ACI 543R-12

Guide to Design, Manufacture, and Installation of Concrete Piles

Reported by ACI Committee 543

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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.

CONTENTS

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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 base can be 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) or 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)
A_c = area of concrete (including prestressing steel), in.^{2} (mm^2)
= $A_g - A_{st}$, for reinforced concrete piles, in.^{2} (mm^2)
A_{core} = area of core of section, to outside diameter of the spiral steel, in.^{2} (mm^2)
A_g = gross area of pile, in.^{2} (mm^2)
A_{ch} = area of core within transverse reinforcement, measured out-to-out of the reinforcement, in.^{2} (mm^2)
A_p = area of steel pipe or tube, in.^{2} (mm^2)
A_{ps} = area of prestressing steel, in.^{2} (mm^2)
A_{sh} = total area of transverse reinforcement in direction considered, in.^{2} (mm^2)
A_{sp} = area of spiral or tie bar, in.^{2} (mm^2)
A_{st} = total area of longitudinal reinforcement, in.^{2} (mm^2)
b_c = width of section in direction considered, in. (mm)
D = steel shell diameter, in. (mm)
d_{core} = diameter of core section, to outside of spiral, in. (mm)
E = modulus of elasticity for pile material, psi (MPa = N/mm^2)
EI = flexural stiffness of the pile, lb-in.^{2} (N-mm^2)
f_c' = specified compressive strength of concrete, psi (MPa)
f_{pc} = effective prestress in concrete after losses, psi (MPa)
f_{ps} = stress in prestressed reinforcement at nominal strength of member, psi (MPa)
f_y = yield stress of nonprestressed reinforcement, psi (MPa)
f_{yh} = yield stress of transverse spiral or tie reinforcement, psi (MPa)
f_{yp} = yield stress of steel pipe or tube, psi (MPa)
f_{ys} = yield stress of steel shell, psi (MPa)
f_{yt} = yield stress of transverse reinforcement, psi (MPa)
g = acceleration of gravity, in./s^2 (m/s^2)
h_c = cross-sectional dimension of pile core, center-to-center of hoop reinforcement, in. (mm)
I = moment of inertia of the pile section, in.^{4} (mm^4)
I_g = moment of inertia of the gross pile section, in.^{4} (mm^4)
K = horizontal subgrade modulus for cohesive soils, psi (N/mm^2)
K = coefficient for determining effective pile length
L = pile length, in. (mm)
L_s = depth below ground surface to point of fixity, in. (mm)
L_u = length of pile above ground surface, in. (mm)
ℓ_e = effective pile length = $K\ell_u$, in. (mm)
ℓ_u = unsupported structural pile length, in. (mm)
M = pile moment, in.-lb (N-mm)
$(N_1)_{60}$ = 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)
n_h = coefficient of horizontal subgrade modulus, lb/in.^{3} (N/mm^3)
P = axial test load on pile, lb (N)
P_a = allowable axial compression service capacity, lb (N)
P_{at} = allowable axial tension service capacity, lb (N)
P_u = factored axial load on pile, lb (N)
R = radius of gyration of gross area of pile, in. (mm)
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 more 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 load 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. Failure to include 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 calibrate 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 additional 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 displacement-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 the same 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-excavation 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-excavation or increasing the pile spacing. Wet rotary pre-excavation 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 reseat 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/ft^2 [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 shear and 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-Δ) 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, Appendix C. 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	Compressive strength reduction factor ϕ_c: 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 in 4.3.3.3 of this report.

Table 4.3.2.8—Allowable service-load stresses in prestressed piles*

Loading condition	Permanent, psi	Temporary, psi
Tension		
Concrete tension†	0	$3\sqrt{f_c'}$
Flexure plus compression		
Concrete tension	0	$6\sqrt{f_c'}$
Concrete tension for marine work	0	$3\sqrt{f_c'}$
Concrete compression	$0.45f_c'$	$0.6f_c'$
Flexure plus tension†		
Concrete tension	0	$3\sqrt{f_c'}$
Concrete compression	$0.45f_c'$	$0.6f_c'$

*Units for allowable stresses and f_c' in the equations in this table are psi (1 psi = 0.0069 MPa). Because the tension stresses are a function of the square root of f_c', if other units are used for f_c', it is also necessary to change the coefficients in front of the radical. Conversions for the equations are:

$3\sqrt{f_c'}$	$(\sqrt{f_c'})/4$	Equation in terms of psi
$6\sqrt{f_c'}$	$(\sqrt{f_c'})/2$	Equation in terms of MPa

†In piles that are expected to be subjected to tension, the ultimate capacity of the prestressing steel should be equal to or greater than 1.2 times the direct tension cracking force, unless the available strength is greater than twice the required factored ultimate tension load; that is, $f_{ps}A_{ps} \geq 1.2(f_{pc} + 7.5\sqrt{f_c'})A_c$, where f_{pc} and f_{ps} are in psi units.

The evaluation of the thrust-moment capacity of prestressed piles under tensile loading is a special case. For tensile loads at small eccentricities, the breaking strain of the prestressing strand rather than the crushing strain of the concrete can control the nominal moment capacity, and this should be considered when developing nominal-strength, moment-thrust interaction diagrams for prestressed piles. The second footnote of Table 4.3.2.8 imposes limits of tension loading that are intended to address this issue.

4.3.3 *Allowable axial service capacities for concentrically-loaded, laterally-supported piles*—Equations for the allowable axial compressive service capacity can be developed for different types of concrete foundation piles by considering the recommended compressive strength reduction factors in 4.3.2.1, a minimum eccentricity factor, and a combined average load factor.

The eccentricity factor is a function of the pile cross-sectional shape (octagonal, round, square, or triangular) for plain concrete piles. For a reinforced concrete pile, the eccentricity factor is also a function of the reinforcing steel ratio, the location of the reinforcement within the cross section, and the concrete and steel strengths. The eccentricity factor for a particular pile section can be determined from its nominal strength interaction diagram as the ratio of the nominal axial strength at a 5 percent eccentricity to the nominal axial strength under concentric loading. The allowable axial service capacity equations in Table 4.3.3 are based on eccentricity factors taken from a Federal Highway Administration (FHWA) report (Davisson et al. 1983) and PCA (1971), in which the general shapes of moment-axial force interaction diagrams for various types of piles were studied in detail.

Table 4.3.3—Allowable service capacity for piles with negligible bending*

Pile type	Allowable compressive capacity
Concrete-filled shell, no confinement	$P_a = 0.32f_c'A_c$
Concrete-filled shell, confinement†	$P_a = 0.26(f_c' + 8.2\ t_{shell}\ f_{ys}/D)\ A_c \leq 0.4f_c'A_c$
Uncased plain concrete‡	$P_a = 0.29f_c'A_c$
Uncased reinforced concrete§‖	$P_a = 0.28f_c'A_c + 0.33f_yA_{st}$
Precast reinforced concrete or cast-in-place reinforced concrete within shell§‖	$P_a = 0.33\ f_c'A_c + 0.39f_yA_{st}$
Pretensioned, prestressed concrete	$P_a = A_c(0.33f_c' - 0.27f_{pc})$
Concrete-filled steel pipe	$P_a = 0.37f_c'A_c + 0.43f_{yp}A_p$

*Based on an eccentricity of 5 percent of pile diameter or width, and an assumed average load factor of 1.4 for ACI 318-02 Chapter 9 factored load combinations and 1.55 for ACI 318-02 Appendix C factored load combinations. In cases of very high live or other loadings such that the average load factor exceeds these values, the allowable capacity equations should be reduced accordingly.

†Shell of 14 gauge minimum thickness (0.0747 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 shell is not designed to resist any portion of axial load. The allowable load P_a should not exceed $0.40f_c'A_c$, on the basis of IBC and other codes.

‡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 0.6, on which the strength coefficient of 0.29 is based, 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, 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.

§Applicable if the longitudinal steel cross-sectional area is at least 1.5 percent of the gross pile area and at least four symmetrically placed reinforcing bars are supplied (with six bars preferred).

‖An eccentricity factor of 0.86 has been assumed for reinforced concrete piles. For reinforced concrete piles with a concrete strength f_c' less than 5000 psi (35 MPa), or for piles with axial reinforcement areas (as a percentage of the gross pile area) greater than 3 percent for round piles or greater than 4.5 percent for square piles, the eccentricity factor should be evaluated from a nominal strength moment-thrust interaction diagram and the allowable capacity equation adjusted accordingly.

The combined average load factor should be computed as the ratio of the factored load to the service load. The allowable axial service capacity equations in Table 4.3.3 assume a combined average load factor of 1.4 for ACI 318-08 Chapter 9, and 1.55 for ACI 318-08 Appendix C. These factors are based on an average of the ACI 318-08 load factors for dead and live load (assuming the dead load is equal to live load), which is generally a conservative assumption. If the controlling loading case is dominated by very high live load or other loadings, such that the actual average load factor exceeds 1.4 under ACI 318-08 Chapter 9, or 1.55 under ACI 318-08 Appendix C, the allowable capacity equations in Table 4.3.3 should be reduced accordingly.

The allowable axial compressive service capacity equations given in this report are specifically restricted to 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 eccentricity). 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.

4.3.3.1 *Concentric compression*—The allowable axial compressive service capacity for laterally supported solid concrete piles can be determined by the equations given in Table 4.3.3. These equations were developed based on the procedures in 4.3.3 and correspond to a nominal factor of safety (ratio of the average load factor to the strength reduction factor) that ranges from approximately 2.1 to 2.6, depending on the pile type. Hollow piles and piles with triangular cross sections should be analyzed and designed using a moment-axial force interaction design method, with a minimum eccentricity of 5 percent of the pile diameter or width, as described in 4.3.2.

4.3.3.2 *Concentric tension*—Concrete piles subjected to axial tension (uplift) loads are designed for the full tension load to be resisted by the steel (refer to 4.5). The allowable tension service capacity for reinforcing steel is

$$P_{at} = 0.5 f_y A_{st} \tag{4.3.3.2}$$

For prestressed concrete piles where the full tension load, or part of that load (part of force carried by strands and part by dowels), is to be resisted at the pile head by unstressed strands extended into a footing or cap, the allowable tension service capacity should be based on an allowable strand stress of 30,000 psi (207 MPa) (PCI 1993). Other high-strength steels, such as post-tensioning bars or very-high-strength reinforcement, are also limited to the 30,000 psi (207 MPa) allowable stress.

4.3.3.3 *Special considerations for prestressed piles*—Prestressed piles are subject to serviceability checks applied to demonstrate that their service load behavior is adequate, in addition to the limiting capacities described in 4.3.2. The allowable service load stress limits given in Table 4.3.2.8 should be determined using concrete compressive strength f_c' corresponding to the age of the concrete under consideration.

4.3.4 *Laterally unsupported piles*—That portion of the pile that extends through air, water, or extremely soft soil (Prakash and Sharma 1990) should be considered unsupported and be designed as a column to resist buckling under the imposed loads (refer to 3.7). The effects of length on the strength of piles should be taken into account in accordance with ACI 318-08 Sections 10.10 and 10.13. Whereas 10.11 and 10.13 of ACI 318-08 give an approximate method suitable for $K\ell_u/r < 100$, 10.10.3 describes the requirements for a rational analysis of the effects of length.

The effective pile length ℓ_e is determined by multiplying the unsupported structural pile length ℓ_u by the appropriate value of the coefficient K from Table 4.3.4a or from ACI 318-08 Chapter 10. For cases in which the top of the pile is free to translate, the coefficient K requires careful consideration and should exceed 1.0, following ACI 318-08 Section 10.10.7.2.

The unsupported portion of a foundation pile is an extension of the laterally supported portion, which can be several times longer than the unsupported portion. Thus, such a pile is deeply embedded for its lower length and, at some depth below the ground surface, could be considered to be fixed. Achieving complete end fixity for a building column is difficult. For many structures using unsupported pile lengths, however, the pile tops are framed into the structure much more heavily than most building columns with a greater resulting end fixity at the top. For shallow penetrations, the pile point should be considered hinged unless test data prove otherwise.

Table 4.3.4a—Values for *K* for various head and end conditions*

Head condition	End conditions: Both fixed	One fixed	Both hinged
Nontranslating	0.6	0.8	1.0
Translating	>1.0	>2.0	Unstable

*For piles doweled to the cap, the degree of fixity at the doweled end could range from 50 to 100 percent, depending on the embedment of the pile into the cap, the design of the doweled connection, and the resistance of the structure to translation and rotation. For fixed ends, the values of K are based on complete fixity and should be adjusted depending on the actual degree of fixity (refer to ACI 318-08, Davisson [1970b], Joen and Park [1990b], and PCI [1993]).

The structural length ℓ_u as defined herein is the unsupported pile length between points of fixity or between hinged ends. For a pile fixed at some depth L_s below the ground surface, the structural length ℓ_u would be equal to the length of pile above the ground surface, L_u, plus the depth L_s.

$$\ell_u = L_u + L_s \tag{4.3.4a}$$

The depth below the ground surface to the point of fixity, L_s, can be estimated by Eq. (4.3.4b) for preloaded clays, or by Eq. (4.3.4c) for normally loaded clay, granular soils, silt, and peat.

$$L_s = 1.4R \text{ where } R = \sqrt[4]{\frac{EI}{k}} \tag{4.3.4b}$$

$$L_s = 1.8T \text{ where } T = \sqrt[5]{\frac{EI}{n_h}} \tag{4.3.4c}$$

The total length of the portion of the pile embedded in the soil should be longer than $4R$ or $4T$ for this analysis to be valid; otherwise, a more detailed analysis is required. Furthermore, the unsupported length above ground should be greater than $2R$ (that is, $L_u > 2R$) or T (that is, $L_u > T$) for Eq. (4.3.4b) and (4.3.4c) to be valid. In most practical cases, the unsupported length above ground, L_u, will be greater than $2R$ or T. For cases where the L_u value does not satisfy the restrictions on Eq. (4.3.4b) and (4.3.4c), modifications of the coefficients in these equations are required (Davisson and Robinson 1965; Prakash and Sharma 1990).

The horizontal subgrade modulus k is approximately 67 times the undrained shear strength of the soil ($k = 67s_u$). It is assumed to be constant with depth for preloaded clay and to vary with depth for normally loaded clay. The value of the coefficient of horizontal subgrade modulus, n_h, for normally loaded clay is equal to k divided by the depth and can be approximated by the best triangular fit (slope of line through the origin) for the top 10 to 15 ft (3 to 4.5 m) on the k-versus-depth plot (Davisson 1970b). Representative values of the coefficient of horizontal subgrade modulus n_h for other soils are shown in Table 4.3.4b. These values also apply to submerged soils.

4.3.5 *Piles in trestles*—For piles supporting trestles or marine structures that could occasionally receive large overloads, the capacities determined on the basis of strength design (4.3.2) or the allowable service capacities determined in 4.3.3 should be reduced by 10 percent. The capacity is reduced further by a reduction factor depending on both the ℓ_u/r ratio and the head and end conditions (4.3.4).

4.4—Installation and service conditions affecting design

Several installation conditions can affect the overall pile foundation design and the determination of pile capacity. Some of these relate to installation methods, equipment, and techniques (Chapter 8). Others relate to the subsoil conditions or the qualifications of the pile contractor. Obviously, the engineer cannot allow for all contingencies in a design, but many can be provided for by proper analysis of subsoil data, preparation of competent specifications, use of qualified contractors, and adequate inspection of the work.

The effects of variations of pile head positions and pile alignment on pile loads and pile strength should be incorporated in the pile design. The selection of tolerances for incorporation into construction specifications should consider the practicality of pile installation to the proposed tolerances with the specific pile type(s) selected for the site-specific subsurface conditions and their potential economical impact, as well as their effects on the pile design.

