

Guide to Design, Manufacture, and Installation of Concrete Piles

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Guide to Design, Manufacture, and Installation of Concrete Piles

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Guide to Design, Manufacture, and Installation of Concrete Piles

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This report presents recommendations to assist the design architect/engineer, manufacturer, construction engineer, and contractor in the design, manufacture, and installation of most types of concrete piles.

Keywords: augered piles; bearing capacity; composite construction; concrete piles; corrosion; drilled piles; foundations; harbor structures; loads; prestressed concrete; quality control; steel reinforcement; soil mechanics; storage; tolerances.

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CHAPTER 1—INTRODUCTION

1.1—General

Piles are slender structural elements installed in the ground to support a load or compact the soil. They are made of several materials or combinations of materials and are installed by impact driving, jacking, vibrating, jetting, drilling, grouting, or combinations of these techniques. Piles are difficult to summarize and classify because there are many types, and new types are still being developed. This report covers only the types of piles currently used in North American construction projects. A pile type can be assigned

a wide variety of names or classifications by various agencies, codes, technical groups, and in various geographical regions. No attempt is made herein to reconcile the wide variety of names used with a given pile type.

Piles can be described by the predominant material from which they are made: steel, concrete (or cement and other materials), or timber. Composite piles have an upper section of one material and a lower section of another. Piles made entirely of steel are usually H-sections or unfilled pipe; however, other steel members can be used. Timber piles are typically tree trunks that are peeled, sorted to size, and driven into place. The timber is usually treated with preservatives, but untreated piles can be used when positioned entirely below the permanent water table. The design of steel and timber piles is not considered herein except when used in conjunction with concrete. Most of the remaining types of existing piles contain concrete or a cement-based material.

Driven piles are typically top-driven with an impact hammer activated by air, steam, hydraulic, or diesel mechanisms, although vibratory drivers are occasionally used. Some piles, such as steel corrugated shells and thin-wall pipe piles, would be destroyed if top-driven. For such piles, an internal steel mandrel is inserted into the pile to receive the blows of the hammer and support the shell during installation. The pile is driven into the ground with the mandrel, which is then withdrawn. Driven piles tend to compact the soil beneath the pile tip.

Several types of piles are installed by drilling or rotating with downward pressure, instead of driving. Drilled piles usually involve concrete or grout placement in direct contact with the soil, which can produce side-friction resistance greater than that observed for driven piles. On the other hand, because they are drilled rather than driven, drilled piles do not compact the soil beneath the pile tip and, in fact, can loosen the soil at the tip. Post-grouting may be used after installation to densify the soil under the pile tip.

Concrete piles are classified according to the condition under which the concrete is cast. Some concrete piles (precast piles) are cast in a plant before driving, which allows controlled inspection of all phases of manufacture. Other piles are cast-in-place (CIP), a term used in this report to designate piles made of concrete placed into a previously-driven, enclosed container. Concrete-filled corrugated shells and closed-end pipe are examples of CIP piles. Other piles are cast-in-situ (CIS), a term used in this report to designate concrete cast directly against the earth. Drilled piers and auger-grout piles are examples of CIS piles.

1.2—Types of piles

1.2.1 Precast concrete piles—This general classification covers both conventionally reinforced concrete piles and prestressed concrete piles. Both types can be formed by casting, spinning (centrifugal casting), slipforming, or extrusion and are made in various cross-sectional shapes, such as triangular, square, octagonal, and round. Some piles are cast with a hollow core. Precast piles usually have a uniform cross section but can have a tapered tip. Precast concrete piles are designed and manufactured to withstand handling and driving stresses in addition to service loads.

1.2.1.1 Reinforced concrete piles—These piles are constructed of conventionally reinforced concrete with internal reinforcement consisting of a cage made up of several longitudinal steel bars and lateral steel in the form of individual ties or a spiral.

1.2.1.2 Prestressed concrete piles—These piles are constructed using steel rods, strands, or wires under tension. The prestressing steel is typically enclosed in wire spirals or ties. Nonmetallic strands have also been studied for use in piles (Sen et al. 1998a,b, 1999a,b), but their use is not covered in this report.

Prestressed piles can either be pre- or post-tensioned. Pretensioned piles are usually cast full length in permanent casting beds. Post-tensioned piles are usually manufactured in sections that are then assembled and prestressed to the required pile lengths in the manufacturing plant or at the job site.

1.2.1.3 Sectional precast concrete piles—These types of piles are either conventionally reinforced or prestressed pile sections with splices or mechanisms that extend them to the required length. Splices typically provide the full compressive strength of the pile, and some splices can provide the full tension, bending, and shear strength. Conventionally reinforced and prestressed pile sections can be combined in the same pile for design purposes if desired.

1.2.2 Cast-in-place concrete piles—Generally, CIP piles may be a corrugated, mandrel-driven, steel shell, or a top-driven or mandrel-driven steel pipe; all have a closed end. Concrete is cast into the shell or pipe after driving. Thus, unless it becomes necessary to re-drive the pile after concrete placement, the concrete is not subjected to driving stresses.

The corrugated shells can be of uniform section, tapered, or stepped cylinders, also known as step-taper. Pipe is also available in similar configurations, but normally is of uniform section or a uniform upper section with a tapered lower section.

CIP pile casings can be inspected internally before concrete placement. Reinforcing steel can be added full-length or partial-length, as dictated by the design.

1.2.3 Enlarged-tip piles—In granular soils, pile-tip enlargement generally increases pile bearing capacity. One type of enlarged-tip pile, also called a compacted concrete pile, is formed by bottom-driving a tube with a concrete plug to the desired depth. The concrete plug is then forced out into the soil as concrete is added. Upon completion of the base, the tube is withdrawn while expanding concrete out of the tip of the tube; this forms a CIS concrete shaft. Alternately, a pipe or corrugated shell casing can be bottom-driven into the base and the tube withdrawn. The resulting annular space (between soil and pile) either closes onto the shell, or else granular filler material is added to fill the space. The pile is then completed as a CIP concrete pile. In either the CIS or CIP configuration, reinforcing steel can be added to the shaft as dictated by the design.

Another enlarged-tip pile consists of a precast reinforced concrete base in the shape of a frustum of a cone that is attached to a pile shaft. Most frequently, the shaft is a corrugated shell or thin-walled pipe, with the shaft and enlarged-

tip base being mandrel-driven to bear in generally granular subsoils. There will be an annular space between the pile and soil, as noted previously. The pile shaft is completed as a CIP pile, and reinforcement is added as dictated by the design. Precast, enlarged-tip bases have also been used with solid shafts, such as timber piles. Precast, enlarged-tip bases can be constructed in a wide range of sizes.

1.2.4 Drilled-in caissons—A drilled-in caisson is a special type of CIP concrete pile that is installed as a high-capacity unit carried down to and socketed into bedrock. These foundation units are formed by driving an open-ended, heavy-walled pipe to bedrock, cleaning out the pipe, and drilling a socket into the bedrock. A structural steel section (caisson core) is inserted, extending from the bottom of the rock socket to either the top or part way up the pipe. The entire socket and the pipe are then filled with concrete. The depth of the socket depends on the design capacity, the pipe diameter, and the nature of the rock.

1.2.5 Mandrel-driven tip—A mandrel-driven tip pile consists of an oversized steel-tip plate driven by a slotted, steel-pipe mandrel. This pile is driven through a hopper containing enough grout to form a pile the size of the tip plate. The grout enters the inside of the mandrel through the slots as the pile is driven and is carried down the annulus caused by the tip plate. When the required bearing is reached, the mandrel is withdrawn, resulting in a CIS shaft. Reinforcement can be lowered into the grout shaft before initial set of the grout. This pile differs from most CIS piles in that the mandrel is driven, not drilled, and the driving resistance can be used as an index of the bearing capacity.

1.2.6 Composite concrete piles—Composite concrete piles consist of two different pile sections, at least one of them being concrete. These piles have somewhat limited applications and are usually used under special conditions. The structural capacity of the pile is governed by the weaker of the pile sections.

A common composite pile is a mandrel-driven corrugated shell on top of an untreated timber pile. Special conditions that can make such a pile economically attractive are a required long length, an available inexpensive source of timber, a timber section that is positioned below the permanent water table, and a relatively low required capacity.

Another common composite pile is a precast section on top of a steel H- or pipe section, with a reinforced point where necessary. A CIP concrete pile constructed with a steel-pipe lower section and a mandrel-driven, thin corrugated-steel shell upper section is another widely used composite pile. The entire pile (shell and pipe sections) is filled with concrete, and reinforcing steel can be added as dictated by the design.

1.2.7 Drilled piles—Drilled piles are installed solely by drilling. Although driven piles can be predrilled, the final operation of their installation is driving.

1.2.7.1 Cast-in-drilled-hole piles—These piles, also known as drilled piers, are installed by mechanically drilling a hole to the required depth and filling that hole with reinforced or plain concrete. Sometimes, an enlarged tip is formed mechanically to increase the bearing area. A steel

liner is inserted in the hole where the sides of the hole are unstable. The liner may be left in place or withdrawn as the concrete is placed. In the latter case, precautions are required to be sure that the concrete shaft does not contain separations caused by the frictional effects of withdrawing the liner. For cast-in-drilled-hole piles 30 in. (760 mm) and larger in diameter, refer to ACI 336.1-01.

1.2.7.2 Foundation drilled piers or caissons—These are deep foundation units that often function like piles. They are essentially end-bearing units and are designed as deep footings combined with concrete shafts to carry the structure loads to the bearing stratum. This type of deep foundation is not covered in this report; for more information, refer to ACI 336.1-01 and ACI 336.3R-93.

1.2.7.3 Auger-grout or concrete-injected piles—Auger-grout piles are usually installed by turning a continuous-flight, hollow-stem auger into the ground to the required depth. As the auger is withdrawn, grout or concrete is pumped through the hollow stem, filling the hole from the bottom up. This CIS pile can be reinforced by a centered, full-length bar placed through the hollow stem of the auger, by reinforcing steel to the extent it can be placed into the grout shaft after completion, or both.

1.2.7.4 Drilled-displacement piles—Drilled-displacement piles are similar to auger-grout piles except that the augers are designed to displace portions of the penetrated soils laterally and to eliminate or minimize soil removed by the auger flights. As compared to auger-grout pile augers, drilled-displacement pile augers typically have a larger hollow-stem pipe, larger flight pitches, and an unflighted-displacement element or bulge to induce lateral-soil displacement. Dependent on the design of the auger flights and the section below the displacement bulge, the piles may be referred to as either full- or partial-displacement piles. As the auger is withdrawn, either grout or concrete is placed by gravity or pressure injection through the auger stem.

1.2.7.5 Drilled and grouted piles—These piles are installed by rotating a casing having a cutting edge into the soil, removing the soil cuttings by circulating drilling fluid, inserting reinforcing steel, pumping a sand-cement grout through a tremie to fill the hole from the bottom up, and withdrawing the casing. Such CIS piles are used principally for underpinning work or where low-headroom conditions exist, such as in basements or under bridges. These piles are often installed through an existing foundation.

