Where a structure is assigned to Seismic Design Category C or D in accordance with Section 1616, minimum reinforcement no less than 0.01 times the cross-sectional area of the pile concrete shall be provided in the top of the pile with a length equal to two times the required cap embedment anchorage into the pile cap, but not less than the tension development length of the reinforcement. The wall thickness of the steel pipe shall not be less than $^3/_{16}$ inch (5 mm).

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

1810.7 Caisson piles. Caisson piles shall conform to the requirements of Sections 1810.7.1 through 1810.7.9.

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

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

1810.7.3 Design. The depth of the rock socket shall be sufficient to develop the full load-bearing capacity of the caisson pile with a minimum safety factor of two, but the depth of the socket in Class 1c rock or better below the shoe shall not be less than 3 feet (914 mm)‡ of the outside diameter of the pipe. The minimum outside diameter of the caisson pile shall be 7 inches (194 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 30 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, $0.33f'_c$; steel pipe, $0.35F_y$ and structural steel core, $0.50F_y$.

1810.7.6 Installation. The rock socket and pile shall be thoroughly cleaned of foreign materials before filling with concrete. Steel cores shall be set 1 inch (25 mm)‡ above the base of the rock socket. Concrete shall not be placed through water except where a tremie or other method approved by the commissioner is used.

1810.7.7 Dimensions of caisson piles. Caisson piles shall consist of concrete pipe piles that are socketed into rock and constructed with steel reinforcement, and in which the socket is observed before the concrete is poured. Steel reinforcement shall be covered with at least $1^1/_2$ inches (38 mm) of concrete. The minimum diameter of caisson piles shall be 7 inches (178 mm). A suitable steel driving shoe shall be welded to the bottom of each caisson pile. The center-to-center spacing of caisson sockets shall be at least two and one-half times the outside diameter of the shell but not less than 4 feet (1219 mm).

1810.7.8 Inspection. All rock sockets shall be inspected to verify rock quality. Inspection may be accomplished by direct observation or by video methods or by a core boring performed prior to the drilling of the socket. Load tests performed in accordance with Section 1808.2.8.3 may be substituted for inspection of rock sockets.

1810.7.9 Caisson piles in soil. Caisson piles as described in Section 1810.7 may be installed in soil provided that the socket is formed entirely in soil of Class 4 or better and the concrete is placed under pressure exceeding 1.5 times the existing total overburden pressure. The socket shall be formed by extending the casing to the bottom of the socket and withdrawing the casing while the concrete is being pumped under pressure. Piles shall be installed in accordance with the provisions of Sections 1810.3 and 1810.7. For diameters less than 12 inches (305 mm), the casing above the socket shall remain in place permanently. Reinforcing to the socket shall be placed in the casing to the depth of the socket prior to placing concrete.

SECTION BC 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 and where concrete and steel are used as part of the pile assembly, the concrete reinforcement shall comply with Sections 1810.1.2.1 and 1810.1.2.2 and the steel section shall comply with Section 1809.3.4 or 1810.6.4.1.

SECTION BC 1812 PIER FOUNDATIONS

1812.1 General. Isolated and multiple piers used as foundations shall conform to the requirements of Sections 1812.2 through 1812.11, as well as the applicable provisions of Section 1808.2.

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

1812.3 Materials. Concrete shall have a 28-day specified compressive strength (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 $2^1/_2$ inch (64 mm) concrete cover requirement permitted to be reduced to 2 inches (51 mm) for Group R-3 and U occupancies not exceeding two stories of light-frame construction, provided the construction method is approved by the commissioner.

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

Exceptions:

1. Isolated piers supporting posts of Group R-3 and U occupancies not exceeding two stories of light-frame construction are permitted to be reinforced as required by rational analysis but not less than a minimum of one No. 4 bar, without ties or spirals, when detailed so the pier is not subject to lateral loads and the soil is determined to be of adequate stiffness.
2. Isolated piers supporting posts and bracing from decks and patios appurtenant to Group R-3 and U occupancies not exceeding two stories of light-frame construction are permitted to be reinforced as required by rational analysis but not less than one No. 4 bar, without ties or spirals, when the lateral load, E, to the top of the pier does not exceed 200 pounds (890 N) and the soil is determined to be of adequate stiffness.
3. Piers supporting the concrete foundation wall of Group R-3 and U occupancies not exceeding two stories of light-frame construction are permitted to

be reinforced as required by rational analysis but not less than two No. 4 bars, without ties or spirals, when it can be shown the concrete pier will not rupture when designed for the maximum seismic load, Em, and the soil is determined to be of adequate stiffness.