4.4.1 *Pile-head location tolerances*—Some tolerance should be allowed between the as-installed position of the pile head and the design location. Deviations from the plan pile-head locations can be caused by survey errors; inaccurate positioning of the pile over its location stake; equipment inadequate to hold the pile on location; the pile drifting off location due to underground obstructions or sloping hard soil strata; misalignment of piles driven through overburden; or by general ground movements after the piles have been driven caused by embankment pressures, construction operations, or other surcharge loads.

The deviation that should be allowed varies with the pile load and group size. A smaller tolerance is required for a single pile carrying a very high load. A larger tolerance can be allowed for a large group of piles under a structural mat. A tolerance of 3 in. (75 mm) in any direction is reasonable for normal pile usage. Marine work and large piles may require larger tolerances.

Generally, an overload of 10 percent on a pile due to deviation of the pile location does not require modifying the pile cap or group. If this overload is exceeded, additional piles should be installed and, where necessary, the pile cap modified so that the center of gravity of the group remains substantially under that of the load.

Table 4.3.4b—Values of n_h

Soil type	n_h, lb/in.^{3}	n_h, kN/m^3
Sand* and inorganic silt		
Loose	1.5	407
Medium	10	2710
Dense	30	8140
Organic silt	0.4 to 3	109 to 814
Peat	0.2	54

*Values given for granular soils are conservative. Higher values require justification by lateral load test (Davisson 1970b).

Sometimes piles driven off of location can be pulled or pushed back into plan location, but this practice is not recommended. If this practice is permitted, the force used to move the pile into proper position should be limited and carefully controlled according to a lateral load analysis, considering the type and size of pile and the soil conditions. This is especially critical for precast piles used for trestle structures where a long moment arm can result in structural damage to the pile even with relatively low forces (refer to 8.4.5).

4.4.2 *Axial alignment tolerances*—Deviations from required axial alignment can result from the pile driven off required alignment but with its axis remaining straight, the pile driven with its axis not on a straight line from pile head to tip, or a combination of these two with the pile bent and the tip off its plan location. Deviations from a straight line axis can take the form of a long sweeping bend or a sharp bend called a dogleg.

The deviation of the pile axis from the specified alignment, whether vertical or battered, should be within the following tolerances:

- Two percent of the pile length for embedded piles driven through sandy soils or soft clays.
- Four percent of the pile length for embedded piles driven through difficult soils of nonuniform consistency, boulder-ridden soils, or batter piles driven into gravel.
- A maximum of 2 percent of the total pile length in marine structures.

Piles driven outside of these tolerances should be reviewed by the engineer. The review should include consideration of horizontal forces and interference with other piles and may require review of the pile cap.

For axial deviations from a straight line (bent piles), the allowable tolerance could range from 2 to 4 percent of the pile length, depending on subsoil conditions and type of bend, which could be sharp, excluding breaks in the pile, or sweeping bends of varying radii. Experience and load tests have demonstrated that, in most cases, the passive soil pressures are sufficient to restrain the pile against the bending stresses that can develop. For severely bent piles, the capacity can be analyzed by soil mechanics principles or checked by load test. When axial alignment cannot be adequately measured for driven piles, the tolerances should be more conservative.

4.4.3 *Corrosion*—The pile environment should be carefully checked for possible corrosion of either the concrete or the load-bearing steel. Corrosion can be caused by direct chemical attack (from soil, industrial wastes, or organic fills), electrolytic action (chemical or stray direct currents), or oxidation.

When the pile is embedded in natural soil deposits, as opposed to recently placed fills, corrosion due to normal oxidation is generally not progressive and frequently very minor. The presence of corrosive chemicals or destructive electric currents should be determined and the proper precautions taken. Soils and water with high sulfate contents require special precautions to address durability (refer to Chapter 6).

Under detrimental corrosive environments, exposed load-bearing steel should be protected by coatings, concrete encasement, or cathodic protection. Concrete can be protected from chemical attack by using special cements, very rich and dense mixtures, special coatings, and sometimes by using steel encasement. Fiberglass jackets have also been used. Pile splices may require special treatment to provide adequate corrosion resistance.

4.4.4 *Splices*—Precast piles are usually designed and constructed in one piece; however, field splices may be needed if the lengths are misjudged. In the cases of very long piles, those long enough to make manufacturing, transportation, and handling inconvenient, field splices will be part of the original design. Some sectional precast piles have standard stock lengths and splicing is a part of their normal manufacture and usage. Sectional piles can also be mandated by headroom limitations at the pile locations or by the limits of the contractor's equipment. The engineer should exercise control over the use of or need for pile splices through their choice of pile types and preparation of specified installation requirements.

Splices driven below the ground surface should be designed to resist the driving forces and the service loads with the same factor of safety as the basic pile material. Above-ground splices and built-up pile sections should be designed to develop the required pile strength for the imposed loads, and also driving forces if they are to be driven after splicing. Splices may need to be designed to resist the full compression, bending, and tension strength of the body of the pile. Torsional strength can be a consideration in some cases. The potential for corrosion should be considered when selecting the final locations for splices. Special protective sleeves or other protective means may have to be provided when the pile splice will be exposed to seawater or other severe corrosion hazards. Bruce and Hebert (1974a,b), Gamble and Bruce (1990), and Venuti (1980) report on the behavior of several different splices and discuss many other splices that may be available.

For the detailed design of the splice, several different critical sections and different failure modes should be considered. For instance, if the splice involves dowels (in any form), the most critical section could be either at the ends of the sections being joined or at the ends of the dowel bars. The capacity could be governed by either the pile strength, splice strength, or bond capacities of either the dowels or the pile reinforcement. The bond problem will be especially severe for pretensioned piles, and the dowels should extend the full development length of the strand.

Many specific requirements can be placed on mechanical splices, including:

- Ends of segments should be plane and perpendicular to the pile axis.
- Splices should have a centering device.
- Splices should be symmetrical about axis of the member.
- Locking and connection devices should be designed and installed to prevent dislodgement during driving.

Adequate confinement reinforcement should be provided in the splice region. Dowel bars that are embedded in the pile as part of the splice mechanism may need to have staggered cutoff points rather than all ending at the same section. Dowel splices should have oversized grout holes to permit easy and complete filling of the holes. The holes can be either drilled or cast.

4.4.5 *Subsoil behavior affecting pile design capacity*—There are several conditions of soil behavior that can develop during or subsequent to pile installation that can affect the structural pile capacity, the geotechnical pile capacity, or both. The possibility of these phenomena should be recognized by the designer when establishing such requirements as type of pile, pile length, reinforcing details, installation procedures, and inspection procedures. Some of these conditions are:

- Soil relaxation and freeze (3.3.5).
- Compaction or densification of subsoils as installation progresses that leads to variations in pile lengths (3.3.6, 8.2.6, and 8.2.7).
- Temporary liquefaction of soils during pile installation (3.3.7) or during seismic events (5.3.1).
- Pile heave or floatation during installation (3.3.8).

4.4.6 *Effect of vibration on concrete*—This is usually a consideration in installing CIP concrete piles using a steel casing or shell. Pile installation is done in two separate operations: driving the shell and filling it with concrete. Usually the concreting operation follows closely behind the driving, provided that the vibrations caused by driving do not damage the fresh concrete. Tests have indicated that pile-driving vibration during the initial setting period of concrete has no detrimental effect on the strength of the pile (Bastian 1970). The minimum distance between driving and concreting operations, however, is often specified as 10 to 20 ft (3 to 6 m) (Davisson 1972b; Fuller 1983). When a minimum distance is not specified, it is generally satisfactory if one open pile remains between the driving operation and a concreted pile or if the minimum distance is 20 ft (6 m), whichever is less. When ground heave or relaxation is occurring, however, the concreting operation should not be closer to pile driving than the heave range or the range within which redriving is required.

The sequence of installation of CIS concrete piles should be controlled to prevent damage to freshly placed concrete by the driving or drilling of adjacent piles. This frequently precludes the installation of adjacent piling on the same day as a means of preventing ground displacements that could harm the immature concrete.

4.4.7 *Bursting of hollow-core prestressed piles*—Internal radial pressures in both open-ended and close-ended hollow precast piles lead to tension in the pile walls and can cause bursting of such piles. These radial pressures can result from driving or installation conditions, such as use of internal jets, water-hammer effects, lateral soil plug pressures, or concrete pressures if filled after installation. They can also develop under service conditions such as gas pressure buildup from decomposition of core form materials, or ice pressure from freezing of free water in the core. The potential effects of such internal pressures should be evaluated during the design of such piles (7.2.5 and 8.3.1.5).

4.5—Other design and specification considerations

The pile-foundation design should include other considerations that may relate to specific types of piles or that may have to be covered in the plans and specifications so that piles are installed in accordance with the overall design. Some of these considerations are closely related to items discussed in Chapters 7 and 8.

4.5.1 *Pile dimensions*—Usually the minimum acceptable diameter or side dimension for driven piles is 8 in. (200 mm). Except for auger-injected piles and drilled and grouted piles, drilled piles are usually a minimum of 16 in. (400 mm) diameter, but 12 in. (300 mm) diameter cases have been reported. If construction or inspection personnel are required to enter the shaft, however, the diameter should be at least 30 in. (760 mm).

4.5.2 *Pile shells*—Pile shells or casings driven without a mandrel should be of adequate strength and thickness to withstand the driving stresses and transmit the driving energy without failure. Proper selection can be made with a wave-equation analysis. Pile shells driven with a mandrel should be of adequate strength and thickness to maintain the cross-sectional shape and alignment of the pile after the mandrel is withdrawn.

Corrugated shells are not considered to carry any axial design load. To be considered load bearing, plain or fluted casings should be a minimum of 0.10 in. (2.5 mm) thick and have a cross-sectional area equal to at least 3 percent of the gross pile section.

4.5.3 *Reinforcement*—Reinforcement will be required in concrete piles primarily to resist bending and tension stresses, but can be used to carry a portion of the compressive load. For bending, reinforcement consists of longitudinal bars with lateral ties of hoops or spirals. When required for load transfer, the main longitudinal bars are extended into the pile cap, or dowels are used for the pile-to-cap connection.

The extent of reinforcement required at any section of the pile will depend on the loads and stresses applied to that section (4.2 and 4.3). Longitudinal bars used to carry a portion of the axial load can be discontinued along the pile shaft when no longer required because of load transfer into the soil, but not more than two bars should be stopped at any one point along the pile.

4.5.3.1 *Reinforcement for precast concrete piles*—Pile beam-column behavior is determined, to a great extent, by the reinforcement ratio. A lightly reinforced section, with approximately 0.5 percent steel, will have approximately the same cracking and yield moments, implying an extremely large reduction in stiffness after cracking leading to imminent collapse. At 1.0 percent steel, the yield moment would be more than twice the cracking moment, but the decrease in stiffness after cracking is still important. At 1.5 percent longitudinal steel content, the yield moment will be 3.5 to 4 times the cracking moment and the loss of stiffness at cracking is less important. Piles with less than 1.5 percent steel have been used successfully in some soil conditions, but great care is required in handling, transportation, and driving to avoid damage due to excessive bending stresses. The loss of stiffness at cracking can be extremely important for a pile in which column length effects become important, such as in piles extending through air or water. Because of this behavior, the committee recommends reinforced concrete piles that are driven to their required bearing values have a longitudinal steel cross-sectional area not less than 1.5 percent nor more than 8 percent of the gross cross-sectional area of the pile. If after a thorough analysis of the handling, driving, and service-load conditions, the designer elects to use less than 1.5 percent (of gross area) longitudinal steel, such use should be limited to nonseismic areas. At least six longitudinal bars should be used for round or octagonal piles, and at least four bars for square piles.

Longitudinal steel should be enclosed with spiral reinforcement or equivalent hoops. Lateral steel should not be smaller than W3.5 or D4 wire (ASTM A1064/A10M-10) and spaced not more than 6 in. (150 mm) on centers. The spacing should be closer at each end of the pile.

4.5.3.2 *Reinforcement for precast prestressed piles*—Within the context of this report, longitudinal prestressing is not considered load-bearing reinforcement. Sufficient prestressing steel in the form of high-tensile wire, strand, or bar should be used so that the effective prestress after losses is sufficient to resist the handling, driving, and service-load stresses (4.5.3.3). Post-tensioned piles are cast with sufficient mild steel reinforcement to resist handling stresses before stressing.

For pretensioned piles, the longitudinal prestressing steel should be enclosed in a steel spiral with the minimum wire size ranging from W3.5 to W5 (D4 to D5) (ASTM A1064/A1064M-10), depending on the pile size. The wire spiral should have a maximum 6 in. (150 mm) pitch with closer spacing at each end of the pile and several close turns at the tip and pile head. The close spacing should extend over at least twice the diameter or thickness of the pile, and the few turns near the ends are often at 1 in. (25 mm) spacing.

Occasionally, prestressed piles are designed and constructed with conventional reinforcement in addition to the prestressing steel to increase the structural capacity and ductility of the pile. This reinforcement reduces the stresses in the concrete and should be taken into account.

4.5.3.3 *Effective prestress*—For prestressed concrete piles, the effective prestress after all losses should not be less than 700 psi (4.8 MPa). Significantly higher effective prestress values (1000 to 1200 psi [6.9 to 8.3 MPa]) are commonly used and may be necessary to control driving stresses in some situations. Refer to 8.3.2, Item j, for additional comments on the use of higher effective prestress values.

4.5.3.4 *Reinforcement for CIP and CIS concrete piles*—Except for pipe and tube piles of adequate wall thickness that are not subject to detrimental corrosion, reinforcement is required in CIP and CIS concrete piles for any unsupported section of the pile and when uplift loads are present. Reinforcement will also be required for lateral loading except for very small lateral loads where the concurrent minimum axial compression loads present are sufficient to develop the required flexural strength.

Unsupported sections should be designed in accordance with 4.3. Sufficient longitudinal and lateral steel should be used for the loads and stresses to be resisted.

Uplift loads can be provided for by one or more longitudinal bars extending through that portion of the pile subjected to tensile stresses. For pipe or tube piles, dowels welded to the shell or embedded in the concrete, together with adequate shear connectors, can be used to transfer the uplift loads from the structure to the pile.

For lateral loads, the pile should be designed and reinforced to take the loads and stresses involved with consideration given to the resistance offered by the soil against the pile, the pile cap, and the foundation walls, as well as the effect of compressive axial loads.

In general, the amount of reinforcement required will be governed by the loads involved and the design analysis. Except for uplift loads, not less than four longitudinal bars should be used. The extent of reinforcement below the ground surface depends on the flexural and load distribution analyses.

For auger-grout piles, the addition of a central reinforcing bar extending at least 10 ft (3 m) into the pile is recommended. This adds toughness to resist accidental bending and tension forces resulting from other construction activities.

4.5.3.5 *Stubs in prestressed piles*—Structural steel stubs (or stingers) are sometimes used as extensions from the tips of prestressed piles. Structural steel stubs most frequently consist of heavy H-pile sections, but other structural shapes, fabricated crosses, steel rails, and large-diameter dowels are also used.

Stubs can be welded to steel plates, which are in turn anchored to the pile. They are, however, most frequently anchored by direct embedment of the stub into the body of the precast pile. Design of the stub attachment requires special attention to provide for proper transfer of the forces between the prestressed pile and the stub. Heavy transverse ties or spiral reinforcement are needed around the embedded portion of the stub to provide confinement, and shear studs are sometimes used to aid in bond development. Vent holes through the web and flanges of the stub may be required to permit the escape of air and water, and thereby permit proper concrete placement (refer to 7.5.3.1, 8.7.2, and 8.7.3).