1.2.7.6 Postgrouted piles—Concrete piles can have grout tubes embedded within them so that, after installation, grout can be injected under pressure to enhance the contact with the soil, to consolidate the soil under the tip, or both.

1.3—Design considerations

The successful design of a concrete pile foundation involves intimate knowledge of the relevant geotechnical and structural design requirements, pile manufacture and transportation details, and pile installation procedures. Suitable piles can be damaged by improper installation, so inspection and control of the pile installation are essential to producing a satisfactory foundation.

Improperly designed pile foundations can perform unsatisfactorily due to: 1) bearing capacity failure of the pile-soil system; 2) excessive settlement due to compression and consolidation of the underlying soil; or 3) structural failure of the pile shaft or its connection to the pile cap. In addition, improperly designed pile foundations could perform unsatisfactorily due to: 4) excessive settlement or bearing capacity failure caused by improper installation methods; 5) structural failure resulting from detrimental pile-installation procedures; or 6) structural failure related to environmental conditions.

Factors 1 through 3 are clearly design-related. Factors 4 and 5 are also design-related in that the designer can lessen these effects by providing adequate technical specifications and outlining proper inspection procedures to be used during the installation process. Factor 6 refers to environmental factors that can reduce the strength of the pile shaft during installation or during service life. The designer can consider environmental effects by careful selection of concrete materials, by selecting a pile section to compensate for future deterioration, using coatings or other methods to impede or eliminate the environmental effects, and implementing a periodic inspection and repair program to detect and correct structural deterioration. Hidden pile defects produced during installation can occur even if the pile design, manufacture, installation, and inspection appear to be flawless (Davisson et al. 1983). Proper inspection during manufacture and installation, however, can reduce the incidence of unforeseen defects. The design of the foundation system, preparation of the specifications, and inspection of pile installation should be a cooperative effort between the structural and geotechnical engineers.

A detailed discussion of the procedures for monitoring pile installation is beyond the scope of this report, although some items that the engineer might want to consider in determining the installation inspection procedures are noted throughout the text. For more detailed information on monitoring pile installation, the reader is referred to general references on pile inspection (Davisson 1972b; Fuller 1983).

In the design of any pile foundation, the nature of the subsoil and the interaction of the pile-soil system under service loads (Factors 1 through 3) usually control the design and are discussed in Chapters 3 and 4. Considerations relating to Factors 4 and 5 are covered in Chapter 8, although some guidance on these factors, as well as Factor 6, is offered in Chapters 3, 4, and 6 in connection with the preparation of adequate technical specifications. With reference to Factor 3, specific recommendations are given in Chapter 4 on providing a pile foundation of adequate structural capacity. The design procedures recommended are based on conservative values obtained from theoretical considerations, research data, and experience with in-service performance. Chapter 5 presents a general discussion of some geotechnical and structural design considerations that can be important when piles are used in regions of high seismicity.

A pile can be structurally designed and constructed to safely carry the design loads, but the pile cannot be considered to have achieved its required bearing capacity until it is

properly installed and functioning as a part of an adequate pile-soil system. Thus, in addition to its required design load structural capacity, driven piles have to be structurally capable of being installed to their required bearing capacity. This necessitates having one set of structural considerations for driving and another for normal service. Usually, under the most severe stress conditions, a pile will endure occur during driving.

Three limits to the load-bearing capacity of a pile can be defined: the first two are structural in nature, whereas the third depends on the ability of the subsoil to support the pile. First, the pile-driving stresses cannot exceed those that will damage the pile. This, in turn, limits the driving force of the pile against the soil and, therefore, the development of the soil's capacity to support the pile. Second, piles are designed to meet structural requirements under applicable loading conditions and codes, with consideration given to the lateral support conditions provided by the soil. Third, the soil should be able to support the pile loads with an adequate factor of safety against a soil-bearing capacity failure and with tolerable displacements. In static pile load tests carried to failure, it is usually the soil that gives way and allows the pile to penetrate into the ground; pile shaft failures, however, can also occur. All three of these limits should be satisfied in a proper pile design.

CHAPTER 2—NOTATION AND DEFINITIONS

2.1—Notation

A	= pile cross-sectional area, in. ² (mm ²)	f_{ys}	= yield stress of steel shell, psi (MPa)
A_c	= area of concrete (including prestressing steel), in. ² (mm ²)	f_{yt}	= yield stress of transverse reinforcement, psi (MPa)
	= $A_g - A_{st}$, for reinforced concrete piles, in. ² (mm ²)	g	= acceleration of gravity, in./s ² (m/s ²)
A_{core}	= area of core of section, to outside diameter of the spiral steel, in. ² (mm ²)	h_c	= cross-sectional dimension of pile core, center-to-center of hoop reinforcement, in. (mm)
A_g	= gross area of pile, in. ² (mm ²)	I	= moment of inertia of the pile section, in. ⁴ (mm ⁴)
A_{ch}	= area of core within transverse reinforcement, measured out-to-out of the reinforcement, in. ² (mm ²)	I_g	= moment of inertia of the gross pile section, in. ⁴ (mm ⁴)
A_p	= area of steel pipe or tube, in. ² (mm ²)	K	= horizontal subgrade modulus for cohesive soils, psi (N/mm ²)
A_{ps}	= area of prestressing steel, in. ² (mm ²)	K	= coefficient for determining effective pile length
A_{sh}	= total area of transverse reinforcement in direction considered, in. ² (mm ²)	L	= pile length, in. (mm)
A_{sp}	= area of spiral or tie bar, in. ² (mm ²)	L_s	= depth below ground surface to point of fixity, in. (mm)
A_{st}	= total area of longitudinal reinforcement, in. ² (mm ²)	L_u	= length of pile above ground surface, in. (mm)
b_c	= width of section in direction considered, in. (mm)	ℓ_e	= effective pile length = $K\ell_u$, in. (mm)
D	= steel shell diameter, in. (mm)	ℓ_u	= unsupported structural pile length, in. (mm)
d_{core}	= diameter of core section, to outside of spiral, in. (mm)	M	= pile moment, in.-lb (N-mm)
E	= modulus of elasticity for pile material, psi (MPa = N/mm ²)	$(N_1)_{60}$	= standard penetration test N -value scaled to a standard hammer efficiency of 60 percent and to a standard effective-overburden pressure of 1 ton/ft ² (96 kPa)
EI	= flexural stiffness of the pile, lb-in. ² (N-mm ²)	n_h	= coefficient of horizontal subgrade modulus, lb/in. ³ (N/mm ³)
f'_c	= specified compressive strength of concrete, psi (MPa)	P	= axial test load on pile, lb (N)
f_{pc}	= effective prestress in concrete after losses, psi (MPa)	P_a	= allowable axial compression service capacity, lb (N)
f_{ps}	= stress in prestressed reinforcement at nominal strength of member, psi (MPa)	P_{at}	= allowable axial tension service capacity, lb (N)
f_y	= yield stress of nonprestressed reinforcement, psi (MPa)	P_u	= factored axial load on pile, lb (N)
f_{yh}	= yield stress of transverse spiral or tie reinforcement, psi (MPa)	R	= radius of gyration of gross area of pile, in. (mm)
f_{yp}	= yield stress of steel pipe or tube, psi (MPa)	R	= relative stiffness factor for preloaded clay, in. (mm)
		s	= spacing of tie sets along length of member, in. (mm)
		s_u	= undrained shear strength of soil, lb/ft ² (kPa = kN/m ²)
		s_{sp}	= spacing of hoops or pitch of spiral along length of member, in. (mm)
		T	= relative stiffness factor for normally loaded clay, granular soils, silt, and peat, in. (mm)
		t_{shell}	= wall thickness of steel shell, in. (mm)
		ρ_s	= ratio of volume of spiral reinforcement to total volume of core (out-to-out of spiral)
		ϕ	= strength reduction factor
		ϕ_c	= strength reduction factor in compression
		ϕ_t	= strength reduction factor in pure flexure, flexure combined with tension, or pure tension

2.2—Definitions

ACI provides a comprehensive list of definitions through an online resource, “ACI Concrete Terminology” (<http://terminology.concrete.org>).

CHAPTER 3—GEOTECHNICAL DESIGN CONSIDERATIONS

3.1—General

In the design of any pile foundation, the nature of the subsoil, the installation means and methods, and the interaction of the pile-soil system under service loads usually control the allowable pile load capacity. This report does not cover in detail the principles of soil mechanics and behavior as they can affect pile-foundation performance. This chapter does include, however, a general discussion of the important geotechnical considerations related to the proper

design of pile foundations. For more detailed information on geotechnical considerations, refer to general references on soil mechanics and pile design (ASCE/SEI 7-05; NAVFAC 1982; Peck et al. 1974; Prakash and Sharma 1990; Terzaghi et al. 1996).

3.2—Subsurface conditions

Knowledge of subsurface conditions and their effect on the pile-foundation design and installation is essential. This knowledge can be obtained from various sources, including prior experience in the geographical area, performance of existing foundations under similar conditions, knowledge of geological formations, geological maps, soil profiles exposed in open cuts, and exploratory borings with or without detailed soil tests. From such information, along with knowledge of the structure to be supported and the character and magnitude of loading (for example, column load and spacing), it is often possible to make a preliminary choice of pile type(s), length(s), and pile design load(s).

On some projects, existing subsurface data and prior experience can be sufficient to complete the final foundation design, with pile driving proceeding on the basis of penetration resistance, depth of embedment, or both. On other projects, extensive exploration and design-stage pile testing can be required to develop final design and installation requirements.

Subsurface exploration cannot remove all uncertainty about subsurface conditions on projects with pile foundations. Additional data on the actual extent of vertical and horizontal subsoil variations at a particular site can be obtained from field observations during production pile installation. Subsurface information collected by the designer for use in developing the design and monitoring pile installation is often inadequate.

A common result of inadequate subsurface exploration is pile-tip elevations that fall below the depth of the deepest exploration. This situation often occurs because a pile foundation was not considered when exploration started. Whereas deeper exploration will not prevent problems from developing during construction in all cases, information from such explorations can be valuable in determining corrective options for solving those problems that do develop. The additional cost of deeper exploration during the design stage is trivial compared with the cost of a construction delay required to obtain additional subsoil information on which to base a decision.

Inadequate subsurface exploration of another nature often develops when the decision to use a pile foundation is made early in the design process. In such cases, there often is a tendency to perform detailed exploration of a preconceived bearing stratum while obtaining only limited data on the overlying strata that the piles have to penetrate. This practice is detrimental because design parameters, such as negative skin friction, are dependent on the properties of the overlying strata. Furthermore, a shortage of information on the overlying strata can also lead to judgment errors by both the designer and the contractor when assessing installation problems associated with penetrating the overlying strata

and evaluating the type of reaction system most economical for performing static load tests.

Test borings should be made at enough locations and to a sufficient depth below the anticipated tip elevation of the piles to provide adequate information on all materials that will affect the foundation construction and performance. The results of the borings and soils tests, taken into consideration with the function of the piles in service, will assist in determining the type, spacing, and length of piles that should be used and how the piles will be classified (for example, end-bearing piles, friction piles, or a combination of both types).