4. Closed ties or spirals where required by Section 1810.1.2.2 are permitted to be limited to the top 3 feet (914 mm) of the piers 10 feet (3048 mm) or less in depth supporting Group R-3 and U occupancies of Seismic Design Category D, not exceeding two stories of light-frame construction.

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

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

1812.7 Reserved.

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

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

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

1812.10 Dewatering. Where piers are carried to depths below water level, the piers shall be constructed by a method that will provide accurate preparation and inspection of the bottom in dry conditions.

1812.11 Method of construction. Methods of construction shall conform to ACI 336.1 "Standard Specification for the Construction of Drilled Piers."

SECTION BC 1813
LIQUEFACTION ANALYSIS

1813.1 General. An assessment of the liquefaction potential shall be determined for each building site. The evaluation of liquefaction potential shall include the following considerations:

1. Noncohesive soils below ground-water table and less than 50 feet (15 240 mm) below the ground surface shall be considered to have potential for liquefaction.
2. The potential for liquefaction on level ground shall be determined on the basis of the structural occupancy categories associated with the uncorrected standard penetration resistance (N) at the site, as defined in Figure 1813.1, or a site-specific analysis performed by an engineer with specific expertise in the evaluation of liquefaction.

1813.2 Site-specific analyses. In evaluating liquefaction potential, the analysis shall consider the following parameters: ground surface acceleration, earthquake magnitude, magnitude scaling factor, effective overburden pressure, hammer energy, cone penetration resistance (where applicable), and fines content. If a site response analysis is conducted, bedrock acceleration time histories and a shear wave velocity profile based on in-situ measurements may be utilized. These analyses may consider the results of laboratory cyclic shear tests.

1813.3 Foundation design analysis. The foundation design analysis shall consider an assessment of potential consequences of any liquefaction and soil strength loss including estimation of differential settlement, lateral movement or reduction in foundation soil-bearing capacity, and may incorporate the potential benefits of any proposed mitigation measures. Such measures may be given consideration in the design of the structure and can include, but are not limited to, ground improvement, pore pressure dissipation, selection of appropriate foundation type and depths, selection of appropriate structural systems to accommodate anticipated displacements, or any combination of these measures.

In evaluating the potential for liquefaction, the effect of settlements induced by seismic motions and loss of soil strength shall be considered. The analysis performed shall incorporate the effects of peak ground acceleration, appropriate earthquake magnitudes and duration consistent with the design earthquake ground motions as well as uncertainty and variability of soil properties across the site. Peak ground acceleration, seismically induced cyclic stress ratios and pore pressure development may be determined from a site-specific study taking into account soil amplification effects and ground motions appropriate for the seismic hazard. Other recognized methods of analysis may be used in the evaluation process subject to the approval of the commissioner. Effects of pore water pressure buildup shall be considered in the design except for the following conditions:

1. The calculated cyclic shear demand is equal to or less than 75 percent of the calculated cyclic shear strength for Structural Occupancy Category I, II and III structures.

2. The calculated cyclic shear demand is equal to or less than 85 percent of the calculated cyclic shear strength for Structural Occupancy Category IV structures.

1813.4 Design considerations. At sites where liquefaction is determined to be probable, the following considerations shall be in the design:

1. Liquefiable soils shall be considered to have no passive (lateral) resistance or bearing capacity value during an earthquake, unless shown otherwise by accepted methods of analysis. The engineer shall submit an analysis for review and approval by the commissioner, demonstrating that the proposed construction is safe against the effects of soil liquefaction.
2. Where liquefiable soils are present in sloped ground or over sloped nonliquefiable substrata and where lateral displacement is possible, the engineer shall submit a stability analysis for review and approval by the commissioner, demonstrating that the proposed construction is safe against failure of the soil and that the effect of potential lateral displacements are acceptable.

SECTION BC 1814
UNDERPINNING

1814.1 General. Where the protection and/or support of adjacent structures is required, an engineer shall prepare a preconstruction report summarizing the condition of the structure as determined from examination of the structure, the review of available design documents and if necessary, the excavation of test pits. The engineer shall determine the requirements for underpinning and protection and prepare site-specific plans, details, and sequence of work for submission to the commissioner. Such support may be provided by underpinning, sheeting, and bracing, or by other means acceptable to the commissioner. Underpinning piers, walls, piles and footings shall be designed and installed in accordance with provisions of this chapter and Chapter 33 and shall be inspected in accordance with provisions of Chapter 17.

1814.1.1 Underpinning and bracing. Where underpinning is used for the support of adjacent structures, the piers, wall piles or footings shall be installed in such manner so as to prevent the lateral or vertical displacement of the adjacent structure, to prevent deterioration of the foundations or other effects that would disrupt the adjacent structure. The sequence of installation and the requirements for sheeting, preloading, wedging with steel wedges, jacking or dry packing shall be identified in the design.