Table 4.5.3.6—Recommended clear cover for reinforcement

Type and exposure	Minimum cover, in. (mm)
CIS piles	3.0 (75)
CIP piles	1.5 (40)
Precast-reinforced piles—normal exposure*	1.5 (40)
Precast-reinforced piles—normal exposure, bars No. 5 (No. 16) and smaller	1.25 (35)
Precast-reinforced piles—marine exposure†	2.0 (50)
Precast-prestressed piles—normal exposure	1.5 (40)
Precast-prestressed piles—marine exposure†‡	2.0 (50)

*A cover on the spiral of 7/8 in. (22 mm) for 10 in. (250 mm) diameter piles and 1-3/8 in. (35 mm) for 12 in. (300 mm) piles have been successfully used for precast piles that are cast vertically and internally vibrated from the bottom up as the concrete is placed, or using self-consolidating concrete.

†For marine exposures, refer to ACI 318-08 Section R7.7.6 when selecting concrete materials and cover values.

‡For prestressed piles under marine exposure, the required cover can range from 2 to 3 in. (50 to 75 mm). For certain types of centrifugally cast prestressed post-tensioned piles, a cover of 1.5 in. (40 mm) has given satisfactory service under 20 years of marine exposure in the Gulf of Mexico (Snow 1983). A 1.5 in. (40 mm) cover is recommended only if such piles are manufactured by a process using no-slump concrete containing a minimum of 658 lb of cement per yd^3 (390 kg/m^3) of concrete.

4.5.3.6 *Cover for reinforcement*—The minimum recommended clear cover for any pile reinforcement is summarized in Table 4.5.3.6 for various pile types and exposure conditions.

4.5.4 *Concrete for CIP and CIS concrete piles*—The designer should consider the factors affecting concrete placement in CIP and CIS piles when preparing specifications for this kind of work. This includes things such as proportioning of the concrete to give a slump in the 4 to 6 in. (100 to 150 mm) range or suitable flow cone values for auger-grout piles and placement methods (refer to 6.1, 6.5, and 8.6).

4.5.5 *Pile-to-pile cap connections*—Pile caps are designed to resist the factored forces (4.2), considering flexure, shear, and development of reinforcement. Chapter 15 of ACI 318-08 defines the appropriate critical sections for these effects and refers to other parts of ACI 318 for details. For nonseismic cases when the pile forces are primarily compressive, the piles are commonly embedded 3 to 6 in. (75 to 150 mm) into the bottom of the cap and the reinforcing steel mat is placed 3 in. (75 mm) above the tops of the piles. The pile cap is made large enough to provide significant lateral cover to the piles, perhaps 10 in. (250 mm) minimum, with consideration of the probable tolerances on pile locations. Test data cited by Souza et al. (2009) suggest that the minimum lateral cover should be at least the pile diameter or width. When the cap and piles are subjected to significant lateral loading, somewhat deeper pile embedment in the cap may be required to accommodate the transfer of pile-head moments and shears. Placement of the lower-mat reinforcement below the pile top can become desirable when high pile-head fixity is desired.

The CRSI Handbook (2008) has both design tables and illustrations of typical details. The *ACI Design Handbook* (SP-17 [Saatcioglu 2009]) also considers pile caps.

Earlier ACI Codes often led the designer to consider deep-beam behavior for the shear design, but ACI 318-08 leads one to strut-and-tie methods of analysis and design, as described in ACI 318-08 Appendix A. Adebar et al. (1990), Siao (1993), and Souza et al. (2009) specifically address pile caps using strut-and-tie concepts.

Section 5.5.4 presents more discussion of the design and behavior of cases where there are significant moments, tension forces, or both that should be transmitted between the piles and pile caps. That section is concerned with seismic effects, but the same general methods apply to cases where the moments or tensions are generated by wind or other lateral forces or by large applied moments from dead and live load effects.

4.5.6 *Pile integrity investigations*—Abnormalities, defects, and damage to concrete-pile cross sections can develop during pile construction due to installation procedures, ground conditions, or both. Previously installed piles can be damaged by lateral or uplift forces induced by subsequent nearby construction activities. Environmental conditions can also lead to changes in pile structural properties during their service life, such as when the piles are exposed to saltwater or aggressive groundwater conditions. The presence of defects and damage in concrete piles can impact both the structural and geotechnical ability of the pile to support load. When defects or damage are suspected, it is often necessary to investigate the nature of the defects or damage and assess their impact on the structural and geotechnical performance of the affected piles, so that appropriate remedial measures, if necessary, can be developed.

CIS concrete piles are particularly susceptible to the development of abnormalities or defects during construction because concrete is placed in direct contact with the soil and exposure to the groundwater conditions. Most CIS piles will have some natural variations in cross-sectional area along the pile as a result of the interaction of the drilling and concrete placement methods with the varying ground conditions. Construction defects can also develop in CIP concrete piles when the casings are damaged during installation or as a result of improper concreting procedures. The potential for development of defects is increased in both CIS and CIP piles when the concrete is placed through water or drilling fluids, or when heavy reinforcement impedes concrete placement. Construction defects can also develop in precast piles (1.2.1) when the pile installation stresses are excessive.

Numerous methods have been developed over the last 40 or more years to assist in investigating the nature and locations of defects or damage in concrete piles. Most of these methods are based on an interpretation of electronic signals reporting the energy transmission or reflection response to excitations. As noted in 3.3.2.3, the recorded dynamic measurements of stain and acceleration during pile driving or during restriking of previously installed piles can be examined for evidence of changes in pile properties along the pile length. Pile driving induces high stresses and strains in piles and is generally referred to as high-strain testing (ASTM D4945-08). Another group of testing methods available, collectively referred to as nondestructive testing (NDT) methods, uses the interpretation of electronic signals induced by low-energy and low-strain excitations.

ACI 228.2R-98 presents a detailed review of various NDT methods and an extensive bibliography to which the reader is referred for additional technical background on NDT. ACI 228.2R-98 Section 2.3 specifically covers stress-wave methods for deep foundations, and ACI 228.2R-98 Table 2.3 summarizes the general advantages and limitations of stress-wave methods for deep foundations. The deep foundations subcommittee of ASTM Committee D18 has prepared standard test methods for the pulse-echo and transient-response methods (ASTM D5882-07) and for the ultrasonic crosshole method (ASTM D6760-08). Refer to Chernauskas (2004) for more information on NDT and evaluation.

Although much of the current knowledge on using NDT for the investigation of deep foundations has been derived from test programs on drilled piers, which are generally of larger diameter than the members covered by this report (refer to 1.2.7.1), the general principles are still applicable to investigations of imperfections or damage in concrete piles. With their long lengths, concrete piles present special problems for NDT methods based on energy transmission or reflection response induced by low-energy and low-strain excitations. Interference or noise in these electronic signals, which complicate NDT interpretation, can result from pile splices; casing corrugations, steps, and tapers; and pile reinforcement. Minor natural cross-sectional abnormalities in CIS piles can also complicate interpretation. In addition, soil stiffness and damping can place limitations on the pile lengths that can be examined by the various methods.

The keys to minimizing the development of imperfections in concrete piles are the selection of pile types appropriate for the particular ground conditions, use of appropriate installation procedures for the particular pile type (Chapter 8), and implementing an appropriate program for inspection and monitoring of the manufacture and installation of concrete piles. A properly maintained log of concrete pile installation provides not only information about the geotechnical capacity, but also significant information about the integrity of driven or drilled piles.

Test programs using various NDT methods and high-strain methods have demonstrated that interpretations of integrity testing methods can result in false results (Holeyman et al. 1988; Samman and O'Neill 1997; Macnab et al. 2000). Although integrity tests can result in useful information to assist in an evaluation of pile integrity, the use of NDT data as the sole basis of evaluation is not recommended. In addition to information provided by properly documented pile installation records and records of subsurface conditions, other testing such as coring, high-strain testing, or pile extraction may also be required to evaluate the NDT results and reach an appropriate conclusion on pile integrity. The ASCE deep foundation committee (Macnab et al. 2000), the ASTM deep foundation committee (ASTM D6760-08), and the DFI auger-cast pile committee (Frizzi 2003) have reached similar conclusions.

CHAPTER 5—SEISMIC DESIGN AND DETAILING CONSIDERATIONS

5.1—Introduction

This chapter focuses only on earthquake-induced loadings on piles. However, dynamic loads on piles can also be initiated by supported machinery, by operational loadings such as ship impacts, by man-made dynamic events such as accidental or intentionally planned blasts, by construction activities, and by other natural events such as wind and wave loadings, which are not covered in this chapter (refer to 4.2.1).

The behavior of pile foundations and their supported structures during an earthquake is a complex soil-pile-structure interaction problem, influencing the conditions (loads and deformations) the piles should sustain and both geotechnical and structural aspects of pile foundation design. The following sections present an overview of some effects of seismic activity on pile foundations (5.2), observations on seismic-pile behavior (5.3), geotechnical and structural design considerations (5.4), and some comments on seismic structural-detailing requirements for piles imposed by model codes for seismic zones (5.5).

Seismic risk and seismic design requirements have been described in various ways. The previous version of this report, and ACI 318 through the 2005 version, referred to cases of moderate and high seismic risk. The 1997 Uniform Building Code (UBC) seismic provisions for foundations were based on specified seismic zones (numbered 0 through 4) for the site location. The UBC contained no special seismic requirements for piles in Seismic Zones 0 and 1 (also called regions of low seismic risk) or Zone 2 (also called regions of moderate seismic risk). For Zones 3 and 4 (also called regions of high risk), the UBC required special treatment of piles, as will be discussed in the following. ACI 318-08 definitions of seismic risk are in terms of Seismic Design Categories (SDC) A through F, to be consistent with terminology in NEHRP (FEMA 2003a,b) reports and the International Building Code (IBC 1808.2.9-2006). The SDC is a classification assigned to a structure based on its seismic use group and severity of the design earthquake ground motions for the site. SDC A and B are low seismic risk cases requiring no or minimal special considerations for seismic pile design. SDC C corresponds to the former moderate seismic risk case, whereas SDC D, E, and F correspond to the former high seismic risk case. SDC E and F are for sites with very high accelerations (mapped maximum considered earthquake spectral response accelerations at 1-second period equal to or greater than 0.75 g), with Seismic Use Groups I and II being assigned to SDC E and Seismic Use Group III being assigned to SDC F. In this report, both the former seismic-risk classifications and SDC are used because the pile-detailing provisions of some model codes, organizations, and research predate the SDC classifications.

5.2—General seismic impacts on pile behavior

The distortions of the earth's crust during an earthquake can generally be categorized as either permanent ground displacements (PGDs) or transient ground shaking (TGS). Fault ruptures and general ground subsidence or upheaval can result in large PGD. Large PGD can also result from earthquake-induced ground settlement, lateral ground-spread movement or flow slides associated with liquefaction, and from earthquake-induced landslides. TGS motions occur as earthquake-induced ground stress waves propagate though the earth's crust.

During an earthquake, piles tend to move with the earth's crust and are distorted in a manner similar to the surrounding ground. Both the soil-mass properties and the pile-material properties and dimensions influence the geotechnical and structural behavior of a pile under the earthquake-induced ground displacements and motions. In the case of the earthquake-induced PGD, the piles will either experience permanent distortions (locked-in pile loads or strains) when the pile materials can accommodate the induced PGD, or structural damage when the induced displacements are larger than the pile materials can sustain. To evaluate the effects of earthquake-induced PGD on piles, it would be necessary to identify the PGD type(s) likely to develop, the potential magnitude of the ground displacements, and the pile strains and loads that would develop when the estimated ground displacements are imposed on the piles. The effects of these PGD actions are superimposed on the pile loads for the supported structure and the earthquake-induced structural loadings.

The preferred design approach is to avoid vulnerable areas where the large PGDs are likely to develop, because estimating the magnitude of earthquake-induced PGD involves considerable uncertainty. Sometimes it is possible to minimize pile damage by modifying the ground conditions by taking liquefaction or landslide remedial measures when avoiding such areas is not possible.

TGS motions during earthquakes also induce strains in piles, as the flexible piles are distorted to conform to the free-field strains imposed by the propagating ground waves. Evaluation of transient-ground wave effects on piles requires an estimate of the free-field strains, which are dependent on the form of the critical waves (shear, compression, and surface), the velocity of the propagating waves in the surrounding ground, the ground strength, and the magnitude of the peak-ground motions (accelerations, velocities, and displacements) and their attenuation with distance from the source. TGS-induced pile strains can then be estimated by imposing the free-field motions on the pile, in the simplest approximation, or by using dynamic soil-pile interaction programs that account for the influence of the pile flexibility and the soil-pile interactions with the ground motions. Seismic-induced pile motions, and ground motions where the pile caps or other foundation elements are embedded below the surface, excite the supported superstructures. Inertial forces generated by the oscillating superstructure produce dynamic forces (axial, shear, and moments) at its base that are transmitted to the substructure. The substructure and its piling will undergo additional dynamic movements as these inertial forces are transferred to the supporting soils. These two types of response are referred to in the literature as kinematic- and inertial-pile response, behavior, or loads. A general summary of seismic soil-structure interaction analysis methods and the kinematic and inertial response of pile-supported structures and a detailed reference list are presented by Gazetas and Mylonakis (1998). The analyses are complex and the kinematic and inertial forces are not generally in phase.

Piles have to be capable of resisting the imposed TGS kinematic and inertial forces and transferring them to the supporting soil without developing displacements that would be intolerable for the particular structure. All elements of the foundation, including the pile caps or other elements connecting the piles and structure, are designed to resist the kinematic and inertial forces plus any effects of PGD that might develop.

Observed behavior of piles during earthquakes, as discussed in the following, suggest that the highest damage rates are associated with areas where PGD develop. Although damage associated with earthquake-induced TGS can be less intense than PGD, TGS can occur over large areas removed from the fault zones. Absent large PGD, the structure-induced inertia forces will generally control seismic pile design rather than the TGS-induced pile strains.

5.3—Seismic pile behavior

5.3.1 *Liquefaction*—Ground motions during earthquakes can result in liquefaction when the soil profile contains saturated liquefiable granular soils. Methods of determining whether the soils at a particular site can experience liquefaction should be used whenever there is significant seismic activity (Kriznitsky et al. 1993; Ohsaki 1966; Poulos et al. 1985; Seed 1987; Seed and Harder 1990; Seed and Idriss 1971, 1982; Seed et al. 1983, 1985; Terzaghi et al. 1996; Tokimatsu and Seed 1987; Youd et al. 2001; ASTM D6066-96(2004)). If liquefaction can develop, its effects on the superstructure and foundation design should be evaluated.

The evaluation of liquefaction potential depends on both the earthquake magnitude and the peak ground acceleration that can be experienced, as well as on the nature of the subsoils at the particular site under study. In general, analyses of the liquefaction potential of soil deposits are made by comparing the soil cyclic-shear resistance to the soil cyclic-shear stresses induced by earthquake ground motions. Liquefaction is likely to be triggered under the earthquake ground motions at those locations within the profile where the cyclic-shear resistance is less than the seismic-induced cyclic-shear stress. The seismic-induced shear stress at a given depth within the soil profile is generally estimated based on the inertial force on the soil column above that depth (Seed and Idriss 1971, 1982; Seed et al. 1983). The cyclic-shear resistance can be estimated using correlations of in-place standard penetration test data and actual field cases where liquefaction did or did not occur (Seed et al. 1985; Seed and Harder 1990). These correlations are based on an adjusted 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), which is denoted by the symbol $(N_1)_{60}$. Liao and Whitman (1986) present a simple correction factor to adjust the measured *N*-value to a standard, effective-overburden pressure of 1 ton/ft^2 (96 KPa) that has been found to model the various relationships proposed (Seed et al. 1985; Seed and Harder 1990; Terzaghi et al. 1996). The Seed et al. (1985) and Seed and Harder (1990) correlations are between the cyclic shear-resistance ratio, the adjusted standard penetration test value $(N_1)_{60}$, and the percent fines for earthquakes with a magnitude M of 7.5 for generally level ground conditions where there are no initial static shear stresses in the soil. Seed et al. (1985) and Seed and Harder (1990) also present preliminary correction factors for adjusting the correlations to other earthquake magnitudes and to conditions where initial shear stresses are present. Tokimatsu and Seed (1987) present a method of estimating settlements of sands under earthquake shaking for level ground.