3.2.1 End-bearing piles—A pile can be considered end bearing when it passes through soil having low frictional resistance and its tip rests on rock or is embedded in a material of high resistance to further penetration so that the load is primarily transmitted to the soil at or close to the pile tip. The capacity of end-bearing piles depends on the bearing capacity of the soil or rock underlying the piles and the structural capacity of the pile shaft. Settlement of piles is controlled primarily by the compression of materials beneath the pile tips.

3.2.2 Friction bearing piles—A friction pile derives its support from the surrounding soil, primarily through the development of shearing resistance along the sides of the pile with negligible shaft loads remaining at the tip. The shearing resistance can be developed through friction, as implied, or it may actually consist of adhesion. The load capacity of friction piles depends on the ability of the soil to distribute pile loads to the soil beneath the pile group within the tolerable limits of settlement of the supported structure.

3.2.3 Combined friction and end-bearing piles—Combined friction and end-bearing piles distribute the pile loads to the soil through both shear along the sides of the pile and bearing on the soil at the pile tip. In this classification, both the side resistance and end-bearing components are of sufficient relative magnitude that one of them cannot be ignored.

3.3—Bearing capacity of individual piles

A fundamental design requirement of all pile foundations is that they are designed to carry the design service load with an adequate factor of safety against a bearing capacity failure. Usually, designers determine the factor of safety against a bearing capacity failure that is required for a particular project, along with the foundation loads, pile type(s) and size(s) to be used, and an estimate of the pile lengths likely to be required. Design should consider the behavior of the entire pile foundation over the life of the structure. Conditions that should be considered beyond the bearing capacity of an individual pile during the relatively short-term installation process are group behavior, long-term behavior, and settlement.

Project specifications prescribe ultimate bearing-capacity requirements, installation procedures for individual piles, or both, to control the actual construction of the foundations. Therefore, during construction of the pile foundation, the designer generally exercises control based on the bearing capacity of individual piles as installed.

An individual pile fails in bearing when the applied load on the pile exceeds both the ultimate shearing resistance of the soil along the sides of the pile and the ultimate resistance of the soil underneath the pile tip. The ultimate bearing capacity of an individual pile can be determined most reliably by static load testing to failure.

Commonly used methods to evaluate the bearing capacity of the pile-soil system include static pile load testing, observed resistance to penetration for driven piles, and static-resistance analyses. The resistance-to-penetration methods include dynamic driving formulas, analyses based on the one-dimensional wave equation, and analyses that use measurements of dynamic strain and acceleration near the pile head during installation. Careful judgment of an engineer qualified in the design and installation of pile foundations is required when using these methods. Frequently, two or more of these methods are used to evaluate bearing capacity of individual piles during design and construction. For example, static load tests to failure (or proof-load tests to some multiple of the design load) may be performed on only a few piles, with the remaining production piles being evaluated on the basis of a resistance-to-penetration method, calibrated against the static load test results.

The design factor of safety against bearing capacity failure of individual piles for a particular project depends on many variables, such as:

- The type of structure and the implications of failure of an individual pile on the behavior of the foundation.
- Building code provisions concerning the load reductions applied (for example, loaded areas) in determining the structural loads applied to the foundations, or overload allowed for wind and earthquake conditions.
- The reliability of methods used to evaluate bearing capacity.
- The reliability of methods used to evaluate pile service loads.
- The construction control applied during installation.
- The changes in subsoil conditions that can occur with the passage of time.
- The manner in which soil-imposed loads, such as negative skin friction, are introduced into the factor of safety calculations.
- The variability of the subsoil conditions at the site.
- Effects of pile-location tolerances on pile service load.

In general, the design factor of safety against a bearing capacity failure should not be less than 2. Consideration of the previously stated variables could lead to the use of a higher factor of safety. When the pile capacity is based on analysis and not proven by static load tests, the design factor of safety should be higher than used when piles are subjected to static load tests.

3.3.1 Load testing—Static pile load tests remain the most reliable tool for the geotechnical design of pile foundations. Static pile load tests may be performed before the final foundation design, in conjunction with the actual pile foundation installation, or both. Tests performed during the design stage can be used to develop site-specific parameters for final design criteria; make economical and technical comparisons

of various pile types and design loads; verify preliminary design assumptions, including comparisons or top-of-pile movements measured in the tests with those predicted by the structural analyses; evaluate special installation methods required to reach the desired bearing strata and capacity; and develop installation criteria.

Tests performed as a part of production-pile installation are intended to verify final design assumptions, establish installation criteria, satisfy building code requirements, develop quality control of the installation process, and obtain data for evaluating unanticipated or unusual installation behavior.

Piles that are statically tested in conjunction with actual pile construction to meet building code requirements, and for quality control, are generally proof-loaded to two times the design service load. Where practical, particularly for tests performed before final design, pile load tests should be carried to soil-bearing failure so that the true ultimate bearing capacity can be determined for the test conditions. Knowing the ultimate bearing capacity of each type of pile tested can lead to a safer or more economical redesign. With known failure loads, the test results can be used to calibrate other analytical tools used to evaluate individual pile-bearing capacity in other areas of the project site where static load tests have not been performed. Furthermore, knowledge of the failure loads aids evaluation of driving equipment changes and any changes in installation or design criteria that can be required during construction.

Sufficient subsoil data (refer to 3.2) should be available to disclose dissimilarities between soil conditions at the test-pile locations and other areas where piles are to be driven. The results of a load test on an individual pile can be applied to other piles within an area of generally similar soil conditions, provided that the piles are of the same type and size and are installed using the same or equivalent equipment, methods, and criteria as that established by the pile test. For a project site with generally similar soil conditions, enough tests should be performed to establish the variability in capacity across the site. If a construction site contains dissimilar soil conditions, pile tests should be conducted within each area of generally similar subsoil conditions, or in the least favorable locations, if the engineer can make this distinction.

The results of a load test on an individual pile are strictly applicable only at the time of the test and under the conditions of the test. Several aspects of pile-soil behavior can cause the soil-pile interaction in the completed structure to differ from that observed during a load test on an individual pile. Some of these considerations are discussed in Sections 3.3.5 through 3.3.8, 3.4 through 3.7, and 3.10. On some projects, special testing procedures might be warranted to obtain more comprehensive data for use in addressing the influence of these considerations on the pile performance under load. These special procedures can include:

- Isolating the pile shaft from the upper nonbearing soils so that the pile capacity is determined within the bearing material.
- Instrumenting the pile with strain rods (telltales) or gauges to determine the distribution of load along the pile shaft.

- Testing piles driven both into and just short of an end-bearing stratum to evaluate the shear resistance in the overlying soil as well as the capacity in the bearing stratum.
- Performing uplift tests in conjunction with downward compression tests to determine distribution of pile load capacity between friction and end-bearing.
- Casting jacks or load cells in the pile tip to determine distribution of pile load capacity between friction and end-bearing.
- Cyclic loading to estimate soil resistance distribution between friction and end-bearing.

Where it is either technically or economically impractical to perform such special tests, analytical techniques and engineering judgment, combined with higher factors of safety where appropriate, should be used to evaluate the impact of these various considerations on the individual pile-test results. In spite of the potential dissimilarities between a single pile test and pile foundation behavior, data from static load tests on individual piles offer the most reliable information for determining the bearing capacity of a single pile under the tested conditions and for monitoring the installation of pile foundations.

Many interpretation methods have been proposed to estimate the failure load from static load test results. Numerous procedures or building code criteria are also used to evaluate the performance of a pile under static test loading. The test loading procedures and duration required by the various interpretation methods are also highly variable.

Acceptance criteria for the various methods are often based on allowable gross pile-head deflection under the full test load, net pile-head deflection remaining after the test load has been removed, or pile-head deflections under the design load. Sometimes, the allowable deflections are specified as definite values, independent of pile width, length, or magnitude of load. In other methods, the permissible displacements can be dependent on only the load or (in the more rational methods) on pile type, width, length, and load. Some methods define failure as the load at which the slope of the load-deflection curve reaches a specified value or requires special testing or plotting procedures to determine yield load. Other methods use vague definitions of failure such as “a sharp break in the load-settlement curve” or “a disproportionate settlement under a load increment.” The scales used in plotting the test results and the size and duration of the load increments can greatly influence the failure loads interpreted using such criteria. These criteria for evaluating the satisfactory performance of a test pile represent arbitrary definitions of the failure load, except where the test pile exhibits a definite plunging into the ground. Some definitions of pile failure in model building codes are too liberal when applied to high-capacity piles. For example, the method that allows a net settlement of 0.01 in./ton (0.029 mm/kN) of test load might be adequate if applied to low-capacity piles, but the permitted net settlements exceed acceptable tolerances when applied to high-capacity piles.

This report does not present detailed recommendations for the various methods for load-testing piles, the methods

and instrumentation used to measure pile response under load test, or the methods of load test interpretation. ASTM D1143/D1143M-09, D3689-07, D3966-07, and Davisson (1970a, 1972a) discuss these items. Building codes usually specify how load tests should be performed and analyzed. When the method of analysis is selected by the engineer, however, it is recommended that the method proposed by Davisson for driven piles be used. Davisson’s method defines pile failure as the load at which the pile-head settlement exceeds the pile elastic compression by 0.15 in. (4 mm) plus 0.83 percent of the pile width, where the pile elastic compression is computed by means of the expression PL/AE (Davisson 1972a; Peck et al. 1974). Davisson’s criterion is too restrictive for drilled piles, unless the resistance is primarily friction, and engineers will have to use their own judgment or modification.

3.3.2 Resistance to penetration of piles during driving—

A pile foundation generally has so many piles that it would be impractical to load- or proof-test them all. It is necessary to evaluate the bearing capacity of piles that are not tested on the basis of the pile-driving record and the resistance to penetration during installation. Final driving resistance is usually weighted most heavily in this evaluation.

Driving criteria based on resistance to penetration are of value and often indispensable in ensuring that all piles are driven to relatively uniform capacity. This will minimize possible causes of differential settlement of the completed structure due to normal variations in the subsurface conditions within the area of the pile-supported structures. In effect, adherence to an established driving resistance tends to permit each pile to seek its own length to develop the required capacity, thus compensating for the natural variations in depth, density, and quality of the bearing strata.

For over a century, engineers have tried to quantify the relationship between the ultimate bearing capacity of a pile and the resistance to penetration observed during driving. Earlier attempts were based on energy methods and Newtonian theory of impact (3.3.2.1). The shortcomings of dynamic pile-driving formulas have long been known (Cummings 1940), but they still appear in building codes and specifications. The agreement between static ultimate bearing capacity and the predicted capacity based on energy formulas are in general so poor and erratic that their use is not justified except under limited circumstances where the use of a particular formula is justified by prior load tests and experience in similar soil conditions with similar piles and driving assemblies (Olson and Flaate 1967; Terzaghi et al. 1996).