1814.2 Use of rock support in lieu of underpinning. Existing structures founded at a level above the level of adjacent new construction may be supported on Class1a and 1b rock in lieu of underpinning, sheeting and bracing or retaining walls, provided that a report by the engineer substantiates the safety of the proposed construction. The engineer shall also certify that the he or she has inspected the exposed rock and the jointing therein and has determined whether supplemental support of the rock face is required.

1814.3 Monitoring of influenced structures. A land surveyor or engineer shall monitor the behavior of influenced structures during construction and for as long as necessary after construction concludes, as determined by the commissioner.

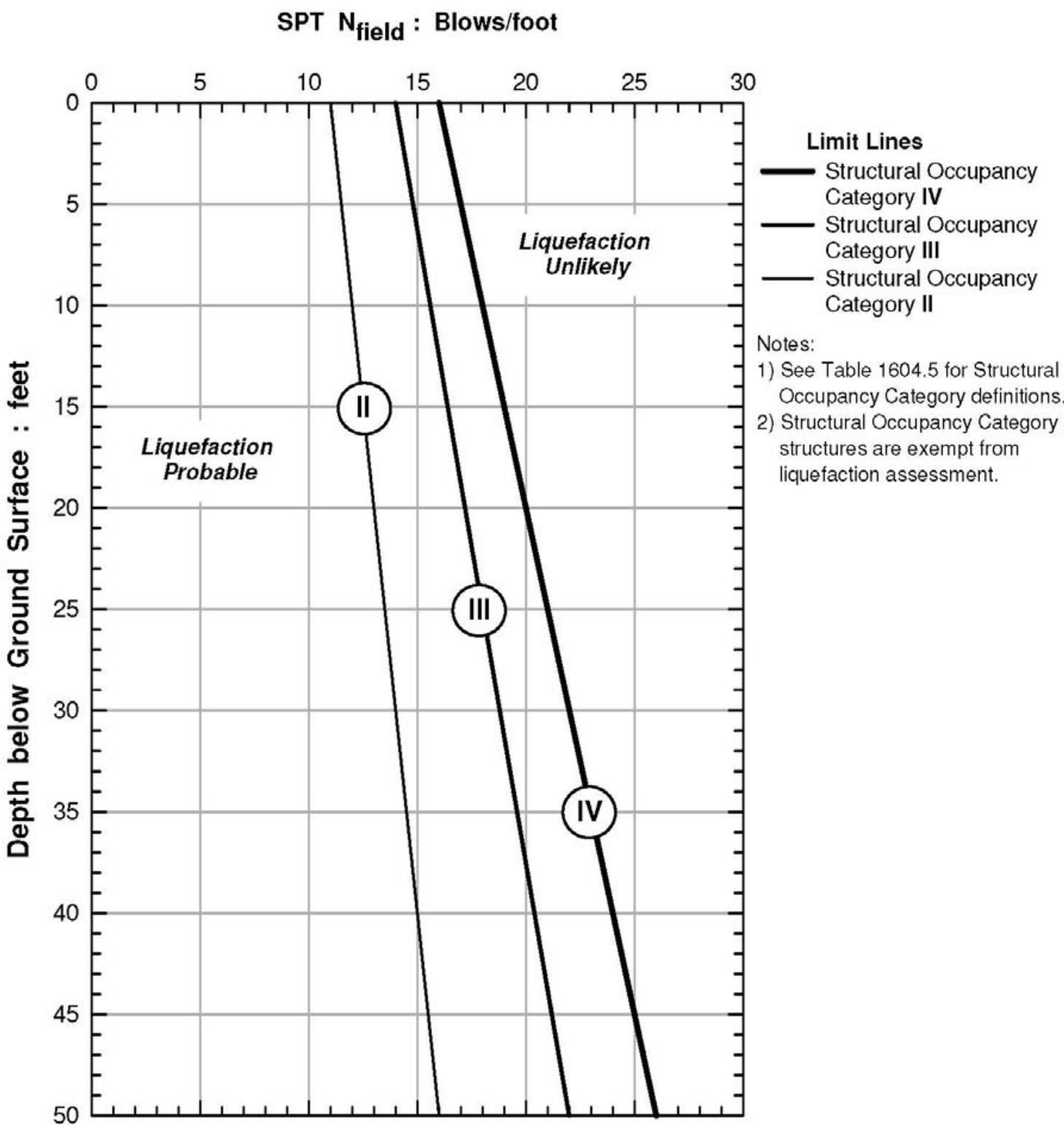

FIGURE 1813.1
LIQUEFACTION ASSESSMENT DIAGRAM