Liquefaction generally does not occur below a depth of approximately 30 ft (9 m) or above 50 to 60 ft (15 to 18 m). Liquefaction of soil strata can lead to a loss of soil strength in the liquefied strata; sand boils; ground fissures; ground settlement on level ground; and settlement plus lateral spreading and flow failures in sloping ground or where a lateral free face is present, such as near coastlines, adjacent riverbanks, or adjacent sunken roadways. Loss of strength in liquefied strata can reduce both axial- and lateral-pile capacity, reduce the lateral constraint against pile buckling under the imposed axial pile loads, and reduce lateral resistance of other substructure elements. Liquefaction is less likely to occur within a driven-pile group because of the soil compaction resulting from the pile driving. Liquefaction can occur around the perimeter of a pile group; therefore, under these conditions, the stability of the group should be evaluated. Ground settlements can induce negative skin friction (NSF or down-drag) loads on piles and other substructure elements in the liquefied zone and in nonliquefiable strata overlying the liquefied zone. Lateral spreading ground can induce large lateral pile displacements, lateral drag forces on piles and other buried substructure elements, and P-Δ effects on piles. These liquefaction effects can impose additional axial, flexural, and shears loads on piles, limiting the ability of piles to resist the normal structural and seismic inertial loads. If liquefaction has to be provided for, the pile-soil capacity should be developed below the zone of possible soil liquefaction.

5.3.2 *Observed pile behavior during earthquakes*—One of the more extensive earlier reports on pile behavior during an earthquake is the Ross et al. (1969) description of highway bridge damage during the 1964 Alaska earthquake. The heaviest reported pile damages were associated with either liquefied ground or lateral movements of abutments and approach fills toward the waterway. Damage to bridges founded on nonliquefiable soils or bedrock was moderate to none.

Cooperative U.S.-Japan geotechnical workshops (Hamada and O'Rourke 1992a,b; O'Rourke and Hamada 1992, 1994; Bardet et al. 1997; Celebri et al. 2004; Boulanger and Tokimatsu 2005) report observations of ground, drilled-pier, and pile behavior during earthquakes and discuss research needs on pile behavior during earthquakes. These documents also provide extensive bibliographies for further information. A detailed summary of these documents is beyond the scope of this report. Summaries of case histories on pile performance during earthquakes can be found in several research reports (Bardet et al. 1997; Meymand 1998; Bobet et al. 2001; Bhattacharya 2003; Boulanger et al. 2003). The nature of the reported seismic-pile damage generally consist of: 1) cracking to crushing or hinge formation at the pile-cap interface; or 2) cracking at a significant depth below the pile-cap interface corresponding to maximum moment points, to boundaries of large soil stiffness changes, or to changes in reinforcement.

Batter piles supporting bulkheads of wharves or bridge abutments have suffered distress because they tend to resist all of the horizontal force in the structure, leading to failure of either the piles or the pile cap (Roth et al. 1992; Bardet et al. 1997; Harn 2004; Schlechter et al. 2004). Longer, more flexible batter piles have performed better. Other seismic pile failures have occurred because of poor connection details between the piles and the cap, lack of adequate pile-section strength and rotational ductility, and because of faulty analyses.

The potential for pile damage is greatest for instances of liquefaction accompanied by lateral soil spreading and large, permanent, lateral pile displacements (large PGD). Observations of pile foundations during earthquakes have shown that piles in firm ground generally perform well, whereas the performance of piles in very soft or liquefied ground can range from excellent to poor, that is, suffer structural damage or excessive deformations (Ross et al. 1969; Bardet et al. 1997; Wilson 1998; Boulanger et al. 2003). Both shake-table modeling (Meymand 1998) and numerical analysis (Bobet et al. 2001) of piles in nonliquefiable profiles suggest that inertial-induced pile moments will generally exceed the kinematic moments unless the piles are lightly loaded. The inertial- and kinematic-induced pile forces are not in phase so that direct summation of the two effects is not a straightforward process (Dickenson and McCullough 2005).

The observed behavior of piles in liquefied and lateral-spreading ground has led to studies attempting to predict the lateral-drag forces on piles and evaluate pile-failure modes (O'Rourke et al. 1994; Wilson 1998; Dobry et al. 2003, 2005; Bhattacharya and Bolton 2004; Bhattacharya et al. 2004; Brandenberg et al. 2005; Boulanger and Tokimatsu 2005). Of particular interest is the pile damage reported in two cases that were not discovered until 20 to 25 years after an earthquake. One of these buildings—the four-story Niigata Family Court House—was in an area where lateral spreads of approximately 3.3 to 6.6 ft (1 to 2 m) occurred and the permanent, lateral ground displacements in the vicinity of the building ranged from approximately 2.6 to 4 ft (0.8 to 1.2 m). After the earthquake, the building was inclined approximately 1 degree. Minor repairs were made to the inclined floors, and the building was used without additional modification for 25 years. The reported damage in two piles beneath this building was observed 25 years after the earthquake when the building was replaced (Hamada 1992). The extent of the pile damage associated with initial pile installation and demolition is not known. Some reports have described this as a pile failure whereas others call it pile damage. Some reports attribute the pile structural damage to bending moments resulting from the lateral-spread forces and imposed pile displacements, whereas others suggest that P-Δ effects had a major role (O'Rourke et al. 1994; Dobry et al. 2003, 2005; Bhattacharya 2003; Bhattacharya and Bolton 2004; Bhattacharya et al. 2004; Brandenberg et al. 2005). This case points out the problems in defining the ductility that piles should be required to sustain and what constitutes seismic pile failure.

5.4—Geotechnical and structural design considerations

Earthquakes can induce axial, moment, and shear loads on pile foundations. Absent PGD conditions, loads are primarily inertially induced by the supported structure, although kinematic loads can become important for lightly loaded piles. In addition to nonseismic design requirements, piles in seismic regions should be designed to geotechnically resist the seismic forces that could reasonably occur under applicable service- or factored-load combinations of ACI 318 or other controlling codes. When evaluating the lateral geotechnical capacity of piles under seismic loading at nonliquefiable sites, the cyclic nature of the seismic loads should be considered (Davisson 1970b; Long and Vanneste 1994).

A majority of the reported pile damage cases referenced in 5.3.2 were in Japan where the earthquake activity is frequent and in locations dominated by man-made land or loose alluvial deposits. The potential for liquefaction under the seismic-design conditions should be investigated (5.3.1). It is desirable to either avoid liquefiable areas or reduce the liquefaction potential by modifying the ground. When such options are not possible, the potential axial and transverse geotechnical drag loads or displacements induced on the piling by settling and spreading ground need to be addressed. The geotechnical pile capacity should be evaluated with these drag loads and displacements superimposed on any earthquake-induced inertial loads and the normal structural loads that could act concurrently. The geotechnical capacity should be developed below any liquefiable zones, and pile testing and other pile installation controls should address this condition.

Piles in seismic zones are also structurally designed to resist the forces that could reasonably occur under applicable seismic service or factored load combinations of ACI 318 or other controlling codes, and to sustain imposed PGD where present. Pile regions without lateral soil support as a result of seismic actions should be designed as columns, with due consideration of any seismic-induced P-Δ effects. Concrete piles structurally designed and detailed to accommodate the thrust, moment, and shear loads imposed by the seismic loading combinations, to transmit these forces between the pile and pile-cap or other structure connections, and to accommodate the installation and handling forces associated with developing the required geotechnical capacities will often satisfy most model code reinforcing-detail requirements and perform adequately structurally.

In areas of seismic risk, however, designing piles or other structural members on the basis of strength alone may not be adequate. These members should also possess adequate ductility and, more importantly, ductility under fully reversed loading conditions, to accommodate the deformations that might occur. A general discussion of flexural ductility and an overview of some code-pile seismic detailing requirements to provided ductility are presented in the following section.

5.5—Seismic detailing of concrete piles

5.5.1 *General*—As noted in 5.3, areas of concentrated rotation can occur where the pile is connected to the pile cap and at points along the length of the pile, such as points of maximum moment under inertial loads or as the result of kinematic loading at interfaces between soil layers with significantly differing stiffnesses. An adequate description of analysis methods suitable for the computation of kinematic deformations and concentrated rotations is beyond the scope of this report. It is important that soil-structure interaction be properly accounted for in such an analysis, as failure to include soil-structure interaction effects can lead to unrealistically large curvature and rotation requirements for the piles. Kinematic analysis is complex and not routinely performed. To account for the uncertainty in the accuracy of the computed kinematic forces and displacements, minimum reinforcement requirements are often imposed to encourage ductile behavior in the affected regions of the pile. Most reinforced- and prestressed-concrete structural members have some inherent ductility, but this can be inadequate for seismic response and analysis purposes unless special measures are taken to enhance it.

Ductility is a measure of toughness and is a function of many factors. As used in this report, ductility is the capacity to undergo measurable amounts of inelastic deformation with little change in the forces causing deformation before reaching a failure state. Curvature or rotational ductility is important to seismic response. Ductility will decrease if the area of tensile reinforcement, its yield strength, or both are increased; if the axial compression force acting on a pile or column is increased; or if the concrete strength is decreased. Ductility will increase if compression reinforcement is added, if the concrete strength is increased, if the axial compression force is decreased, or if the compression zone of the member is provided with confinement reinforcement.

The most common example of using confinement reinforcement to provided ductility is the spiral required in spirally reinforced concrete columns according to ACI 318-08 Eq. (10-5), which is expressed, with slightly modified notation, in Eq. (5.5.1a).

$$\rho_s = 0.45\frac{f_c'}{f_{yh}}\left(\frac{A_g}{A_{core}} - 1\right) \tag{5.5.1a}$$

where ρ_s is the spiral steel ratio; f_c' is the compressive strength of concrete; f_{yh} is the yield stress of spiral reinforcement (in the past, f_{yh} was limited to 60 kip/in.^{2} [414 MPa], but the upper limit was increased to 100 kip/in.^{2} [689 MPa] in ACI 318-05 and later editions); A_g is the gross area of member cross section; and A_{core} is the area of core of section, to outside diameter of the spiral.

The spiral steel ratio ρ_s is a volume ratio relating the volume of steel in the spiral to the volume of concrete contained within the spiral

$$\rho_s = \frac{4A_{sp}}{d_{core}s_{sp}} \tag{5.5.1b}$$

where A_{sp} is the area of wire or bar used in spiral; d_{core} is the outside diameter of spiral; and s_{sp} is the spacing or pitch of spiral along length of member.

Equation (5.5.1a), which is referred to in this section as the ACI Spiral, was developed to assure ductile behavior of columns under static axial load. Tests and experience show that columns with this amount of spiral reinforcement exhibit considerable toughness and ductility (ACI 318-08 Section R10.9.3).

Experience from past earthquakes and from laboratory tests demonstrates that the ACI Spiral also provides significant ductility in flexural modes and provides a significant shear strength contribution. Although the ACI Spiral leads to ductile members, the selection of the spiral steel ratio, bar area, and spacing is unrelated to flexural or shear requirements but rather is related to axial compression considerations. Major improvements in flexural ductility can be obtained with lighter spirals than the ACI Spiral.

The ACI Spiral has been widely adopted for use in the design of building columns and bridge piers to resist major seismic forces and deformations where the goal is to provide flexural ductility. For example, the ACI Spiral is used in ACI 318-08 Chapter 21 for SDC D, E, or F cases, with a lower limit of

$$\text{Minimum } \rho_s = 0.12\frac{f_c'}{f_{yh}} \tag{5.5.1c}$$

The minimum ρ_s requirement of Eq. (5.5.1c) governs when the ratio of A_g/A_{core} becomes less than approximately 1.27, which occurs only in large columns.

Because the ACI Spiral was explicitly derived for circular spirals, it does not address the requirements for square or rectangular transverse reinforcement arrangements. The ACI 318-08 Chapter 21 requirements for SDC D, E, or F cases for square or rectangular transverse reinforcement are more empirical than for circular spirals, but the equations are similar in format. They are

$$A_{sh} = 0.3(sb_cf_c'/f_{yt})[(A_g/A_{ch}) - 1] \tag{5.5.1d}$$

$$\text{but not less than } A_{sh} = 0.09sb_c f_c'/f_{yt} \qquad (5.5.1e)$$

where A_{sh} is the total area of transverse reinforcement in the direction considered; s is the spacing of tie sets along length of member; b_c is the width of section in the direction considered; f_{yt} is the yield stress of transverse reinforcement; and A_{ch} is the area of core within transverse reinforcement, measured out-to-out of the reinforcement.

There are many detailed requirements on the arrangement of this transverse reinforcement, and it is required for the same distances as for the spiral. One key requirement is that the spiral or tie reinforcement be spaced so that the longitudinal reinforcement is laterally supported to prevent bar buckling.

5.5.2 *Transverse confinement reinforcement for piling*—Although the ACI Spiral provides excellent flexural ductility, it is extremely difficult to provide the resulting amount of spiral reinforcement in many practical pile cases, such as small piles or square piles with longitudinal reinforcement arranged in a circular pattern. This difficulty arises because the area ratio A_g/A_{core} is large for square members containing circular spirals. Octagonal piles have a significantly lower ratio of A_g/A_{core} than square piles of the same diameter and cover to the spiral, and are fairly extensively used. High concrete strengths also lead to large steel ρ_s requirements. It is not desirable to have the spiral pitch too small because it can impede concrete placement during manufacturing. Also, as the pitch becomes smaller, there is an increased tendency for the concrete cover outside of the closely spaced spiral to spall off during pile-driving operations. Hence, although the ACI Spiral is widely adopted for column design, its adoption for piling is less universal. Some examples of the seismic transverse confinement steel recommendations in various model codes and other organizations are summarized in the following subsections.

5.5.2.1 *Uniform Building Code 1997 provisions*—The Uniform Building Code (1997) adopts the ACI transverse reinforcement requirements, but for special seismic detailing requirements of concrete piles, it limits the transverse steel requirement to that required to satisfy Eq. (5.5.1b) for spiral or circular hoop reinforcement, or to satisfy Eq. (5.5.1e) for rectangular hoop reinforcement in nonprestressed concrete piling in Seismic Zones 3 and 4, or SDC D. For prestressed concrete piles, UBC prescribes minimum volumetric ratios ρ_s of spiral reinforcement of 0.021 for 14 in. (355.6 mm) piles and 0.012 for 24 in. (610 mm) piles, and linear interpretation between these values for intermediate pile sizes, unless a smaller value can be justified by rational analysis. This transverse reinforcement is required to extend below the pile cap to 1.2 times the flexural length, which is defined as the length to the first point of zero deflection.

5.5.2.2 *PCI 1993 provisions*—The PCI (1993) Committee on Prestressed Concrete Piling recommends minimum transverse confinement steel requirements for the ductile region of prestressed piles in low-to-moderate and high seismic risk areas, where soils are not subject to liquefaction. For detailing purposes, PCI defines the ductile regions as the full pile length for piles with lengths of 35 ft (10.7 m) less; and the greater of 35 ft (10.7 m) or the distance from the bottom of the pile cap to the point of zero curvature plus three pile diameters for piles with lengths greater than 35 ft (10.7 m). The 1993 PCI recommendations are repeated for reference.