Cummings (1940) suggested that the dynamics of pile driving be investigated by wave-equation analysis. Computers have made the one-dimensional wave-equation analysis of pile driving an indispensable tool for the foundation engineer (3.3.2.2). Field instrumentation that measures and records shaft strain and acceleration near the pile top is available and has prompted attempts to predict the ultimate bearing capacity using these measurements (3.3.2.3).

Although the development of the wave-equation analysis and methods based on strain and acceleration measurements represent a vast improvement over the fundamentally

unsound dynamic formulas, these refined methods are not reliable substitutes for pile load tests (Selby et al. 1989; Terzaghi et al. 1996). Some driving and soil conditions defeat all of the geotechnical engineer's tools except the static load test (Davisson 1989; Prakash and Sharma 1990). Such problems have occurred with the wave equation as well as with methods based on dynamic measurements (Davisson 1991; Terzaghi et al. 1996).

Despite their shortcomings, resistance-to-penetration methods of estimating bearing capacity based on the wave equation remain a valuable tool because of the impracticality of testing all piles on a project, their use as a design tool for evaluating the pile driveability and driving stresses, and their use in equipment selection. Static load tests are still needed to confirm bearing capacity and calibrate the penetration-resistance method used to extend quality control over the remaining piles. In some instances, the increased use of dynamic measurements has actually been associated with an increase in the frequency of performing static load tests because such load test data are required to calibrate capacity predictions (Schmertmann and Crapps 1994).

3.3.2.1 Dynamic formulas—Piles are long members, with respect to their width, and do not behave as rigid bodies. Under the impact from a hammer, time-dependent stress waves are set up in the pile and surrounding soil. All of the dynamic formulas ignore the time-dependent aspects of stress-wave transmission and are therefore fundamentally unsound.

The term “dynamic formula” is misleading, as it implies a determination of the dynamic capacity of the pile. Such formulas have actually been developed to reflect the static capacity of the pile-soil system as measured by the dynamic resistance during driving. This is also true of the wave-equation analysis and methods based on strain and acceleration measurements (3.3.2.2 and 3.3.2.3). Under certain subsoil conditions, penetration resistance as a measure of pile capacity can be misleading in that it does not reflect soil phenomena such as relaxation or freeze (3.3.5), which can either reduce or increase the final static pile-soil capacity.

Dynamic formulas, in their simplest form, are based on equating the energy of a hammer blow to the work done as the pile moves a distance (set) against the soil resistance. The more complicated formulas also involve Newtonian impact principles and other attempts to account for the many individual energy losses within the hammer-capblock-pile-soil system. These formulas are used to determine the required resistance to penetration (blows per inch [mm]) for a given load or to determine the load capacity based on a given penetration resistance or set.

Some dynamic formulas are expressed in terms of ultimate pile capacity, whereas others are expressed in terms of allowable service capacity. All dynamic formulas are empirical and provide different safety factors, often of unknown magnitude. In general, such formulas are more applicable to noncohesive soils. The applicability of a formula to a specific pile-soil system and driving conditions can be evaluated by load tests to failure on a series of piles.

Dynamic formulas have been used successfully when applied with experience and judgment, and with proper recognition of their limitations. Because the formulas are fundamentally unsound, however, there is no reason to expect that the use of a more complicated formula will lead to more reliable predictions, except where local empirical correlations are known for a given formula under a given set of subsurface conditions.

When pile capacity is to be determined by a dynamic formula, the required penetration resistance should be verified by pile load tests, except where the formula has been validated by prior satisfactory experience for the type of pile and soil involved. Furthermore, such practices should be limited to relatively low pile capacities. Attempts to use empirical correlations for a dynamic formula determined for a given pile type and site condition with other pile types and different site conditions can lead to either ultraconservative or unsafe results.

3.3.2.2 Wave-equation analysis—The effects of driving a pile by impact can be described mathematically according to the laws of wave mechanics (Isaacs 1931; Glanville et al. 1938). Cummings (1940) discussed the defects of the dynamic formulas that do not consider the time-dependent aspects of stress-wave transmission and pointed out the merits of using wave mechanics in making a rational analysis of the pile-driving process.

Early developments in application of the wave-equation analysis to pile driving were advanced by Smith (1951, 1955, 1962). The advent of high-speed digital computers permitted practical application of wave-equation analysis to pile equipment design and the prediction of pile-driving stress and static pile capacity. The first publicly available digital computer program was developed at Texas A&M University (Edwards 1967).

Since that time, wave-equation analysis has taken its place as a standard tool used in pile foundation design and construction control. Through the sponsorship of the Federal Highway Administration, wave-equation programs are readily available through public sources (Goble and Rausche 1976, 1986; Hirsch et al. 1976), as well as from several private sources. Today, with both wave-equation analysis software and computer hardware readily available to engineers, there is no reason to use dynamic formulas.

The one-dimensional wave equation mathematically describes the longitudinal-wave transmission along the pile shaft from a concentric blow of the hammer (Edwards 1967; Hirsch et al. 1970; Lowery et al. 1968, 1969; Mosley and Raamot 1970; Samson et al. 1963; Smith 1951, 1955, 1962). Computer programs can take into account the many variables involved, especially the elastic characteristics of the pile. The early programs were deficient in their attempts to model diesel hammers. Research (Davisson and McDonald 1969; Goble and Rausche 1976, 1986; Rempe 1975; Rempe and Davisson 1977) has demonstrated that it is essential that the effects of gas forces and the steel-on-steel impact, which occurs when the ram contacts the anvil, be included in the wave equation model for a diesel hammer.

both of these effects will result in an invalid evaluation of the dynamic pile behavior.

In wave-equation analysis of pile driving, an ultimate pile capacity (lb or N) is assumed for a given set of conditions, and the program performs calculations to determine the net set, or downward movement (in. or mm) of the pile. The reciprocal of the set is the driving resistance, usually expressed in hammer blows per inch (mm) of pile penetration. The analysis also predicts the pile shaft forces as a function of time after impact, which can be transformed to the driving stresses in the pile cross section. The process is repeated for several ultimate resistance values. From the computer output, a curve showing the relationship between the ultimate pile capacity and the penetration resistance can be plotted. The maximum calculated tensile and compressive stresses can also be plotted as a function of either the penetration resistance or the ultimate load capacity. In the case of diesel hammers and other variable-stroke hammers, the analysis is performed at several different strokes, or equivalent strokes in the case of closed-top diesel hammers, to cover the potential stroke range that might develop in the field.

Although results are applicable primarily to the set of conditions described by the input data, interpolations and extrapolations for other sets of conditions can be made with experience and judgment. Routine input data describing the conditions analyzed include such parameters as:

- Hammer ram weight.
- Hammer stroke.
- Stiffness and coefficient of restitution of the hammer cushion (and pile cushion if used).
- Drive head weight.
- Pile type, material properties, dimensions, weight, and length.
- Soil quake and damping factors.
- Percentage of pile capacity developed by friction and end bearing.
- Distribution of frictional resistance over the pile length.

With diesel hammers, the model should deal with the effects of both gas force on the hammer output and the steel-on-steel impact that occurs as the ram contacts the anvil, as noted previously.

Wave-equation analysis is a reliable and rational tool for evaluating the dynamics of pile driving and properly takes into account most of the factors not included in the other dynamic formulas (3.3.2.1). Although wave-equation analysis is based on the fundamentally sound theory of one-dimensional wave propagation, it is still empirical. The primary empirical content is the input parameters and mathematical model for the soil resistance. Fortunately, the simple mathematical soil model and empirical coefficients proposed by Smith (1951, 1955, 1962) appear to be adequate for approximating real soil behavior in a wide variety of, but not all, driving conditions.

Except for conditions where unusually high soil quake or damping are encountered, a wave-equation analysis coupled with a factor of safety of 2 can generally provide a reasonable driving criterion, provided proper consideration is given to the possible effect of soil freeze or relaxation (3.3.5). When the

required pile penetration resistance is determined by a wave-equation analysis, the results of such analysis and the pile capacity should be verified by static load tests. With pile load tests carried to failure, adjustments in the soil-input parameters can be made, if necessary, to calibrate the wave equation for use at a given site. Information from dynamic measurements and analysis (3.3.2.3) can also assist in refining input to the wave-equation analysis concerning hammer, cushion, pile, and soil behavior.

The wave equation is an extremely valuable design tool because the designer can perform analyses during the design stage of a pile foundation to evaluate both pile drivability and pile-driving stresses for the various stages of installation. These results aid in making design decisions on pile-driving equipment for the pile section ultimately selected and ensuring that the selected pile can be installed to the required capacity at acceptable pile-driving stress levels. For precast piles, the analysis is helpful for selecting the hammer and pile cushioning so that the required pile load capacity can be obtained without damaging the pile with excessive driving stresses (Davisson 1972a). Such analyses are also useful in estimating the amount of tension, if any, throughout the pile length as well as at proposed splice locations. This is especially important in the case of precast and prestressed piles that are much weaker in tension than in compression. A drivability study can be used to aid in developing design and specification provisions related to equipment selection and operating requirements, cushioning requirements, reinforcing or prestressing requirements, splice details, and preliminary driving criteria. Therefore, it is possible to design precast and prestressed piles with greater assurance that driving tensile and compressive stresses will not damage the pile.

The wave-equation analysis, however, does not predict total pile penetration (pile embedment). Procedures for pile penetration prediction have been presented previously.

3.3.2.3 Dynamic measurements and analysis—Instrumentation and equipment are available for making measurements of dynamic strains and accelerations near the pile head as a pile is being driven or re-struck. Procedures for making the measurements and recording the observations are covered in ASTM D4945-08.

The measured data, when combined with other information, can be used in approximate analytical models to evaluate dynamic pile-driving stresses, structural integrity, static bearing capacity, and numerous other values blow-by-blow while the pile is being driven (Rausche et al. 1972, 1985). Subsequently, the recorded information can be used in more exact analyses (Rausche 1970; Rausche et al. 1972, 1985) that yield estimates of both pile bearing capacity and soil-resistance distribution along the pile. Determination of static pile capacity from the measurements requires empirical input and is dependent on the engineering judgment of the individual performing the evaluation (ASTM D4945-08; Fellenius 1988). The input into the analytical models may or may not result in a dynamic evaluation that matches static load test data. It is desirable and may be necessary to

brate the results of the dynamic analysis with those of a static pile load test (ASTM D4945-08, refer to 3.3.1 of this report).

Dynamic measurements and analyses can provide design information when site-specific dynamic measurements are obtained in a pile-driving and load-testing program undertaken during the design phase of a project. Without such a test program, the designer has to decide on the type of pile, size of pile, and the pile-driving equipment relying on other techniques and experience. The wave-equation analysis is a useful design tool that helps provide information leading to the necessary design decisions (3.3.2.2). Dynamic measurements and analyses find use in the verification of the original design and development of final installation criteria after production pile driving commences. The ability to make dynamic measurements is a useful addition to the geotechnical engineer's resources when properly used. There are, however, limitations to the use of this method in determining static pile load capacity and these methods are not a reliable substitute for pile load tests (Selby et al. 1989; Terzaghi et al. 1996).

Specialized equipment and test procedures have been developed that can subject deep-foundation members to a force pulse with a duration that is significantly longer than the short impact force durations a pile experiences during pile driving. Pile-head forces and displacements are measured and recorded during the test. Pile-head accelerations are also generally recorded as a backup for the displacement measurement. The specialized equipment for the force-pulse test may consist of a drop hammer with a special cushion element between the ram and the pile head that moderates the blow and lengthens the duration of the impact event, or the pulse may be induced by an explosive force used to accelerate a reaction mass. The hammer mass, drop, and cushioning, or the explosives and reaction mass, are specially designed to achieve the desired peak-force level and pulse duration. In addition to the procedures for performing the test provided in ASTM D7383-10, information on the method and some of the procedures proposed for evaluating the test results are provided by Middendorp et al. (1992), Mullins et al. (2002), and Kusakabe et al. (2000). Although the force-pulse duration of this test method (typically 0.1 to 0.2 seconds) is considerably longer than the typical force-pulse duration (approximately 0.02 seconds) during pile-impact driving, it is still a dynamic test and mobilizes both damping forces and soil resistance forces. Hence, use of the test to estimate static pile-load capacity encounters some of the same difficulties discussed previously for methods using dynamic strain and acceleration data measured during pile driving, and it is not a reliable substitute for static pile-load tests (refer to 3.3.1).

3.3.3 Static-resistance analysis—The application of static analysis uses various soil properties determined from laboratory and field tests, or as assumed from soil boring data. The pile capacity is estimated by applying the shearing resistance (friction or adhesion) along the embedded portion of the pile and adding the bearing capacity of the soil at the pile end. Such analyses should reflect the effects of pile taper, cross-sectional shape (square or round) and surface texture, the compaction of loose granular soils by driving displace-

ment-type piles, and the effects of the installation methods used. Each of these factors can have an influence on the final load-carrying capacity of a pile (Nordlund 1963).

Progress has been made in understanding how soil properties affect pile capacity and numerous static pile design procedures have been proposed. Statistical comparisons of predicted and measured pile capacities (Olson and Dennis 1982; Olson 1984; Olson and Long 1989) suggest that the factors of safety required with such methods to provide a reasonable margin of safety against pile bearing failure are significantly greater than the factor of safety of 2 normally used with static load tests. The margin of uncertainty with static-capacity predictions indicate that site-specific load testing remain an integral part of the design process for driven piles and drilled shafts (O'Neill 2001). When pile length is selected on the basis of experience or static-resistance analysis, static load tests should be performed to verify such predictions.

3.3.4 Rock sockets for drilled-in caissons—The design of drilled-in caissons (1.2.4) requires the determination of an adequate rock socket for the working loads involved. The design of the rock socket is usually based on the peripheral bond between the concrete filling and the rock. If the socket can be thoroughly cleaned out and inspected, and the concrete can be placed in the dry, it may be possible to use a combination of both end bearing and bond to develop the required load. The combined use of both end bearing and bond, however, may not be permitted by the applicable building code.

3.3.5 Relaxation and soil freeze—In some soils, pile-bearing capacity can decrease after initial pile installation as a result of a phenomenon commonly referred to as relaxation. Relaxation is evidenced by a reduction in penetration resistance after initial driving and could be accompanied by a loss of bearing capacity. In other soil profiles, pile capacity increases after initial installation. This phenomenon is commonly referred to as freeze and is associated with regain of strength of soils after being disturbed during the driving process. Although soil freeze and relaxation are generally associated with driven piles, these phenomena can also occur in piles installed by jacking. If soil relaxation or freeze occurs, the final penetration resistance during initial driving of the pile is generally not an indication of the actual pile static capacity. In such cases, dynamic methods of capacity prediction (3.3.2) will not produce valid results without modifications based on a load test or redriving results.

The possibility of these phenomena should be recognized by the designer when establishing such requirements as type of pile, pile length, and driving resistance. Relaxation can be checked by re-driving some piles several hours after initial installation to determine if the driving resistance has been maintained. Soil freeze can also be checked by re-driving, but load testing is more positive. Sufficient time should be allowed before testing to permit the soil strength to be regained. This required time could range from a few hours to as long as 30 days. Re-tapping of piles produces more valid information if the hammer-cushion-pile system is driven as for initial driving.

3.3.6 Compaction—Many soils are compacted and densified through the process of pile driving, especially when displacement-type piles are installed without pre-excitation such as jetting or predrilling. The soil strength properties are usually increased, although the extent and degree to which they will increase are not easy to predict. Compaction is usually progressive as more piles are driven within a group. Installation sequence or methods should be controlled to prevent extreme variations in pile lengths due to ground compaction (8.2.6 and 8.2.7).

3.3.7 Liquefaction—Liquefaction is usually associated with earthquake or large vibratory forces combined with liquefiable granular soils. This can result in loss of pile capacity, lateral support, or both. Although it is not generally necessary to consider this in normal pile foundation design, it is necessary to consider liquefaction in seismically active regions. Liquefaction during seismic events is discussed further in Chapter 5.

Some soils exhibit temporary liquefaction during pile driving with corresponding reduction in penetration resistance. The reestablishment of the soil resistance can be detected by re-driving the pile, but under severe conditions where re-driving immediately creates liquefaction, the capacity of the pile may have to be determined by static load testing.

3.3.8 Heave and flotation—Pile heave is the upward movement of a previously driven pile caused by the driving of adjacent piles. The designer should be alert to possible pile heave, include provisions in the specification to check for this phenomenon, and take precautionary measures. Heave of friction piles may have no detrimental effect on pile-soil capacity, but can affect the structural capacity of the pile if it is weak in tension.

Heave can take place when driving piles through upper cohesive soils that do not readily compress or consolidate during driving. Under severe conditions, heave is quite evident from the upward movement of the ground surface. When heave conditions exist, elevation checks should be taken on the tops of the driven piles. Such level readings can be taken on the tops of pile casings that cannot elongate significantly. For spirally corrugated pile shells (that is, shells with corrugations aligned similarly to a spiral reinforcing hoop), check levels should be made on pipe telltales bearing on the pile tips, because heave that causes only shell stretch should not affect the pile capacity.

Heave can often be limited or even eliminated by pile pre-excitation or increasing the pile spacing. Wet rotary pre-excitation methods can also cause ground heave if the recirculation pressures exceed the overburden pressure (Ray et al. 1979). The shells for CIP concrete piles should be left unfilled until the pile-driving operation has progressed beyond the heave range. CIS concrete piles and sectional concrete piles having joints that cannot take tension should not be used under heave conditions unless positive measures are taken to prevent heave.

If pile heave occurs, the unfilled shells or casings for CIP concrete piles and most precast concrete piles can be redriven to compensate for heave. CIS concrete piles containing full-

length reinforcement can be subjected to a limited amount of re-driving to reseal the pile. CIS concrete piles without internal reinforcement should be abandoned if heaved. Sectional precast concrete piles having slip-type joints can be redriven to verify that they are sound and that the joints are closed. In the case of sectional piles, however, all of the heave should be considered to have occurred at a single joint and the joint should not have been opened completely as a result of pile heave. If necessary, CIP piles can be redriven to compensate for heave after the shell is filled with concrete, if proper techniques are used. A wave-equation analysis can be used to aid in the design of the hammer-cushion combination required for such redriving.

Flotation can occur when pile shells or casings are driven in fluid soils and a positive buoyancy condition exists. Check elevations should be made as for heave, and the piles redriven if required. It may be necessary to create negative buoyancy or use some means to hold the piles down until the casings are filled with concrete.

3.4—Settlement

The investigation of the overall pile foundation design for objectionable settlement involves the soil properties and the ability of the soil to carry the load transferred to it without excessive consolidation or displacement, which in time could cause settlements beyond that for which the structure is designed. The soils well below the pile tips can be affected by loading, and such effects vary with the magnitude of load applied and the duration of loading. Many of the design considerations discussed in this chapter relate to the evaluation of settlement. The soil mechanics involved are beyond the scope of this guide. The long-term settlement of a pile foundation under service loading is not the same as the settlement observed in a short-term static load test on an individual pile (3.3.1).

3.5—Group action in compression

The bearing capacity of a pile group consisting of end-bearing piles or piles driven into granular strata at normal spacing (3.6) can be considered to be equal to the sum of the bearing capacities of the individual piles. The bearing capacity of a friction pile group in cohesive soil should be checked by evaluating the shear strength and bearing capacity of the soil, assuming that the pile group is supported by shear resistance on the periphery of the group and by end bearing on the base area of the group. The use of group reduction formulas based on spacing and number of piles is not recommended.

3.6—Pile spacing

Pile spacing is measured from center to center. The minimum recommended spacing is three times the pile diameter or width at the cutoff elevation. Several factors should be considered in establishing pile spacing. For example, the following considerations might necessitate an increase in the normal pile spacing:

- a) For piles deriving their principal support from friction.

- b) For extremely long piles, especially if they are flexible, to reduce interference with adjacent piles.
- c) For CIS concrete piles where pile installation could damage adjacent unset concrete shafts.
- d) For piles carrying very high loads.
- e) For piles that are driven in obstructed ground.
- f) Where group capacity governs.
- g) Where passive soil pressures are considered a major factor in developing pile lateral load capacity.
- h) Where excessive ground heave may occur.
- i) Where there is a mixture of vertical and batter piles.
- j) Where densification of granular soils can occur.

Special installation methods can be used as an alternative to increasing pile spacing. For example, predrilling in Cases b, e, and h or a staggered installation sequence in Case c might be used. Closer spacing might be permitted for end-bearing piles installed in predrilled holes. Under special conditions, the pile spacing might be determined by the available construction area.

3.7—Lateral support

All soils, except extremely soft soils (s_u less than 100 lb/ft² [5 kPa]), will usually provide sufficient lateral support to prevent the embedded length of most common concrete-pile cross sections from buckling under axial load. In extremely soft soil, however, very slender pile sections can buckle. All laterally unsupported portions of piles should be designed to resist buckling under all loading conditions and should be treated as columns in determining effective lengths and buckling loads.

3.8—Batter piles

Batter piles are commonly used to resist large horizontal forces or to increase the lateral rigidity of the foundation under such loading. When used, batter piles tend to resist most, if not all, of the horizontal loading. The design should reflect this type of behavior. The use of batter piles to resist seismic forces requires extreme care because these piles restrain lateral displacement and may require unattainable axial deformation ductility. When batter piles are used, a complete structural analysis that includes the piles, pile caps, structure, and the soil is necessary if the forces are to be properly accounted for, including the possibility of tension developing in some piles. Hrennikoff (1950), Saul (1968), and Reese et al. (1970) have reported suitable analyses.

When batter piles are used together with vertical piles, the design of the foundation structure should consider that the batter piles will accept a portion of the vertical load. The inclination and position of the batter piling should be selected so that when a lateral load is applied, the resultant of the lateral and vertical loadings is axial, and the effects of bending moments are kept to a minimum. Bending stresses due to the weight of the pile itself, such as those that occur for a long freestanding portion of a batter pile in marine structures, should be taken into consideration. Where negative skin friction can develop (3.10.1), the weight of the settling soil and drag forces on the pile can induce both bending and axial loads in batter piles.