Regions of low to moderate seismic risk (SDC C)—In regions of low to moderate seismic risk, PCI (1993) recommends the lateral reinforcement for prestressed concrete piles meet the following steel ratio

$$\rho_s = 0.12\frac{f_c'}{f_{yh}} \geq 0.007 \qquad (5.5.2.2a)$$

with two limits on materials:

- $f_c' \leq 6000$ psi (40 MPa)
- $f_{yh} \leq 85{,}000$ psi (585 MPa).

Regions of high seismic risk (SDC D, E, or F)—In regions of high seismic risk, PCI (1993) recommends the following minimum amounts of confinement reinforcement.

Reinforcement of circular ties or spiral

$$\rho_s = 0.25\frac{f_c'}{f_{yh}}\left(\frac{A_g}{A_{core}} - 1\right)\left(0.5 + 1.4\frac{P_u}{A_g f_c'}\right) \qquad (5.5.2.2b)$$

but not less than

$$\rho_s = 0.12\frac{f_c'}{f_{yh}}\left(0.5 + 1.4\frac{P_u}{A_g f_c'}\right) \qquad (5.5.2.2c)$$

where P_u is the maximum factored axial compressive load on pile with two limits on materials:

- $f_c' \leq 6000$ psi (40 MPa); and
- $f_{yh} \leq 85{,}000$ psi (585 MPa).

Reinforcement of square spiral or ties

$$A_{sp} = 0.3s_{sp}h_c\frac{f_c'}{f_{yh}}\left(\frac{A_g}{A_{core}} - 1\right)\left(0.5 + 1.4\frac{P_u}{A_g f_c'}\right) \qquad (5.5.2.2d)$$

but not less than

$$A_{sp} = 0.12s_{sp}h_c\frac{f_c'}{f_{yh}}\left(0.5 + 1.4\frac{P_u}{A_g f_c'}\right) \qquad (5.5.2.2e)$$

where A_{sp} is the total area of area of transverse reinforcement in the direction considered, and h_c is the cross-sectional dimension of pile core measured center-to-center of spiral or tie reinforcement; and with the limit that

- $f_{yh} \leq 70{,}000$ psi (480 MPa)

PCI also recommends that the center-to-center spacing of the transverse reinforcement in the ductile region not exceed the lesser of one-fifth of the pile diameter, six times the longitudinal-strand diameter, or 8 in. (203 mm).

The formats, but not the numerical constants, of the aforementioned PCI equations for prestressed concrete piles in high seismic risk regions (SDC D, E, or F) follow research conducted in New Zealand (Joen and Park 1990a, Pam 1988) and NZS 3101. The *PCI Bridge Design Manual* (PCI 2004) contains provisions closer to the 1982 New Zealand Code, together with modifications to that code suggested by Pam (1988). Pam (1988) recommended adding the effect of the prestressing force to the effect of the axial compression when finding the required confinement reinforcement. These provisions lead to much heavier spirals than required by the PCI (1993) formulations. Pam (1988) recommended a maximum spiral spacing of four times the longitudinal-strand diameter.

5.5.2.3 *ACI 318 provisions*—Before ACI 318-99, the ACI Code did not cover the design and installation of portions of piling embedded in ground (refer to ACI 318-95 Section 1.1.5). This was modified in the subsequent ACI 318 versions to the extent that ACI 318 provides requirements for concrete piling in high seismic risk regions or high seismic performance- or design-categories (ACI 318-08 Section 21.12.4). In abbreviated form, ACI 318-08 Section 21.12.4, requires:

- Continuous longitudinal reinforcement in the pile region resisting the design tension forces, and detailing of the reinforcement into the pile and pile cap.
- Tests demonstrating that grouted bars or dowels, when used as connections, develop at least 125 percent of the specified bar yield strength.
- Transverse reinforcement in accordance with ACI 318-08 Section 21.6.4 for a distance equal to the greater of five pile widths or 6 ft (1.83 m) below the pile top, or below the unsupported length when the pile penetrates air, water, or soil incapable of providing lateral pile support.
- The length of transverse reinforcement provided in precast concrete piles be sufficient to account for potential pile tip variations.
- Piles caps containing batter piles to be designed to resist the full compressive strength of the batter piles acting as short columns.

As can be seen from this listing, the ACI 318 requirements only deal with unsupported lengths of piling, the short zone below the pile cap where a plastic hinge might form, and the pile-to-cap connections.

5.5.2.4 *NEHRP 2003 and IBC 2006 provisions*—The International Building Code (IBC) was developed by the groups producing the three major model building codes (BOCA, UBC, and SBC) and was issued in 2000 with the intent of replacing the three model codes. The IBC piling provisions contained a collection of requirements derived from the three codes, and have undergone significant revision since 2000. The IBC seismic requirements for piling (IBC 1808.2.9-2006) closely, but not exactly, follow the NEHRP recommended provisions for seismic regulations for new buildings (FEMA 2003a,b).

IBC and NEHRP recommend special minimum transverse confinement provisions for piles in SDC C through F that differ for the zone of the pile just below the pile cap where a plastic hinge might form and at boundaries between soft or liquefiable soils and stiffer soils; the remaining length of the pile significantly affected by the flexural demand from earthquake motions; and the remaining portions of the piles below the flexural length. These recommended transverse-confinement reinforcement provisions for concrete piles vary based on pile type, and some provisions are dependent on the site class category and the site's susceptibility to liquefaction.

The various definitions of the flexural length, sometimes called the ductile length or region, where transverse-confinement reinforcement is required also varies with the pile type and SDC. For example, in the IBC:

- The flexural length is defined as the depth to the first point of zero deflection (IBC 1808.2.9-2006, applicable to all piles).
- For reinforced precast piles, the full pile length requires special transverse reinforcement.
- For prestressed concrete piles in SDC C, the upper 20 ft (6.1 m) of the pile appears to be defined as the ductile length with a recommended minimum transverse steel ratio, whereas the portion below 20 ft (6.1 m) requires only half of the recommended transverse steel ratio.
- For prestressed concrete piles in SDC D through F, the ductile region is defined as the greater of 35 ft (10.7 m) or three pile diameters below the first point of zero curvature, whereas only half of the recommended transverse steel ratio is required below the ductile region.
- For CIS and CIP piles in SDC C, the zone requiring special transverse confinement reinforcement is defined as the greater of the upper one-third of the pile, 10 ft (3 m), or that required by analysis.
- For CIS and CIP piles in SDC D through F, the flexural length is defined as the point where the required moment strength is less than 0.4 times the cracking moment strength of the concrete section.

NEHRP provisions are similar, but have slightly different wording, and with varying definitions of the zone where transverse confinement reinforcement is required.

The varying IBC and NEHRP definitions of the flexural length or ductile region for various concrete pile types conflict each other. A rational technical definition of the earthquake-induced flexural demand based on different pile types is needed. The IBC 1808.2.9-2006 definition based on the depth to the first point of zero deflection is identical to the UBC definition (refer to 5.5.2.1), whereas the IBC definition based on the first point of zero curvature for prestressed concrete piles is identical to the recommendations of PCI (1993); refer to 5.5.2.2. The definition based on the moment demand and the cracking moment strength appears to have originated in the NEHRP, but its logic is not obvious because they also use the first point of zero curvature definition for prestressed piles.

Because curvature is equal to M/EI, the first point of zero curvature also corresponds to the first point of zero moment. Hence, the flexural length definition based on zero curvature could lead to an underestimation of the ductile demand region when the pile head is other than pinned. On the other hand, the IBC prescribed definition of the flexural length as a minimum of 35 ft (10.7 m) (IBC 1809.2.3.2.2), for prestressed piles could greatly overestimate the flexural demand zone for inertial loading. Furthermore, there does not appear to be a rational reason for the imposition of a prescribed minimum of 35 ft (10.7 m) length for prestressed piles in SDC D through F while prescribing only a minimum of 10 ft (3 m) for CIS and CIP piles.

For prestressed concrete piles, IBC and NEHRP specifically state that ACI 318-08 Chapter 21 does not apply to this pile type and provide provisions for minimum transverse confinement reinforcement in the flexural length that follows PCI (1993) (that is, Eq. (5.5.2.2a) for SDC C piles and Eq. (5.5.2.2b) through (5.5.2.2e) for SDC D through F) except that an upper limit of $\rho_s = 0.021$ is indicated for spiral reinforcement and the minimum concrete strength f_c' is 5000 psi (34.5 MPa). For the portion of the pile below the flexural length, the recommended minimum transverse reinforcement is one-half of that recommended in the flexural length. For SDC C structures, the IBC does not provide special seismic limits on spiral or tie sizes and spacing requirements, so the static code provisions for prestressed piles apply. For SDC D through F structures, IBC provisions limit the maximum spacing of spiral and hoop reinforcement in the ductile zone to the lesser of one-fifth the pile width, six times the longitudinal-strand diameter, or 8 in. (200 mm). Note that the 8 in. (200 mm) criterion on the seismic spacing is greater than the 6 in. (150 mm) limitation imposed by the IBC nonseismic pile reinforcing requirements. This inconsistency likely developed from the use of NEHRP seismic pile provisions, which do not address nonseismic pile requirements. It is also of interest to note that IBC does not require more stringent transverse confinement requirements for prestressed piles in the potential high-moment-demand region near the pile-cap interface, which are imposed for other pile types.

For reinforced precast (nonprestessed) piles in SDC C structures, IBC provisions require transverse-confinement reinforcement within three pile widths below the cap consisting of closed ties or spirals of 0.375 in. (10 mm) or larger diameter spaced at the lesser of eight longitudinal bar diameters or 6 in. (150 mm). In the remainder of the pile, the tie or spiral spacing can be increased to the lesser of 16 bar diameters or 8 in. (200 mm). For precast piles in SDC D through F structures, IBC provisions require the transverse-confinement reinforcement within the three pile widths of the cap to be provided by closed ties or spirals in accordance with ACI 318-08 Sections 21.4.6.2 through 21.4.6.4, which would require that the reinforcement in this region satisfy Eq. (5.5.1a) and (5.5.1b) through (5.5.1e) for columns. For Site Classes A through D that are not subject to liquefaction, the spiral steel ratio can be limited to one-half of that required by Eq. (5.5.1a) and (5.5.1b). The maximum tie or spiral spacing in this zone would be the lesser of one-fourth of the pile width, six longitudinal bar diameters, or 4 to 6 in. (100 to 150 mm), depending on the pile width and transverse bar spacing (ACI 318-08 Section 21.6.4.3). At depths below the three pile diameters, the IBC transverse reinforcement requirements for SDC D through F structures are the same as SDC C structures (0.375 in. [10 mm] or larger ties or spirals spaced at the lesser of 16 longitudinal bar diameters or 8 in. [200 mm]). Again, the 8 in. (200 mm) spacing criteria on the seismic spacing in the length below three pile widths is greater than the 6 in. (150 mm) limitation imposed by the IBC nonseismic pile reinforcing requirements.

For uncased, auger grout, and concrete-filled shell piles, the IBC provisions for SDC C structures piles requires transverse reinforcement with 3/8 in. (10 mm) or larger closed ties or equivalent spirals spaced at the lesser of eight longitudinal bar diameters or 6 in. (150 mm) in the upper three pile widths, and spaced at no more than 16 longitudinal bar diameters in the remainder of the flexural length. For SDC D through F structures, transverse reinforcement in the zone within three pile widths of the cap is to be in accordance with ACI 318-08 Sections 21.6.4.2 through 21.6.4.4 (refer to Eq. (5.5.1b) through (5.5.1e)), except that for Site Classes A through D, which are not subject to liquefaction, the spiral steel ratio can be limited to one-half of that required by Eq. (5.5.1a) and (5.5.1b). Transverse reinforcement is also required to be a minimum of No. 3 (No. 10) bars for pile widths of 20 in. (500 mm) or less and No. 4 (No. 13) bars for pile widths greater than 20 in. (500 mm). In the flexural length below three pile widths, the required minimum spacing of the transverse steel is the lesser of 12 longitudinal bar diameters, one-half of the pile width, or 12 in. (300 mm). Note that the IBC definition of flexural length for these piles is different for the SDC D through F structures than for SDC C structures. IBC waives the transverse reinforcement requirements for concrete-filled shells that meet the shell confinement requirements of Chapter 4, Table 4.3.2.1.

For concrete-filled steel pipe and tube piles, the IBC does not mandate transverse reinforcement but requires a minimum wall thickness of 3/16 in. (5 mm) as opposed to the minimum 0.1 in. (2.5 mm) thickness for nonseismic loadings.

5.5.3 *Seismic axial reinforcement for piling*—Although ACI 318-08 Section 21.12.4.2 requires continuous longitudinal reinforcement in the pile region resisting the design tension forces, and detailing of the reinforcement into the pile into the pile cap, it does not prescribe a minimum longitudinal steel ratio. Hence, ACI 318-08 provides no special longitudinal steel provisions beyond those required for nonseismic loads. Similarly, PCI (1993) does not propose additional prestress in seismic regions. The IBC, however, imposes additional minimum longitudinal steel ratios for some pile types in SDC C through F structures, as follows:

- For reinforced precast piles, IBC provisions require a minimum longitudinal reinforcement equal to 1 percent of the concrete section for the full pile length in seismic SDC C through F structures, as opposed to the 0.8 percent IBC requires for nonseismic cases. Both of these values are less than the 1.5 percent recommended in this report to cover handling and installation conditions (refer to 4.5.3.1).
- For prestressed concrete piles, IBC requires no additional strand requirements other than the length-dependent, minimum effective prestress limits IBC imposes on nonseismic cases, some of which are less than

recommended in this report for nonseismic loading (refer to 4.5.3.2).

- For uncased, auger grout, and concrete-filled shell piles, where the IBC has no minimum longitudinal steel requirements for nonseismic cases, IBC provisions indicated longitudinal steel ratios of 0.25 percent for SDC C structures and 0.5 percent for SDC D through F structures. This longitudinal steel, which is to consist of a minimum four bars, is to extend throughout the flexural demand zone as indicated in 5.5.2.4.
- For concrete-filled pipe and tube piles, the IBC increases the minimum longitudinal steel ratio required in SDC C through F structures by increasing the minimum wall thickness to 3/16 in. (5 mm) from the nonseismic value of 0.1 in. (2.5 mm).

5.5.4 *Pile-to-cap connections*—The pile connections to pile caps or other structure elements are designed and detailed to resist combinations of pile-head axial, shear, and moment forces that can develop under the appropriate load combinations. In addition to the usual compression force, most seismic design cases can include significant pile-head moments and even significant tension forces. Moment-resistant connections, tension-resistant connections, or both, have been made by extending pile reinforcement into the cap; by adding dowels that extend into the pile cap and are grouted with epoxy or cement-based materials into drilled or preformed holes in the pile; by extending the pile a significant distance into the cap; or by some combination of these treatments. The connection is also designed be able to transfer the applicable factored shear loads.

Pile reinforcement can be exposed by chipping away concrete after driving, or by extending the reinforcement cage for CIS and CIP piles. The necessary lengths are governed by the development length in tension of the bars or strands being used. Bars are often hooked to reduce the required embedment lengths. Exposed strands have been frayed out (broomed) in the outer 6 in. (150 mm), have been bent 90 degrees in some tests, and have had other things done to enhance bond.

When dowel tubes or preformed holes are cast into the pile head to receive steel reinforcement as a tension connector to the pile cap, PCI (2004) recommends that the total area of the dowel tubes be less than 6 percent of the gross pile section, and that the termination points of the dowel tubes should be staggered by at least 1 ft (300 mm).