3.9—Axial load distribution

Axial load distribution includes both rate of transfer of load from the pile to the soil and distribution of load between friction and end bearing (soil-resistance distribution). The distribution of load can be approximated by theoretical analysis, by special load-test methods, or by properly instrumented load-test piles. Any theoretical analysis of distribution of load between pile and soil should take into account all the factors, such as type of soil and soil properties, vertical arrangement and thickness of soil strata, group behavior, type of pile (including pile material, surface texture, and shape), and effects of time.

The full design load can be considered to act on the pile down to the surface of the soil layer that provides permanent support. Below that level, the loads applied to the pile will be distributed into the soil at rates that will vary with the type of soil, type and shape of pile, and other factors.

Even for piles classified as end-bearing, some part of the load may be transferred from the pile to the soil along that portion of the pile embedded in soil that provides permanent lateral support. Where negative skin friction conditions exist (3.10.1), the full pile load, including the negative friction load, should be considered to act at the top of the bearing stratum. Davisson (1993) provides analyses and case histories of negative skin friction effects.

3.10—Long-term performance

Every pile foundation represents an interaction between the piles and the subsurface materials that surround and underlie the foundation. In the design of pile foundations, it is imperative to consider the changes in subsoil conditions that can occur over time and adversely affect the performance of the foundation. Typical consequences of possible changes are long-term consolidation of the soil that surrounds or underlies the piles, lateral displacements due to unbalanced vertical loads or excavations adjacent to the foundations, consolidation effects of vibrations and fluctuation in groundwater, and scour. It is sometimes neither possible nor practical to evaluate the effects of such changes by means of pile load tests. In many instances, judgment decisions should be made based on a combination of theory and experience. Some of these possible changes in subsurface conditions, however, are not predictable and thus cannot be evaluated accurately by the designing engineer.

3.10.1 Long-term consolidation and negative skin friction—If piles extend through soft compressible clays and silts to final penetration into suitable bearing material, the upper strata can carry some portion of a test load or working load by friction. The frictional capacity of these compressible upper strata could be temporary, however, and prolonged loading can cause consolidation of these soils, with an increasing part of the design dead load being carried by the underlying bearing material. Under such conditions, temporary live loads may not have a major effect on the load distribution. Analyses of long-term effects should be performed by qualified professionals who have adequate information about the project.

Possible long-term settlements due to the consolidation of compressible strata located beneath, or even at a considerable depth below, the pile tip should be evaluated. Such settlements of pile groups and entire foundations cannot be evaluated by means of load tests alone. They can, however, be estimated with a reasonable degree of accuracy by means of appropriate soil borings, soil samples, laboratory tests, and soil mechanics theory.

Downward movement of the soil with respect to the pile, resulting from consolidation of soft upper layers through which the pile extends or the shrinkage of certain types of clay soils when the moisture content decreases, produces negative skin friction loading on the pile. Consolidation is generally caused by an additional load being applied at the ground surface, such as from a recently placed fill or by lowering of the water table, and continues until a state of equilibrium is reached again. Under negative friction conditions, the positive skin friction over the upper portion of the piles can be reversed completely, causing negative skin friction (downdrag) and an increase in the total load that will be carried by the piles. The critical section of the pile can be located at the surface of the permanent bearing strata. The magnitude of this load is limited by certain factors, such as the shearing resistance between the pile surface and the soil, the internal shear strength of the soil, the pile shape, and the volume of soil affecting each pile (Davisson 1993). If subsoil conditions are of this type, data from load tests conducted on piles of different length, piles instrumented to reveal actual load distribution, or piles cased off through the consolidation zone, together with the results of laboratory tests that evaluate the stress-strain properties of the subsoil, can be used to determine appropriate design criteria.

Negative skin friction loads should be considered when evaluating both the soil bearing capacity and the pile shaft strength requirements (4.2.2.2). Evaluation of pile load tests should account for the positive friction developing during the short-term load-test duration as opposed to the negative friction that develops at long-term service conditions (Davisson 1993). In addition to axial negative skin friction loads, the weight of the settling soil and drag forces on the pile sides can induce bending loads in batter piles.

3.10.2 Lateral displacement—Pile foundations for retaining walls and abutments, as well as many other types of structures, can be acted upon by lateral forces developed in the subsoil beneath the structures. Such deep-seated lateral forces against pile foundations are commonly due to unbalanced vertical loads produced by such things as the added weight of adjacent fill or reduction in subsoil pressures caused by adjacent excavation. If the subsoil consists of material susceptible to long-term lateral movements, displacements of pile foundations can be progressive and become very large. Moreover, under such conditions, piles can be subjected to large shear and flexural stresses and should be designed accordingly.

3.10.3 Vibration consolidation—If a friction pile foundation in loose granular soil is subjected to excessive vibrations, unacceptable settlements can occur as a result of the densification of the granular soil that surrounds or underlies

the piles. The design of pile foundations under such conditions calls for judgment and experience in addition to theoretical analysis based on adequate subsoil data. It may be necessary to develop the pile capacity within strata below those affected by the vibrations.

3.10.4 Groundwater—The design should consider the possible effects of groundwater fluctuations on the long-term performance of pile foundations. Lowering of the groundwater level can cause consolidation of soft clay and plastic silt. If such compressible strata surround or underlie the piles, then consolidation can result in negative skin friction loads (3.10.1) and settlement of the foundations. On the other hand, a rise in the groundwater table in loessial soil can cause settlement of friction-pile foundations if they are subjected to vibrations or shock loadings. Also, certain types of clay soils are subject to shrinking or swelling as the moisture content changes, which could adversely affect the pile-foundation performance. Under such conditions, steps should be taken to isolate the pile from the zone of variable moisture content and develop the pile capacity in the soils of constant moisture content or, as an alternative, whatever precautions are necessary should be taken to maintain a fairly constant moisture content in the soils. If swelling of the soil occurs before the full load is on the pile (or for lightly loaded piles), it may be necessary to provide tension reinforcement in the pile.

For pile foundations bearing in sand, raising the water table results in an effective stress decrease and a corresponding reduction in pile bearing capacity. This phenomenon commonly occurs where piles are driven in a deep excavation where temporary dewatering has taken place.

3.10.5 Scour—For pile foundations of bridges or other structures over water, or for structures adjacent to water subject to wave action that might undermine the foundation, the possibility of scour should be considered in the design. Where upper soil materials can be removed by scour, the piles should have adequate axial and lateral capacities produced by sufficient penetration below the depth of scour for the various loading conditions. Furthermore, that portion of the pile extending through the zone of possible scour should be designed for lateral loads and buckling (4.3.4).

3.11—Lateral capacity

Horizontal and eccentric loads cause bending stresses in the piles and affect the distribution of the total axial load to individual piles in the group. Lateral forces on piles will depend on the environment and function of the supported structure and can be produced by wind, waves, berthing and/or mooring of ships, ice action, earth pressures, seismic action, or mechanical causes. Batter piles are frequently used to resist lateral loads (3.8).

The ability of vertical piles to resist lateral loads depends on:

- Pile type, material, and stiffness.
- Subsoil conditions.
- Embedment of pile, pile cap, and foundation wall in the soil.
- Degree of fixity of pile-to-cap connection.

- Pile spacing.
- Existence and magnitude of axial loads.

Lateral loading is often repetitive and in some cases reversible in direction, which can lead to an increase in the pile deflections and moments over those observed under a single cycle of loading. Group-effect limitations are more severe for laterally loaded piles than for those with axial loads only (Davisson 1970b).

Laterally loaded, short, very stiff piles can fail in a lateral soil bearing capacity mode by rotating through the soil (Davisson and Prakash 1963). Most foundation piles, however, are sufficiently flexible with respect to the soil so that the pile bending moments and shears induced by the lateral loads will exceed the structural strength of the pile shaft before reaching a soil bearing capacity mode. Although lateral bearing capacity should be checked, the lateral design of foundation piles are most typically controlled by either the tolerable lateral displacements or the structural strength of the pile shaft.

In evaluating the lateral capacity of vertical piles, the soil resistance against the pile, pile cap, and foundation walls should be considered. Soil resistance can contribute substantially to the lateral capacity of a pile group or pile foundation, providing that the soil is present for the loading conditions under consideration. The presence of axial compressive loads can contribute to the pile's lateral (bending) capacity by reducing tension stresses caused by bending due to lateral loads. Design methods for lateral loading of concrete piles should consider axial loads, whether compression or tension, and lateral soil resistance. If lateral load capacity is critical, it should be investigated or verified by field tests under actual in-service loading conditions, including the vertical dead load that could be considered permanent. ASTM D3966-07 discusses procedures for testing piles under lateral loads.

For evaluating bending and shear stresses in piles due to horizontal loads, moments, or both, applied at or above the ground surface, the distribution of moment and shear forces along the pile axis should be determined by flexural analysis, including the horizontal subgrade reaction of the soil. Nondimensional solutions based on the theory of a beam on elastic foundations (Hetenyi 1946) are available for a variety of distributions of horizontal subgrade modulus with depth (Reese and Matlock 1956; Matlock and Reese 1962; Broms 1964a,b, 1965; Davisson 1970b; NAVFAC 1982; Prakash and Sharma 1990). The value of the horizontal subgrade modulus used in the analysis should consider group effects and, where warranted, the influence of cyclic loading (Davisson 1970b; Long and Vanneste 1994).

In such analyses, the flexural stiffness of the pile shaft, EI , can be taken as the calculated EI_g for the gross section, unless the horizontal loads and moments, when acting with the applicable concurrent axial loads, are sufficient to cause cracking over a significant length of the pile. When the magnitude of the applied horizontal loads and moments are sufficient to cause cracking along a significant portion of the pile, the flexural stiffness can be calculated in accordance with the recommendations of ACI 318-08 Section 9.5 (effective moment of inertia) or Sections 10.10 and 10.13

(approximate evaluation of slenderness effects), unless a more refined analysis is used.

The use of nondimensional analysis is an efficient first step in the design of a laterally loaded pile, as it permits judging what, if any, unfavorable conditions exist, and if more refined analysis is warranted (Terzaghi et al. 1996). Nondimensional solutions are also valuable tools for evaluating the computer output of more refined analysis. Where more detailed analyses are required to account for complex variations of the subgrade modulus with depth, variations in flexural stiffness EI of the pile shaft along the length, or the nonlinear behavior of the horizontal soil reactions with deflection, computer programs can be used to solve the beam on elastic foundation problems in finite difference form (Matlock and Reese 1962; Reese 1977). The influence of a nonlinear soil resistance-deflection relationship can also be determined using nondimensional solutions in an iterative procedure (Prakash and Sharma 1990).