Tests of six pile-to-cap connections were conducted in New Zealand (Pam 1988; Joen and Park 1990b), using 15.75 in. (400 mm) prestressed concrete piles of octagonal cross section with 0.5 in. (12.5 mm) seven-wire longitudinal strands and added reinforcing bars in some cases. The transverse confinement spirals satisfied the 1982 New Zealand Code in a plastic hinge region (or approximately twice that suggested by Eq. (5.5.2.2b)) for a distance of 22 in. (560 mm) below the cap and approximately one-half that amount over the remaining test length. For two tests, the piles were extended 31.5 in. (800 mm) into the pile cap, with the embedded pile surface being roughened. An additional light spiral was placed around the pile stub before the cap concrete was cast. These two tests gave the best behavior, and the connection was easiest to build. Three of the tests were on piles that had the concrete removed to expose the strands. In two cases, concrete was removed for a distance of 23.6 in. (600 mm) and the concrete pile extending 2 in. (50 mm) into the cap, for a total embedment of 25.6 in. (650 mm). A spiral equal to that in the precast pile was placed around the exposed anchorage strand length. In the third case, a strand length of 33.5 in. (850 mm) was exposed and, in addition, had a bond enhancement called an olive. This involved unwinding the outer six wires of the strand for approximately 12 in. (300 mm), slipping a 1/2 in. (12.5 mm) ID hex nut over the central wire and then twisting the six outer wires back into place. These three piles all behaved adequately, with ductility factors of at least 9. The third case, with the longer exposed length and the olive was strongest, but all reached or exceeded the theoretical moment capacities. The authors felt that the rather heavy spirals enhanced the bond capacity of the exposed strands as well as the capacity of the piles that were merely extended into the cap. The sixth test had four 0.79 in. (20 mm) diameter reinforcing bars set into 1.57 in. (40 mm) diameter holes drilled 20.9 in. (530 mm) into the top of the pile. The bar extensions into the cap had 90-degree standard hooks. The strength exceeded the computed strength and the ductility factor was approximately 12. The larger deformations were concentrated at the interface of the pile to cap and a very large crack developed. The authors noted that the available space for dowels within the spiral could severely limit the moment capacity that could be developed.

The building codes have some requirements for pile-pile cap connections. For SDC D, E, or F, ACI 318 requires that if the forces are transferred by post-installed dowels, the grouting system shall be demonstrated by test to be capable of developing 125 percent of the specified yield stress.

Tests of connections of piles to wharf slabs, under simulated seismic conditions, have been conducted at the University of California at San Diego (UCSD) and the University of Washington (UW). The UCSD tests included one 24 in. (610 mm) diameter reinforced concrete pile (Sritharan and Priesteley 1998) and two 24 in. (610 mm) diameter octagonal prestressed piles (Krier et al. 2008). Anchorage to the wharf slab in each test involved bulb-ended bars. The bulb was a hot-forged upset approximately 1.5 times the diameter of the bar.

In one prestressed pile, designed for nominal seismic forces, four No. 9 (No. 29) bars were grouted into 60 in. (1.52 m) preformed holes with the bulb-end extending 15 in. (380 mm) into a 24 in. (610 mm) thick slab. The pile end extended 2 in. (51 mm) into the slab. The second prestressed pile and the reinforced concrete pile, both designed for major seismic forces and deformations, each had eight No. 10 (No. 32) bulb-ended bars that extended approximately 27.5 in. (700 mm) into 36 in. (914 mm) thick slabs. In addition, each specimen had eight No. 9 (No. 29) bond bars. The bond bars were 24.5 in. (622 mm) long bars with a bulb-end at the lower end and a bar head (ASTM A970/A970M-09) at the top, with the head placed with 3 in. (75 mm) clear cover from the top of the slab. The overlap of the two bar sets was approximately 18 in. (460 mm). The piles had substantial spirals and the bar embedments into the slabs were also enclosed in spirals. All three tests reached large ductility values in the simulated seismic tests.

The UW tests (Roeder et al. 2001, 2005) included eight pile-wharf slab connections using 16.5 in. (419 mm) octagonal piles connected to 24 in. (610 mm) thick slabs. Two connections were modeled as reinforced concrete pile extensions and the other six were prestressed piles. All tests used doweled connections in various arrangements. One test had a bond bar arrangement similar to the UCSD tests and one had T-headed dowel bars similar to the ASTM A970/A970M-09 heads. One pile extension case had a spiral surrounding the dowels in the slab, and all cases had spirals in the pile sections. Five of the prestressed specimens had axial loads equal to approximately 10 percent of the pile axial load strength during the simulated seismic tests.

The two reinforced concrete extensions and the prestressed pile without axial load all reached large deformations while maintaining the full moment capacity in the simulated seismic tests. The five prestressed piles with axial load all suffered significant loss in moment capacity at larger deformations. There are differences that may be related to details of the anchorage to the deck, but the predominant effect was of the axial load.

The 2006 IBC assumes connections will be either by exposed pile reinforcement or by post-installed dowels, with no specific recognition of the case of the pile embedded into the cap. The required reinforcement embedment lengths are the development lengths in compression or tension, as appropriate to the case. SDC C cases require confinement reinforcement surrounding the anchored steel, with at least half the reinforcement required for a column. SDC D, E, or F cases consider tension and bending separately. A pile in tension is to be anchored to develop the smaller of the tensile strength of the pile reinforcement, the pile-soil uplift capacity multiplied by 1.3, or the maximum force from the appropriate factored loads. A pile in bending is to be anchored to develop the smaller of the nominal strength of the pile or the factored loads from the appropriate load combinations.

Pile caps are obviously designed to resist all of the applied forces. The 2006 IBC specifically requires that connections between a pile cap and batter piles be designed for the "nominal strength of the pile acting as a short column." ACI 318 contains a similar requirement.

5.5.5 *Needed research*—Most of the reversed bending tests of piles have been conducted on octagonal pretensioned members (Falconer and Park 1982; Park and Falconer 1983; Sheppard 1983; Banerjee et al. 1987). Tests of square members with round and square reinforcement patterns and round members of both reinforced and prestressed concrete are needed, along with supporting analytical work. These tests should include a full range of confinement reinforcement ratios or areas, and should include tests with and without axial loads. As of 2010, no tests have included tension thrusts. Both solid and hollow members should be considered. In addition to studies of the rotation capacities that are possible from various members, studies of the rotational demands or requirements that can be imposed by the supported structure with various soil profiles are needed.

5.6—Vertical accelerations

Experience from the 1994 Northridge earthquake in California reveals that at/near the epicenter, vertical accelerations approached the magnitude of horizontal accelerations. This is significant because accelerations on the order of 1.0 g were recorded. The ramifications of high vertical accelerations should be considered by the structural engineer relative to piling because severe axial overloading of piles can occur under earthquake conditions. One example is piles of wharves or platforms in container terminals, where laden containers are stacked almost permanently. In geographic areas where high vertical accelerations are possible, it may be advisable to consider another case of loading that codes do not now consider, namely, normal service axial load plus that produced by vertical seismic accelerations.

CHAPTER 6—MATERIALS

6.1—Concrete

Durable concrete is essential to produce successful pile foundations. Although cement is the principle constituent in producing durable concrete, consideration of other requirements beyond cement type such as water-cementitious material ratio (*w*/*cm*), admixtures, strength, air entrainment, adequate consolidation, adequate cover over reinforcement, and curing are essential to producing a durable concrete structure. Refer to ACI 201.2R-08 for information on concrete durability.

6.1.1 *Cementitious materials*

6.1.1.1 *Cement*—Portland cement should conform to either ASTM C150/C150M-11 (Types I, II, III, or V) or ASTM C595/C595M-11 (Types IS, IS[MS], P, or IP). Selection of the appropriate specification and cement types for a particular concrete pile project should be based on the environment to which the piles are to be exposed and the durability requirements given in ACI 318-08 Chapter 4.

The principal consideration in the selection of cement type for sulfate resistance in ACI 201.2R-08 is the tricalcium aluminate (C_3A) content. For example, concrete piles with moderate exposure to sulfate-containing soils or water, that is, soils containing 0.1 to 0.2 percent by weight of water-soluble sulfate (SO_4) or water containing 150 to 1500 ppm sulfate, should be made with cement containing not more than 8 percent tricalcium aluminate, such as ASTM C150/C150M-11 Type II cement or moderate sulfate-resistant blended cement (MS). Similarly, for severe sulfate exposure, use ASTM C150/C150M-11 Type V cement, which contains no more than 5 percent tricalcium aluminate, and for very severe sulfate exposure, use ASTM C150/C150M-11 Type V cement with a pozzolan or slag admixture.

In areas where Type V cement is not available, a comparable substitution should be specified—for example, Type II with tricalcium aluminate less than 8 percent with Type F fly ash at approximately 20 to 50 percent by mass of the total cementitious material (refer to 6.1.1.2). Similarly, the proportion of silica fume can be expected to be in the range of 7 to 15 percent by mass of the total cementitious material and the proportion of slag can be expected to be in the range of 40 to 70 percent by mass of the total cementitious material. When blending multiple pozzolans and slag, the individual proportions of each can be expected to be less than the ranges referenced previously.

Concrete in seawater environments, with portland cement containing 5 to 10 percent tricalcium aluminate, has been reported to show less cracking due to steel corrosion than cement with less than 5 percent tricalcium aluminate (ACI 201.2R-08). Therefore, if seawater rather than fresh water is expected, the use of Type V cements to address sulfate resistance is not recommended because, even though the low tricalcium aluminate cement increases the sulfate resistance, it also increases the risk of steel corrosion. This condition is accounted for in ACI 318-08 Section 4.2.1, which classifies seawater as moderate sulfate exposure, even though seawater generally contains sulfate in exceeding the moderate exposure limits.

6.1.1.2 *Fly ash*—If fly ash or other pozzolans (refer to 6.1.4) are used, then the amount recommended by ACI 211.4R-08 can be used. Because the fly ash content affects the rate of strength development, practical considerations may limit the amount of fly ash used for precast pile applications to less than permitted by ACI 211.4R-08. Some state highway department specifications also place limits on the use of fly ash in piles. Fly ash or other pozzolans should conform to ASTM C618-08. The calcium-oxide content of the fly ash used in production concrete should be no more than two percentage points higher than the calcium oxide content of the fly ash used in the approved trial mixtures.

6.1.1.3 *Slag cement*—Slag used in concrete mixtures should conform to the requirements of ASTM C989-10. The calcium-oxide content of the slag used in production concrete should be no more than two percentage points higher than the calcium oxide content the slag used in the approved trial mixtures.

6.1.2 *Aggregates*—Concrete aggregates should conform to ASTM C33/C33M-11. Aggregates should be examined petrographically for potential alkali reactivity in accordance with ASTM C295/C295M-11. Aggregates that are identified as potentially reactive should be tested for alkali reactivity by evaluating expansion potential in accordance with ASTM C227/C227M-11. Aggregates that fail these tests but have been shown by special tests or actual service to produce concrete of adequate strength and durability can be used if authorized by the engineer. In general, avoid the use of reactive aggregate in concrete piles. Further information on the potential for adverse reactions between the alkali of the cement and the silica in the aggregates is contained in ACI 201.2R-08, ACI 221R-96, and ACI 221.1R-98.

6.1.3 *Water*—Water used for curing, washing aggregates, and mixing concrete for concrete piles should conform to the requirements in ACI 318-08 Chapter 3.

6.1.4 *Admixtures*—Specific information on admixtures is given in ACI 201.2R-08, ACI 212.3R-10, and ACI 212.4R-04.

6.1.4.1 *Air-entraining admixtures*—Concrete for piles that will be exposed to freezing and thawing should contain an air-entraining admixture. The use of air-entraining admixtures, however, does not reduce the need to protect fresh concrete from freezing conditions during the early stages of hydration. Such freezing can severely damage the strength and durability of the concrete.

Air-entraining admixtures used in concrete for piles should conform to ASTM C260/C260M-11. The amount of air entrainment and its effectiveness depends on the admixtures used, the size and nature of the coarse aggregates used, and other variables. Too much air in the concrete mixture will lower the concrete strength, and too little air reduces its resistance to freezing-and-thawing damage. ACI 201.2R-08 recommends that the entrained air content of fresh concrete be in the range of 3 to 7 percent, depending on the size of coarse aggregate and on the severity of exposure.

6.1.4.2 *Other admixtures*—Water-reducing admixtures, retarding admixtures, accelerating admixtures, water-reducing and retarding admixtures, and water-reducing and accelerating admixtures should conform to ASTM C494/C494M-11 or ASTM C1017/C1017M-07.

6.1.4.3 *Chlorides*—The use of admixtures that contain significant amounts of chloride should be minimized in reinforced concrete, particularly in marine environments. The use of chloride-free admixtures may be warranted if the total chlorides and water-soluble chlorides that may be present in the concrete would exceed the recommended limits given in ACI 201.2R-08. Significantly lower limits are applied to prestressed concrete than to reinforced concrete.

6.1.4.4 *Calcium chloride*—Calcium chloride should not be used as an admixture in concrete that will be exposed to severe sulfate-containing solutions as defined in ACI 318-08 Chapter 4 and should never be used with prestressed concrete.

6.1.5 *Water-cementitious material proportions*—The *w/cm* proportioning is related to the specified strength and predicted durability.

6.1.5.1 *Guidelines*—Guidelines for selecting appropriate *w/cm* are given in ACI 211.1-91 and ACI 301-10. Limitations on the *w/cm* for durability requirements are addressed in ACI 318-08 Chapter 4.

The effects of lowering the *w/cm* include increases in strength, durability, and resistance to sulfate attack. The lower permeabilities of concretes with low *w/cm* increase the resistance to penetration of fluids. This results in an increased resistance to the degrading effects of assorted chemical agents and to freezing-and-thawing cycling effects. The use of water-reducing agents, high-range water reducers, and pozzolans can help lower the *w/cm* of a mixture.

6.1.5.2 *Cement content*—The amount of cement in a mixture is an important variable. In the past, the recommended minimum cement content of a concrete pile mixture was 564 lb/yd^3 (335 kg/m^3) because of durability considerations. In aggressive environments, such as for marine usage, a minimum of 658 lb/yd^3 (390 kg/m^3) was recommended. For conventional structural concrete, 752 lb/yd^3 (445 kg/m^3) was considered a reasonable maximum.

Reduced coarse-aggregate concrete mixtures, containing approximately 800 lb/yd^3 (475 kg/m^3) of coarse aggregate and with as much as 846 lb/yd^3 (500 kg/m^3) of cement, have been reported (Raymond International 1970; Snow 1976; Fuller 1983). These mixtures were developed for some of the more difficult placement conditions encountered with CIP piles, such as long piles with corrugated shells (8.6.4).

These historical cement contents may not be appropriate for concretes containing current pozzolans or admixtures, or both. The *w/cm*, cement type and chemical composition, pozzolan content and type, and air content will be more predictive of durability than cement content alone.

The concrete mixture proportions may need to be adjusted in the case of pumping or tremie placement to produce a fluid, workable mixture for the particular conditions. Generally, rich mixtures (564 to 752 lb/yd^3 [335 to 445 kg/m^3]) of cement, higher slumps (6 to 8 in. [150 to 200 mm]), smaller-sized coarse aggregates (3/4 in. [20 mm] maximum size or less), and higher proportions of the fine aggregate (43 percent or more sand) are used for tremie placement. A plasticizing admixture can also be beneficial. Anti-washout admixtures may also be helpful for tremie-placed concrete.

6.1.5.3 *Water content*—The correct water content is important to a concrete mixture. Too little water results in placement difficulties, whereas too much water can seriously decrease strength and durability characteristics. The optimum quantity is the least water that will provide proper hydration and permit effective placement of the concrete. The desired workability should not be achieved merely by the addition of water. The durability of the finished product decreases with an increasing *w/cm*.

6.1.6 *Control tests*

6.1.6.1 *Slump tests*—Slump tests made in accordance with ASTM C143/C143M-10, flow tests made in accordance with ASTM C939-10, and slump-flow tests of self-consolidating concrete made in accordance with ASTM C1611/C1611M-09 are measures of the workability of concrete mixtures. It is recommended that job specifications that refer to ASTM C1611/C1611M-09 include an upper limit on the Visual Stability Index described in Appendix X1 of that specification. Slump test results are loosely related to the total water content of the mixture. The slump of a concrete mixture should be limited to the minimum slump that is consistent with the placement requirements and methods. Slump tests should be performed at the time of placement when strength samples are obtained or whenever the possibility of an inappropriate slump exists. Refer to 6.5 for monitoring fluidity of grout mixtures for auger-grout piles.

6.1.6.2 *Air content tests*—The presence of entrained air should be verified during placement when strength samples are obtained or when an inappropriate air content is suspected. Entrained-air tests should be made in accordance with ASTM C173/C173M-10 or ASTM C231/C231M-10, as applicable. Indicators, such as the Chase meter, should be frequently calibrated for a given mixture for a specific project.

6.1.6.3 *Unit weight measurements*—Unit weight measurements should be performed with each set of strength samples and air content measurement. The unit weight of freshly mixed concrete is determined in accordance with ASTM C138/C138M-10. The unit weight measurement is a direct measurement of the yield of the concrete mixture and may serve as a secondary measurement of the air content.

6.1.6.4 *Strength tests*—Compressive strength tests should be performed on samples obtained at the time of placement. At least one set of test specimens should be obtained for each 50 yd^3 (40 m^3) of concrete placed, with at least one set for each day's production. Samples should be obtained in accordance with ASTM C172/C172M-11 and ASTM C31/C31M-10 and tested in accordance with ASTM C39/C39M-11. A set consists of at least three test specimens. Those cylinder specimens used to control transfer of prestressing force and early handling conditions for piles should be field-cured under the same conditions as the concrete piles.

The test age for concrete compressive strength should be 28 days or the age designated for determination of the specified value of f_c' or, when specified, at the earliest age at which the concrete can receive its full load or maximum stress. The use of fly ash or heavy dosage rates of admixtures can slow the strength gain of concrete, requiring strength testing at a later age, such as 56 days. For prestressed concrete, additional tests are required to establish the strength at the time of prestress transfer. Additional specimens for early tests (7 or 14 days) may also be desirable with CIP or CIS piles to provide early warnings of any potential concrete quality problems.

For augered CIP piles, strength tests of the grout are usually made on 2 in. (50 mm) cubes in accordance with ASTM C109/C109M-11. One gang mold will typically have three cube compartments. A set of six to nine cubes is typically made, with two or three cubes tested at 7 days, two or three cubes at 28 days, and the remaining cubes used for early strength tests or held in reserve for testing at a later date if required. The failure stresses for tests on cube specimens are approximately 15 percent higher than for tests on cylinder specimens used for determining f_c'.

6.1.6.5 *Maturity testing*—The maturity method is a procedure for estimating concrete strength as represented by the relationship between degree hours of curing and compressive strength. This method of estimating strength has gained use by many transportation agencies for the strength acceptance of concrete. The maturity method when performed in accordance with ASTM C1074-11 is an approved procedure for determining the termination curing of concrete by ACI 308.1-11. Refer to ACI 308R-01 for additional information. Accordingly, this method could be useful for the purposes of determining when curing can be terminated and when forms can be removed. Final strength verification should be by tests of cylinders in accordance with ACI 318-08 Section 5.1.2. Brettmann et al. (2004) have suggested that the maturity method can be a useful tool for evaluating the early strength gain in auger-grout piles.

6.1.6.6 *Curing temperatures*—Proper precautions and monitoring should be performed to control the temperature rise in the freshly placed concrete as well as the maximum temperature the concrete will achieve during the initial curing. Information on heating rates, maximum temperatures, and cooling rates are given in 7.5.5.3.

6.2—Grout

Grout for auger-injected piles, preplaced-aggregate piles, and drilled piles (1.2.7.3 through 1.2.7.5) should consist of a mixture of approved cement, fine aggregate, admixtures, and water. The grout should be mixed to provide a grout capable of maintaining the solids in suspension. This mixture should also be capable of being pumped without difficulty. The mixture should be capable of laterally penetrating and filling any voids in the soils or preplaced aggregates. Admixtures should include those pozzolans and grout fluidizers possessing characteristics that will increase flowability of the mixture, improve cement dispersion, and neutralize setting-shrinkage of the cement mortar. If grout expansion is considered appropriate for the application, the expansion should be limited to 4 percent. Grout used to fill prestressing ducts of post-tensioned prestressed piles usually consists of portland cement, admixtures, and water proportioned to produce a pumpable mixture.

For auger-grout or cement-injected piles (1.2.7.3 and 1.2.7.4), checking the flow rate of the grout is a quality-control tool for monitoring the fluidity of the mixture. The flow rate is determined as the time of efflux for a specific volume of grout from a standardized flow cone. The flow cone specified in ASTM C939-10 has a volume of 105.3 in.^{3} (0.001725 m^3) and a discharge tube diameter of 0.5 in. (12.7 mm). The discharge diameter of the standard ASTM C939-10 flow cone cannot be modified. The U.S. Army Corps of Engineers standard for measuring the flow of grout (CRD-C611) is essentially the same as ASTM C939-10.

The ASTM C939-10 flow cone was intended for use with grouts having efflux times of 35 seconds or less. When the efflux time exceeds 35 seconds, or when there is a break in the continuity of discharge prior to essentially emptying the cone, the grout is too thick for the flow rate to be properly evaluated by ASTM C939-10. For such grouts, ASTM C939-10 recommends flowability be determined by the flow table method found in ASTM C109/C109M-11, using five drops in 3 seconds. The flow table used in ASTM C109/C109M-11 is described in ASTM C230/C230M-08.

The grouts used with auger-grout piles are generally too thick to permit proper monitoring of the flow rate with the ASTM C939-10 flow cone method. Because the current Corps of Engineers flow cone method (CRD-C611) is identical to ASTM C939-10, it is also not applicable to auger-grout piles. Therefore, if the engineer uses only current reference standards in preparing the specifications, the only option would be to use a flow table (ASTM C109/C109M-11). Without a published and accepted standard, there is an older U.S. Army Corps of Engineers test method for flow of grout mixtures by the flow cone method (CRD-C79-77). This method describes a flow cone with a volume and discharge-tube diameter identical to the ASTM C939-10 cone. However, the 0.5 in. (12.7 mm) discharge tube on the CRD-C79-77 flow cone can be removed to expose a 0.75 in. (19 mm) opening. Historically, the CRD-C79-77 flow cone, modified to use the 0.75 in. (19 mm) discharge opening, has been used as an index of grout fluidity for auger-grout piles. The typical grout efflux range of 10 to 25 seconds used for auger-grout piles is based on observations using the 0.75 in. (19 mm) opening of the CRD-C79-77 flow cone (Moskowitz 1994; Frizzi 2003; Neely 1990). The CRD-C79-77 flow cone is still available from some testing equipment suppliers and is used to monitor auger-grout pile mixtures (Moskowitz 1994; Frizzi 2003). The continued use of the 0.75 in. (19 mm) opening is desirable because both contractors and engineers are familiar with the flow rates observed with this particular cone design and can relate it to past experience. If it becomes unavailable in the future, it may be necessary to custom fabricate a cone with a 0.75 in. (19 mm) opening or resort to flow table methods, unless an acceptable replacement standard for CRD-C79-77 is developed.

6.3—Reinforcement and prestressing materials

6.3.1 *Reinforcement*—Reinforcement steel should conform to the latest revision of ASTM A615/A615M-09, ASTM A706/A706M-09, ASTM A955/A955M-11, A996/A996M-09, or A1064/A1064M-10 as appropriate.

6.3.2 *Prestressing strand*—Strand used for prestressing should conform to ASTM A416/A416M-10 Grade 250 (1725) or Grade 270 (1860).

6.3.3 *Prestressing wire*—Wire used for prestressing should conform to ASTM A421/A421M-10.

6.3.4 *Prestressing bars*—High-strength steel bars used for prestressing should conform to ASTM A722/A722M-07.

6.3.5 *Epoxy-coated reinforcement*—Epoxy-coated steel has been used as lateral reinforcement (spiral or ties) in concrete piles. The use of epoxy-coated longitudinal reinforcing bars or prestressing strand in concrete piles is limited. In these limited instances, manufacturers have reported that some adjustments are required, such as special chucks to grip the strand and special treatments at the form ends or bulkheads, when producing precast piles with epoxy-coated strand. There are no definitive reports on the performance of concrete piles with epoxy-coated strands under handling, driving, or in-service conditions. In the absence of information on installation and long-term service behavior, the committee neither endorses nor condemns the use of epoxy-coated reinforcement or strand in prestressed piles. Alternatives are available that address the control of potential corrosion (ACI 222R-01). Higher-quality concrete, with lower *w/cm* and air entrainment to reduce permeability, has been used. Adequate cover, within the limits recommended in this report, is another protective measure that can be used. Admixtures such as silica fume (ASTM C1240-11) and corrosion inhibitors are gaining use for durable concrete in marine environments. If, after consideration of these alternate methods to resist corrosion, epoxy-coated steel is used, such steel should conform to ASTM A775/A775M-07, ASTM A882/A882M-04(2010), or ASTM A884/A884M-06, as applicable.

6.4—Steel casing

6.4.1 *Load-bearing casing*—Steel casing intended for permanent load bearing, in composite action with CIP concrete, should have a thickness of not less than 0.1 in. (2.5 mm). The steel used in the casing should meet the requirements of ASTM A252-10, ASTM A283/A283M-03(2007), or ASTM A1011/A1011M-11 (refer also to 4.4.3).

The suitability of the intended materials for welding should be predetermined. The ASTM A252-10 specification does not strictly imply weldability. Other steel specifications can be used, provided that the yield, elongation, and other items are satisfactory.

6.4.2 *Non-load-bearing casing*—Steel casing not intended for permanent load bearing in composite action with CIP concrete should meet the requirements of ASTM A1008/A1008M-11 or ASTM A1011/A1011M-11.

6.5—Structural steel cores and stubs

Steel used as permanent, load-bearing structural cores or as extensions (stubs) for concrete piles should meet the requirements of ASTM A36/A36M-08, ASTM A242/A242M-04(2009), or ASTM A572/A572M-07. The thickness of steel in any part of the structural steel core should not be less than 3/8 in. (10 mm).

6.6—Splices

Materials used for splicing concrete piles should conform to the specifications listed in this chapter where possible. Structural design aspects of pile splices are discussed in 4.4.4.

Doweled splices involve inserting splice bars (dowels) into holes drilled or preformed in one or both pile segments. The space between the hole and bar is filled with a suitable material. These materials have included epoxies, quick-setting cement-based materials, and melted materials that are often sulfur-based with various additives. This material also normally fills the space between the concrete ends of the pile segments, which enables the joint to transmit compression.

Epoxy or other quick-setting compounds should have strength and durability at least equal to the concrete materials in the pile. Test methods for evaluating epoxy compounds should conform to the recommendations of ACI 503R-93.

CHAPTER 7—MANUFACTURE OF PRECAST CONCRETE PILES

7.1—General

Established plants or casting yards currently manufacture most precast concrete piles, although job-site casting yards can be used for large projects. Modern production methods and quality controls developed by the manufacturers generally lead to high-quality products and usually require less control and field inspection than job-site casting. Established prestressed or precast concrete manufacturing plants are often certified by nationally recognized agencies, thus providing recognized quality control.

Minimum requirements and basic construction procedures should be established so that the design requirements for quality, strength, and durability will be realized for all conditions, whether the piles are produced in an established precasting plant or by job-site casting. Engineers should consider specifying that, at a minimum, precast/prestressed concrete manufacturing plants have a quality-control program that is equivalent to that established by PCI MNL 116-99. Engineers should consider requiring inspection of the prestressing plant during fabrication of the piles by personnel knowledgeable in pile fabrication.

7.2—Forms

7.2.1 *General requirements*—Formwork should be in accordance with ACI 347-04 and ACI 318-08, except as modified herein.

7.2.2 *Type*—Suitable permanent forms (usually of metal, plastic, or concrete) are constructed so that the tolerances given in 7.6.3 can be maintained. Wood or wood forms with fiberglass coatings can be used for short runs of special shapes and should be constructed to produce work of a quality equal to that produced by permanent-type forms. In all cases, a concrete foundation for the casting bed is recommended. All forms for prestressed concrete piles should be constructed to permit movement of the member during transfer of the prestressing force without damage. Offsets at form joints due to misalignment or open joints should be avoided. Fins or offsets in the cast pile can cause stress concentrations and can cause shallow cracks to form in the concrete. Grinding the form surfaces may be required to correct offsets. Leaky joints should be sealed.

Pans or trough-type forms can have a slight taper or draft to the vertical sides to facilitate stripping. A maximum draft or taper of 1/4 in./ft (20 mm/m) on each vertical side will generally be acceptable, provided that the cross-sectional area of the pile is not less than the specified section with true vertical sides. Verify that specified steel cover is maintained when tapered forms are used.

Slipforming can be used for the manufacture of precast piles for both solid and hollow cross sections. Hollow piles can be formed by a traveling mandrel and top form or screed. Solid sections require a traveling top form only. In both cases, the lower half of the pile section is formed by a fixed mold of conventional design. The traveling mandrel and screed should be metal and have smooth surfaces. The method of controlling the concentricity of the mandrel, strand, spiral, and reinforcement locations should satisfy the job requirements (refer to 7.2.5, 7.5.3, and 7.6.3 for other discussion).

7.2.3 *End forms*—End forms or bulkheads should be stiff enough to prevent distortion during placement and compaction of the concrete and should be fastened securely to the pile form so that the pile head will remain in a true plane perpendicular to the pile axis. Form joints and end forms should be sufficiently tight to prevent excessive loss of cement paste during concrete placement and vibration. Holes or slots for longitudinal reinforcement should be plugged or sealed to prevent grout leakage.

7.2.4 *Chamfers and rounded corners*—All corners of square piles should be chamfered or rounded. Chamfers or radii of approximately 3/4 to 1 in. (20 to 25 mm) are commonly used. Chamfers at the pile head and tip are recommended to prevent concrete spalling during pile driving. Chamfers are generally not used on hollow cylinder piles.

7.2.5 *Hollow cores*—Hollow cores or voids in piles should be concentric with the pile centerline or axis and parallel to the edges of the cross section throughout the entire length of the hollow section. Stay-in-place forms should be of an approved, water-resistant material such as plastic, treated paper, or fiber that will resist breakage or deformation during placing of the concrete. Cores can also be formed with removable metal or inflatable rubber mandrels. Hold-downs and positioning devices should be adequate to maintain position of the core within the tolerances given in 7.6.3.

Stay-in-place core forms should be vented to prevent a potential long-term buildup of internal gas pressures caused by deterioration of the core form material. Freezing of free water inside hollow piles can cause pile breakage. Where severe freezing conditions exist, vents or holes should be provided to permit circulation or drainage of the water. Vent holes may also be required to aid in control of water hammer effects (refer to 4.4.7 and 8.3.1.5).

7.3—Placement of steel reinforcement

7.3.1 *General requirements*—All reinforcing steel and prestressing steel should be accurately positioned and satisfactorily protected against the formation of rust or other corrosion before placement in the concrete.