Consideration of nonlinear soil behavior leads to nonlinear relationships between the applied loads and the resulting moment and shear distribution along the pile. Therefore, when the designer has sufficient information on soil properties to accurately define the horizontal soil reaction relationships (p - y curves), and the conditions warrant the use of nonlinear soil reactions, the distribution of the factored moment and factored shear along the pile axis should be determined by performing the analysis using the applied factored horizontal loads and moments.

The most frequently recommended procedures for estimating the p - y curves (Reese 1984; O'Neill and Reese 1999) were developed on the basis of a few heavily instrumented lateral-load tests on single, vertical piles (Matlock 1970; Cox et al. 1974; Reese et al. 1974; Reese and Welch 1975). Comparisons of predicted behavior with lateral-load test results at other sites (Murchison and O'Neill 1984) illustrate the empirical nature of the p - y curve approach and the uncertainty in extrapolating the procedures to other sites and soil types. Application of the procedures requires considerable judgment and a thorough knowledge of the stress-deformation-time properties of soils (Terzaghi et al. 1996). Methods of modifying the p - y procedures for group effects are in early stages of development, with the relatively few tests being restricted to small groups (generally nine piles or less) and to a small number of sites.

3.12—Uplift capacity

Engineers should exercise caution when applying tension pile load test results to the design of the tension-resisting portion of a structure. Because of the nature of tension test configurations, a tension load test measures only the ability of a pile to adhere to the soil. In service, however, the tension capacity is limited to how much soil weight (buoyant weight) the pile can pick up without exceeding the adhesion to the pile. Therefore, the geometric characteristics (pile length, shape, and spacing) of the pile-soil system also come into play.

For an interior pile in a group of piles, the ultimate pile tension capacity is limited to the buoyant weight of the soil volume defined by the square of the pile spacing times the

pile length. Exterior piles in a group of piles can attach to more soil, but no general agreement exists at this time on the amount.

In summary, the tension capacity for a foundation is limited by both the adhesion to the pile developed from a load test and the amount of soil buoyant weight available to resist tension. The lower capacity indicated for these two limits is used.

CHAPTER 4—STRUCTURAL DESIGN CONSIDERATIONS

4.1—General

This chapter deals primarily with issues that should be considered when addressing the structural design requirements for piles, although the discussion of loads in 4.2 is also of interest when evaluating the geotechnical capacity of the piles. The recommendations in this chapter deal primarily with nonseismic loading. Seismic design considerations are presented in Chapter 5.

4.2—Loads and stresses to be resisted

Stresses in piles result from either temporary or permanent loads. Temporary stresses include those the pile may be subjected to before being put into service, such as handling and driving stresses, and stresses resulting from in-service loading of short and intermittent duration (such as wind, wave, ship, and other impact loads, and seismic loading). Permanent stresses include those resulting from dead and live loads of relatively prolonged duration.

The piles and the soil-pile system are designed to be able to resist the unfactored service loads in all reasonable combinations. These forces should not cause excessive foundation deformations, settlement, or other damage. Furthermore, there should not be a collapse of the foundation system at the factored loads. The pile should be designed to resist the maximum forces that could reasonably occur, regardless of their source. The factored ultimate load combinations in ACI 318-08, Chapter 9 and Appendix C, or other controlling codes should be considered.

All piles or pile groups should be stable under all applicable load combinations. For normal-sized piling, stability will be provided by groups consisting of at least three piles supporting an isolated column. Wall or strip footings not laterally supported should be supported by a staggered row of piles. Two-pile groups are stable if adequately braced in a direction perpendicular to the line through the pile centers. Individual piles are stable if the pile tops are laterally braced in two directions by construction such as a structural floor slab, grade beams, struts, or walls.

4.2.1 Temporary loads and stresses

4.2.1.1 Handling stresses—Concrete piles that are lifted, stored, and transported are subjected to substantial handling stresses. Bending and buckling stresses should be investigated for all conditions, including handling, storing, and transporting. For lifting and transporting stresses, the analysis should be based on 150 percent of the weight of the pile to allow for impact. Pick-up and blocking points should be arranged and

clearly marked so that all stresses are within the allowable limits and cracking does not occur (refer to 7.7, 8.3.2, and 8.4).

4.2.1.2 Driving stresses—Driving stresses are complex functions of pile and soil properties and are influenced by the required driving resistance, the type and operation of the driving equipment used, and the method of installation. Both compressive and tensile stresses occur during driving and can exceed the yield or tensile cracking strengths of the pile material. Dynamic compressive stresses during driving are usually considerably higher than the static compressive stresses resulting from the service load.

The design of the pile and the driving system should provide adequate structural strength to resist the expected driving stresses without damaging the pile. Generally, these installation stresses can be evaluated during design by wave-equation analysis (3.3.2.2). During construction, dynamic measurements can also provide useful information for evaluating driving stresses (3.3.2.2).

4.2.1.3 Tensile and shear stresses—Piles are sometimes subjected to temporary axial tensile stresses resulting from such things as wind, hydrostatic forces, seismic action, and swelling of certain types of clays when the moisture content increases. Temporary bending and shear stresses can result from seismic forces, wind forces, and wave action or ship impact on waterfront and marine structures.

4.2.1.4 Seismic stresses—Earthquake loads on pile foundations can be both lateral and vertical, and result primarily from horizontal and vertical ground accelerations transmitted to the structure by ground action on the piles. Earthquake loads and the design and detailing of piles to resist seismic forces and motions are discussed in Chapter 5.

4.2.2 Permanent loads and stresses

4.2.2.1 Dead- and live-load stresses—Dead and live loads cause compressive, tensile, bending, and shear stresses, or combinations of these stresses, in piles. The calculation of the compressive force to be carried by a pile should be based on the total dead load and the live load that is reasonably expected to be imposed on the pile. Service live loads are reduced in accordance with accepted engineering principles and the governing building code. The magnitude of the resulting compressive force can vary along the pile length according to the distribution of the load into the soil (refer to 3.9).

Some tension forces can be nearly permanent, such as those due to prolonged hydrostatic pressure. Tension in the pile can dissipate with depth below the ground surface, depending on subsoil conditions, pile type, and other factors.

Tall, slender structures, such as chimneys, power-transmission structures, and towers, are very sensitive to lateral loads. The forces that can be induced in piles of such structures should be carefully investigated for all possible loading combinations and load-factor combinations to identify the most critical pile forces in both tension and compression.

Horizontal and eccentric loads cause bending stresses in the piles and affect the distribution of the total axial load to individual piles in the group. The determination of the moment distribution along a pile subjected to lateral loading

should consider the soil-pile interaction. Factors influencing soil-pile interaction and available analysis methods are discussed in 3.11.

In some structures, second-order deflection ($P-\Delta$) effects can become important. In such cases, their pile foundations should be designed to resist the increased forces associated with these effects (refer to ACI 318-08 Section 10.10.2.2).

4.2.2.2 Negative skin friction—Downward movement of the soil with respect to the pile produces negative skin friction loading on the pile and pile bending stresses if the piles are battered. Downward movement of soil can also impose soil loads on buried parts of structures, such as the walls of pumping stations and utility pipes, which results in additional loads being transmitted to pile tops. Pile loads induced by soil forces on buried portions of structures in settling ground and negative skin friction loads on piles should be considered when evaluating both the soil bearing capacity and the pile shaft strength requirements (refer to 3.10.1).

4.3—Structural strength design and allowable service capacities

4.3.1 General approach to structural capacity—The most common use of foundation piles is to provide foundation support for structures, with axial compression frequently being the primary mode of pile loading. Building codes and regulatory agencies limit the allowable axial, service-load capacities for various pile types based on both soil-pile behavior and on structural-material behavior. Although the permissible pile capacity is frequently controlled by the soil-pile behavior in terms of soil bearing capacity or tolerable displacements, it is also possible for the structural strength of the pile shaft to control this capacity.

Historically, the design of foundation piles has been on an allowable, service-capacity basis, with most building codes and regulatory agencies specifying the structural requirements for the various types of piling on an allowable, unit-stress basis. For example, both the Uniform Building Code (1997) and the International Building Code (IBC 1808.2.9-2006) limit the allowable concrete compressive stress for CIP concrete piles to $0.33f'_c$ and provide provisions for the allowable stress to be increased by concrete confinement (up to a maximum value of $0.40f'_c$), provided required conditions are met. Similarly, both of these codes limit the allowable compressive stress on prestressed concrete piles to $(0.33f'_c - 0.27f_{pc})$. These allowable unit stresses were first published in the 1970s and are for the conditions of a fully embedded and laterally supported pile. They were based on strength design concepts (Davisson et al. 1983; Fuller 1979; PCA 1971) and were also the basis of previous recommendations of this committee.

Whereas axial compression may often be the primary mode of loading, concrete piles are also frequently subjected to axial tension, bending, and shear loadings as well as various combinations of loading, as noted in 3.1. Concrete piles are designed to have adequate structural capacity for all modes and combinations of loading that they will experience. For combined flexure and thrust loadings, the structural adequacy

can be evaluated most readily through the use of moment-thrust interaction diagrams and strength design methods.

This section recommends provisions for ensuring that concrete piles have adequate structural capacity based on strength design methods. Recommendations are provided in 4.3.2, 4.3.4, and 4.3.5 for the direct use of strength design methods based on ACI 318-08. Because of the historical use of allowable capacities and stresses in piling design, however, recommendations are also provided for allowable axial service capacities for concentrically-loaded, laterally-supported piles. The allowable service capacities P_a given in 4.3.3 are intended specifically for cases in which the soil provides full lateral support to the pile and where the applied forces cause no more than minor bending moments resulting from accidental eccentricities. Laterally-supported piles subjected to larger bending moments should be treated in accordance with the strength design provisions in 4.3.2, 4.3.4, and 4.3.5 of this report. Laterally-unsupported piles should be treated as columns in accordance with ACI 318-08 and the provisions in 4.3.2, 4.3.4, and 4.3.5 of this report.

Foundation piles behave similarly to columns, but there can be major differences between the two regarding lateral support conditions, and construction and installation methods. The piles to which the basic allowable stresses apply are fully supported laterally, whereas columns may be laterally unsupported or sometimes supported only at intervals. The failure mode of a column is due to structural inadequacy, whereas pile-foundation failures are caused by either inadequate capacity of the pile-soil system (excessive settlement) or of the structural capacity of the pile. A column is sometimes a more critical structural element than an individual pile. A column is an isolated unit whose failure would probably cause collapse of that portion of the structure supported by the column. A single structural column, however, is often supported by a group of four or more piles with the column load shared by several piles.

The structural design of the pile should consider both temporary and permanent loads and stresses. For example, driving stresses during pile installation (4.2.1.2) can govern the structural design of the pile. Experience from driving precast piles leads to a recommendation that the minimum concrete compressive strength f'_c should be 5000 psi (35 MPa) and that greater strength is often necessary. The structural design of the pile should also consider the subsoil conditions, as they affect the magnitude and distribution of forces within the pile.

4.3.2 Strength design methods—The provisions for strength design of the concrete piles given herein were developed using strength design principles from ACI 318-08, although no attempt has been made to completely follow the column design requirements of ACI 318. In ACI 318-02, the load factors and load combinations were revised to be compatible with those of ASCE 7, and strength reduction factors ϕ were revised to maintain a consistent level of safety. The load factors and strength reduction factors for the pre-2002 versions of ACI 318 now appear as alternate load and strength reduction factors in ACI 318-08.

The revised ACI 318-08 load factors and strength reduction

Table 4.3.2.1—Recommended compressive strength reduction factors ϕ_c

Pile type	Compressive strength reduction factor ϕ_c	
	For use with factored loads based on Chapter 9 of ACI 318	For use with factored loads based on Appendix C of ACI 318
Concrete-filled shell, no confinement	0.60	0.65
Concrete-filled shell, confinement*	0.65	0.70
Uncased, plain or reinforced concrete†	0.55	0.60
Precast reinforced concrete or CIP reinforced concrete within shell	0.65	0.70
Pretensioned, prestressed reinforced concrete	0.65	0.70
Concrete-filled steel pipe	0.70	0.75

*Shell of 14 gauge minimum thickness (0.747 in. [1.9 mm]), shell diameter not over 16 in. (400 mm), for a shell yield stress f_{ys} of 30,000 psi (210 MPa) minimum, f'_c not over 5000 psi (35 MPa), noncorrosive environment, and the shell is not designed to resist any portion of axial load. The increase in concrete strength due to confinement should not exceed 54 percent.

†Auger-grout piles, where concreting takes place through the stem of a hollow-stem auger as it is withdrawn from the soil, cannot be internally inspected. The strength reduction factor of 0.55 to 0.6, dependent on the load factors used, represents an upper boundary for ideal soil conditions with high-quality workmanship. A lower value for the strength reduction factor may be appropriate, depending on the soil conditions and the construction and quality control procedures used. The designer should carefully consider the reliable grout strength testing methods and the minimum cross-sectional area of the pile, taking into account soil conditions and construction procedures. The addition of a central reinforcing bar extending at least 10 ft (3 m) into the pile is recommended, as this adds toughness to resist accidental bending and tension forces resulting from other construction activities.

factors have not been incorporated into all building codes, so this report contains recommendations applicable to both the revised factors in ACI 318-08, Chapter 9, and alternate factors in ACI 318-08, Appendix C.

The general strength design requirement for piling is that the pile be designed to have design strengths at all sections at least equal to the required strengths calculated for the factored loads. The factored loads should be determined using the load factors and combinations of service loads as stipulated in ACI 318-08 Section 9.2 or, where applicable, using the load factors and combinations of service loads as stipulated in ACI 318-08 Section C9.2. The design strength of the pile is computed by multiplying the nominal strength of the pile by a strength reduction factor ϕ , which is less than 1. The nominal strength of the member should be determined in accordance with ACI 318 and the strength reduction factors used should be compatible with the load-factor system being used.

The strength reduction factors ϕ recommended herein for various types of loading conditions generally follow ACI 318 Sections 9.3 and C9.3, except that strength reduction factors for compression, ϕ_c , have been determined by the committee for the pile member types not covered by ACI 318. Recommended strength reduction factors for various forms of loading, as well as additional recommendations, are provided in 4.3.2.1 through 4.3.2.8. Further recommendations for the use of the strength design method with piling are provided in 4.3.4 and 4.3.5.

4.3.2.1 Compressive strength—The recommended compressive strength reduction factors ϕ_c for various types of concrete piles are given in Table 4.3.2.1 for use with ACI 318-08 load factors or the alternate ACI 318-08 Appendix C load factors, as applicable. These reduction factors are based on consideration of construction experience and the different behaviors under loads approaching the failure loads for the various pile types. In addition to the application of a strength reduction factor, all piles subjected to compression should be designed for the eccentricity corresponding to the maximum moment that can accompany the loading condition, but not less than an eccentricity of 5 percent of the pile diameter or width.

The uncased concrete members (CIS piles), as a general class, cannot be inspected after placement of the concrete, and there have been many problems with penetration of the surrounding soil into the pile section in some soil types and with some construction techniques. It is also uncertain to what degree the reinforcement can be placed in its designed position in a reinforced uncased pile. The strength reduction factor is a function of both the dimensional reliability of the cross section and the dependence of the member strength on the strength of the concrete actually attained in the member. The strength reduction factor is set lower for uncased piles: 0.55 for ACI 318-08 Chapter 9 factored loads, and 0.6 for ACI 318-08 Appendix C factored loads. In some soil types, local experience may indicate that lower values of ϕ_c are prudent. Davisson et al. (1983) provides an extensive discussion of these design factors.

4.3.2.2 Flexural strength—For concrete piles subjected to flexure without axial load or flexure combined with axial tension, the ACI 318-08 strength reduction factor ϕ_t is 0.9 for loading combinations in both Chapter 9 and Appendix C. This strength reduction factor can be used for piles subject to the limitations of 4.3.2.7 of this report. For piles subjected to flexure combined with axial compression, the recommended compressive strength reduction factor ϕ_c given in Table 4.3.2.1 should be used for the applicable loading factors.

For reinforced concrete piles, prestressed concrete piles, or concrete-filled pipe piles subjected to flexure and low values of axial compression, the ϕ can be increased from the recommended compression value ϕ_c to the recommended value for flexure without axial load, ϕ_t , in accordance with the procedures given in ACI 318-08 Sections 9.3.2.2 and C.9.2.2.

4.3.2.3 Tensile strength—For concrete piles subjected to axial tension (uplift) loads, the strength reduction factor ϕ_t value used should be 0.9 for loading combinations in ACI 318-08 Chapter 9 and Appendix C. In addition to the application of a strength reduction factor, all piles subjected to tension should be designed for the eccentricity corresponding to the maximum moment that can accompany the loading

condition, but not less than an eccentricity of 5 percent of the pile diameter or width.

4.3.2.4 Strength under combined axial and flexural loading—The design and analysis of concrete piles, except concrete-filled shell piles with confinement, that are subjected to a significant bending moment in addition to axial forces, should be done using moment-thrust interaction diagram information developed in accordance with ACI 318-08 Chapter 10. For unsupported pile sections, the effect of pile slenderness on axial load and bending moment capacity should be considered (refer to 4.3.4, 4.3.5, and ACI 318-08 Section 10.10). The ϕ in 4.3.2.1 and 4.3.2.2 of this report, as adjusted by the provisions of 4.3.2.7 where applicable, and the loading factors and combinations in accordance with ACI 318-08 Chapter 9 or Appendix C, as applicable, should be used. Under no circumstances should the axial capacity exceed the capacity corresponding to an eccentricity of 5 percent of the diameter or width of the pile.

Many of the design aids for reinforced concrete columns (SP-17 [Saatcioglu 2009]; CRSI 2008) can also be used for the design of piles to resist bending plus axial force. Some adjustments, however, are necessary to account for different values of ϕ . Fully understanding any assumptions made in the preparation of the design aids, especially the inclusion or exclusion of the ϕ factor, is imperative. PCI (1993, 2004, 2005) has published design data for pretensioned concrete piles, and a basic approach to the calculation of moment-thrust interaction relationships is given by Gamble (1979). Interaction diagrams showing the impact of slenderness effects on axial load and bending moment capacities of prestressed concrete piles are reported by Anderson and Moustafa (1970) and PCI (1993).

The assumptions made for the analysis of concrete-filled pipe require some specific comments. It can be assumed that there is adequate bond between the concrete and the pipe so that the strains in concrete and steel match at the interface. This assumption cannot be universally true; for example, at sections near the ends of the pipe, the quality of bond can vary, and judgment should be used by the engineer. The concrete compression failure strain can be taken as 0.003. The pipe wall can be modeled either as a continuous tube or as a number of discrete areas of steel evenly spaced around the perimeter of the section. The pipe wall can act as tension or compressive reinforcement, but it cannot act as confinement reinforcement at the same time. The assumption of adequate bond is reasonable in this case, but it is not feasible when considering loading in a case where the objective is to anchor a major tension force into the concrete piling in a permanent structure. Shear connectors or other positive anchorages are required in this scenario.

When a concrete-filled shell is counted on for confinement, the shell is effective in increasing the concentric compression capacity. The shell, however, has only minor effects on the bending capacity, which significantly increases the sensitivity of the member to eccentricity of load. The procedure in Davisson et al. (1983) is recommended for constructing the moment-thrust interaction diagram to address eccentricities for concrete-filled shell piles with confinement.

4.3.2.5 Shear strength—Piles that have significant bending moments will often have significant shear forces. Provisions of ACI 318-08 Chapter 11 should be followed when designing shear reinforcement. Special attention is required when piles have both significant tension and significant shear forces. The strength reduction factor for shear (0.75 for ACI 318-08 Chapter 9 load factors or 0.85 for ACI 318-08 Appendix C load factors, as applicable) should be used with reinforced concrete piles, prestressed concrete piles, and pipe piles. For nonreinforced piles, the shear strength reduction factor for plain concrete (0.55 for ACI 318-08 Section 9.3.2, or 0.65 for ACI 318-08 Section C.3.5) should be used. The shear strength reduction factors should be adjusted by the provisions of 4.3.2.7 where applicable.

4.3.2.6 Development of reinforcement—Development of stress in embedded reinforcement (bond) should correspond to the information in ACI 318-08 Chapter 12.

4.3.2.7 Special considerations for uncased CIS piles—The compression strength reduction factors in 4.3.2.1 for uncased, plain, or reinforced concrete piles are an upper boundary for ideal soil conditions with high-quality workmanship and sound quality-control procedures. A lower value for the strength reduction factor may be appropriate, depending on the soil conditions and the construction and quality control procedures used (Davisson et al. 1983). Davisson et al. (1983) provides recommendations only for adjustment of the compression strength reduction factors for piles; they do not address similar adjustments for the flexure, shear, or tension strength reduction factors. Hence, the recommended strength reduction factors for flexure, shear, or tension in 4.3.2.2 through 4.3.2.5 have not been reduced for potential hidden defects consistent with the lower compression strength reduction factors recommended for uncased, plain, or reinforced concrete piles.

When uncased CIS piles are subject to flexure, shear, or tension loads, the design should consider the soil conditions, the quality-control procedures that will be implemented, the likely workmanship quality, and local experience, and adjust the strength reduction factors for the loading modes in 4.3.2.2 through 4.3.2.5 in a manner similar to recommendations in 4.3.2.1.

As a preliminary recommendation, it is recommended that strength reduction factors in 4.3.2.2 through 4.3.2.5 be adjusted by an additional reduction factor that should not be greater than the ratio of the 4.3.2.1 compressive strength reduction factor for the particular pile type to the ACI 318-08 compressive strength reduction factor for nonspiral reinforced columns (Chapter 9 or Appendix C, as applicable). For example, the upper bound of this additional reduction factor for an uncased, plain, or reinforced concrete pile would be 0.85, that is, 0.60/0.70.

4.3.2.8 Prestressed piles—Prestressed piles designed by strength design methods also require serviceability checks to demonstrate that their service load behavior is adequate, in addition to the limiting capacities found through strength design. These serviceability checks should be performed in accordance with the recommendations of the design report.