All prestressing steel and unstressed reinforcing steel should be free from loose rust, dirt, grease, oil, or other lubricants or substances that can impair its bond with the concrete. Slight rusting, provided it is not sufficient to cause pits visible to the unaided eye, should not be cause for rejection of unstressed reinforcement. Prestressing strand should be free of pitting and excessive rust. A light oxide is permitted (ACI 318-08 Section 7.4.3). All tie wire, metal chairs, and other supports for reinforcement should have a minimum cover as given in Chapter 4, or should be of noncorrosive material or protected by a layer of noncorrosive material.

Strands and spiral reinforcement, including square spirals, may require spacer rings and hold-up supports during concrete placement to maintain the strand pattern, to prevent necking down of the strand by the spiral turns, and to overcome the natural sag due to the weight of the strand and spiral reinforcement. Spacer hoops fabricated from two or three turns of spiral wire with an outside diameter equal to the inside diameter of the strand group can be installed inside the strand group circle to maintain the strand pattern (or reinforcing bar cage) concentric with the pile cross section within ±1/4 in. (6 mm) and to prevent necking. Strand and spiral cage hold-up supports at the spacer-ring locations are required to maintain full-length concentricity in the section of longer piles. The frequency of support required, typically 25 to 35 ft (7.5 to 10.5 m), will depend on the weight of the strand and spiral reinforcement and the pile length. Special support may be required to maintain tolerances for piles containing heavy spirals or additional reinforcement.

7.3.2 *Placement of unstressed steel reinforcement*—Unstressed reinforcement should be placed in accordance with requirements of ACI 318-08 Chapter 7. Details of reinforcing steel should conform to ACI 315-99.

7.3.3 *Placement of prestressed reinforcement*—Placement of prestressed reinforcement and the application and measurement of prestressing force should conform to industry standards such as ACI 318-08 Chapter 18 and PCI MNL 116-99.

7.3.4 *Dowel placement*—Cutoff points for dowel holes can be staggered to avoid stress risers. The same applies to added unstressed steel cast into the pile.

7.3.5 *Detensioning prestressed strands*—Prestressing strands should be detensioned in accordance with PCI MNL 116-99. The detensioning method should minimize any longitudinal movement of the pile in the prestress bed. Strand-detensioning procedures should minimize unsymmetrical stresses in the cross section and avoid shock from suddenly detensioned strands.

7.3.6 *Pile end conditions*—For prestressed piles, strands projecting after transfer should be ground or burnt flush at the pile ends to eliminate protruding steel that can cause end spalling. Under hard driving or poor pile-cushioning conditions, however, spalling of the pile head has also been observed in piles with flush strands. In such cases, it may be necessary to recess the strand approximately 1/2 in. (13 mm) at the pile head. For reinforced precast piles, hold the ends of the longitudinal reinforcement 2 in. (50 mm) below the end face of the concrete.

7.4—Embedded items

7.4.1 *Embedded items*—Sleeves, inserts, pipe, or other embedded items should be accurately set in the forms and secured to prevent movement during concrete placement and compaction.

7.4.2 *Embedded jet pipes*—Internal jet-pipe assemblies embedded in the pile should have threaded or glued joints, as in the case of plastic pipe, to prevent the migration of pressurized water into the concrete section. Steel fittings should be used where the jet pipe exits the side of the pile and where it turns 90 degrees to run down the axis of the pile; plastic pipe can be used for the vertical run.

7.5—Mixing, transporting, placing, and curing concrete

7.5.1 *Mixing*—Mixing should conform to the general requirements in ACI 318-08 Chapter 5. Detailed recommendations are given in ACI 304R-00. The *w/cm* (by mass) should be in strict conformance with the design specifications and not greater than 0.40 for concrete piles exposed to salt water or potentially corrosive groundwater. Additional information on mixture design is given in Chapter 6 of this report.

7.5.2 *Transporting*—The mixture proportions and the means of transportation should be such that the concrete will arrive at its point of final placement without segregation or loss of materials and without requiring the addition of water, over that originally specified, to achieve proper workability.

7.5.3 *Placing*—The placing of concrete should conform to ACI 304R-00 and ACI 318-08 Chapter 5, except as modified herein.

7.5.3.1 *Long-line casting*—Precast concrete piles require use of a concrete mixture having a low *w/cm*. In standard mixtures without water-reducing admixtures, slumps generally range from 0 to 3 in. (0 to 75 mm), and special care is required in handling, placing, and compacting the concrete. The use of high-range water-reducing admixtures will affect the measured slump. Slumps of 5 to 7 in. (125 to 175 mm) are not uncommon for mixtures with high-range water-reducing agents. In these mixtures, the *w/cm* is important, not the slump. Usually, concrete for precast piles is deposited directly into the forms from a bucket, pipe, chute, or conveyor.

Compaction should be by high-frequency vibrators. The concrete should be vibrated internally or externally, or both, as required to consolidate the concrete. Uniformly consolidated concrete is particularly important in a pile that can be subjected to very high impact loading during driving. Special care is necessary to consolidate the concrete in congested areas, such as at the head of the pile where additional ties or spiral reinforcing are often placed, and where reinforcing steel or sleeves are used for doweling. Detailed recommendations are given in ACI 309R-05.

When shoes, steel stubs, or mechanical splicing attachments (4.4.4, 4.5.3.5, and 8.7) are cast at the ends of precast piles, particular care should be taken to consolidate concrete around such items during casting. Vent holes through the web and flanges of the stub may be required for proper concrete placement. These vent holes permit the escape of air and water during casting that might otherwise be trapped and result in voids.

Slipforming techniques require extremely close control of the concrete consistency, vibration, and the speed of travel of the mandrel or form. The method of slipforming should be such that the pile is formed to the true cross section without sloughing, internal spalling, or plucking of the concrete surface.

7.5.3.2 *Centrifugal casting*—Hollow cylindrical piles manufactured by the centrifugal process are formed and compacted by centrifugal force in a suitable machine so that the pile molds can be revolved at speeds necessary to obtain an even distribution and dense packing of the concrete without the creation of voids behind the reinforcing steel. External vibrators and internal rollers may be used to help compact the concrete.

Metal forms should be used for centrifugal casting. The forms should be well-braced and stiffened against deformations under pressure of the wet concrete during spinning. If pretensioning is used, the form should be sufficiently rigid to resist the prestressing force without allowing deformation, which would reduce the spinning speed.

Filling of the mold and spinning should be a continuous operation, and spinning should take place before any of the concrete in the mold has taken an initial set. Excess water forced to the center needs to be drained or expelled. Concrete slump for pretensioned piles should not exceed 1-1/2 in. (40 mm), and for post-tensioned piles, it should be close to zero. The concrete pile should not be removed from the mold until the concrete has attained sufficient strength to prevent damage.

7.5.4 *Finish*—Unformed concrete surfaces should be floated and lightly troweled. Water and air bubbles can appear on sloping surfaces such as the upper boundaries on octagonal or circular piles. Spading, rodding, and thorough vibration will help to minimize the formation of bubbles but will not eliminate them. Minor water and air bubbles are normally acceptable, provided they are less than 3/8 in. (10 mm) deep. Bubble holes deeper than 3/8 in. (10 mm) require patching or filling if full concrete cover is essential.

7.5.5 *Curing*—The curing of concrete should follow the recommendations of ACI 308R-01, except as modified herein. For accelerated curing, refer to ACI 517.2R-92. Hot weather concreting should conform to ACI 305R-10. Cold weather concreting should conform to ACI 306R-10.

7.5.5.1 *Water curing*—For water curing, unformed surfaces should be covered with burlap, cotton, or other approved fabric mats and kept continuously wet using spray nozzles or perforated soaker hoses. Ponding is generally not feasible for curing concrete piles.

If forms are removed before the end of the curing period, curing should be continued as on unformed surfaces, using suitable materials. Refer to ACI 308.1-11 and ACI 308R-01 for required duration of curing.

7.5.5.2 *Membrane curing*—The use and application of liquid membrane-forming compounds for curing should follow the recommendations of ACI 308R-01. Liquid membrane-forming curing compounds should comply with the requirements of ASTM C309/C309M-11. For maximum beneficial effect, liquid membrane-forming compounds should be applied after finishing and as soon as the free water on the surface has disappeared and no water sheen is visible, but not so late that the liquid curing compound will be absorbed into the concrete. If finishing has not been completed before the loss of a visible film of water, additional water should be applied using a misting nozzle. The surface should be maintained with a visible film of water until just before the application of the compound. The curing compound should be applied just after the visible water sheen has disappeared. Membrane-forming curing compounds are not recommended as a sole means of curing.

7.5.5.3 *Accelerated curing*—Accelerated curing with low-pressure steam or other heat sources, such as hot-water or hot-oil lines under the form or electrical heating elements fastened to the form, are frequently used for curing precast concrete piles in established casting yards. The following guidelines are applicable for accelerated curing by these methods.

If steam is used to accelerate curing, it should be distributed evenly along the bed and be contained within a curing chamber that maintains a saturated curing atmosphere at all times. The chamber, usually an insulated tarp or rigid tunnel, should allow free circulation of the steam. If convectiveor conductive-heat sources are used, a curing cover is also required to retain the heat and thus allow the entire concrete section to be cured at a uniform temperature. Additionally, if convective- or conductive-heat sources are used, the open surface of the concrete should be sealed with a strip of plastic (for example, polyethylene) to prevent loss of moisture from the fresh concrete. Sufficient thermometers and temperature regulators should be provided to maintain uniform temperatures throughout the length of the bed.

A preset period of approximately 2 to 4 hours is required before the application of heat, with the required duration being dependent on the ambient temperature and the concrete mixture design. Type II cement, fly ash, and some admixtures in the mixture usually require a preset period of 3+ hours. The preset period can be determined by ASTM C403/C403M-08. During the preset period, the fresh concrete should be protected from the sun and wind, which can lead to a loss of moisture and subsequent loss of strength. In cold weather, hold the concrete temperature between 70 and 100°F (21 and 38°C) during the preset period.

At the completion of the preset period, apply heat uniformly over the full product line such that the rate of temperature rise in the enclosure does not exceed 60°F/h (33°C/h). The maximum temperature should not exceed 165°F (74°C). The curing should continue until the desired transfer strength is developed, usually 10 to 15 hours.

Prestressed piles cured at high temperatures should not be allowed to cool while the strands are fully anchored to the pretensioning bed because thermal cracks can develop before the prestress force can be transferred. Longitudinal thermal cracks are likely to develop in large piles (18 in. [450 mm] or larger) if the concrete is suddenly subjected to cold ambient temperatures. To minimize this, a cool-down steam cycle should be used. The heat source is terminated and the temperature in the enclosure is allowed to decline at a rate of 40°F/h (22°C/h) until it is within 20°F (11°C) of the outside ambient temperature. For 2- or 3-day production cycles (a weekend, for instance), thermal cracking can be avoided by reducing the maximum curing temperature to 130°F (54°C) for the 10- to 15-hour heat period, then turning off the heat and keeping the line covered until detensioning a day or two later.

7.6—Pile manufacturing

7.6.1 *Post-tensioned*—Post-tensioned piles are often manufactured in sections from 12 to 16 ft (3.7 to 4.9 m) long and can be cast either centrifugally or in vertical forms. During casting, longitudinal ducts are formed for the prestressing steel that is stressed after the sections are assembled to make up the required pile length.

Adjacent sections should be aligned within a maximum tolerance of 1/4 in. (6 mm). The maximum circumferential deviation in the alignment of the holes for prestressing steel should not exceed 1/4 in. (6 mm) at the joint.

Abutting joint surfaces should be covered by a sealing material of sufficient thickness to fill all voids between end surfaces except at the prestressing steel duct. After the sealing material is applied, pile sections should be brought into contact and held together by compression while the sealing material sets.

The ducts should be pressure-grouted after prestressing. The grout pressure should be held for approximately 2 minutes, forcing the free water in the grout into the pores of the walls of the post-tensioning ducts and packing the grout. The prestress in the tendons should be maintained by the stressing chucks until the grout has attained sufficient strength to adequately bond the steel and transfer the prestress without slip. Piles should not be handled or moved in any way detrimental to the pile during this period. Prestressing steel ducts should be grouted in accordance with the provisions of ACI 318-08 Chapter 18, and Chapter 6 of this report.

7.6.2 *Prestressing*—Minimum concrete strengths should be 3500 psi (24 MPa) for pretensioned piles at the time of stress transfer, and 4000 psi (28 MPa) for post-tensioned piles at the time of prestressing, unless higher strengths are required by the design.

7.6.3 *Tolerances*—Except as modified in this chapter or otherwise specified, precast concrete piles should be manufactured to dimensional tolerances conforming with the requirements of ACI 117-10.

The permitted departure of the pile head from a plane at right angles to the longitudinal axis of the pile according to ACI 117-10—1/4 in./ft (20 mm/m) of head dimension—may be too large for conditions where the piles will be subjected to hard driving. In such cases, the engineer may want to specify square driving heads with closer tolerances. Square pile ends may also be required when using mechanical splices (4.4.4). The departure from a straight line parallel to the centerline of the pile permitted according to ACI 117-10—1/8 in. per 10 ft (1 mm/m) of length—should be interpreted as the as-built straightness, including the cumulative effects of forming, curing, and long-term storage.

7.7—Handling and storage

Piles should not be handled or stored in any way that will result in damage to the pile. Piles should be lifted and blocked for storage at pre-designated points, such that bending stresses will be within acceptable limits.

Concrete strength at the time the pile is lifted from the bed should not be less than 3500 psi (24 MPa). Impact stresses due to handling or storage should not exceed the values given in Chapter 4. For calculating handling stresses, a 50 percent impact factor is recommended (4.2.1.1). Piles should be stored in a manner that will not result in net tensile stress under the dead weight of the pile.

Where the sides and bottom of the pile are accessible, lifting is usually accomplished by tongs or slings around the pile. Inserts or lifting loops can be used where this is not possible. Inserts should have the specified minimum cover. For piles to be used in marine or other corrosive environments, where the loop will be above the mud line, the loop should be cut off below the surface of the pile so that proper allowance for cover is provided. Recesses formed by loop cutoff should be plugged with epoxy mortar. Epoxy compounds should conform to requirements given in 6.6.

Handling holes are not recommended where driving conditions result in net tension in the section.

CHAPTER 8—INSTALLATION OF CONCRETE PILES

8.1—Purpose and scope

Many methods have been successfully used to install concrete piles, and new techniques and methods are constantly being developed. These methods differ according to the type of concrete pile being installed, the purpose to be served by the pile, the forces to be resisted, the soils into which the piles are installed, the structure to be supported, and the pile orientation (vertical or battered). The methods of installation to be used will also differ according to the practical aspects of the particular site and its location and the economic factors involved.

A detailed description of all installation techniques and equipment operations used to install concrete piles is beyond the scope of this report. For more detailed information on pile-installation techniques and equipment, refer to general references on pile installation (ASCE/SEI 7-05; Davisson 1972b; Fuller 1983; Gerwick 1993; Gendron 1970; equipment manufacturers' manuals).

The primary purpose of this chapter is to provide general principles by which driven piling can be properly installed. In discussing the most common methods, the intent of this chapter is not to limit or restrict new techniques and methods, provided they can be shown to fulfill the recommendations of this report. Only limited recommendations for drilled piles are included herein. Additional information on the installation of drilled piles can be found in publications by Neely (1990), The Deep Foundation Institute (DFI 1995; Moskowitz 1994; Frizzi 2003), The Federal Highway Administration (Brown et al. 2007), and in ACI 336.1-01 and ACI 336.3R-93.

The installation method should not permanently impair the ability of the soil to support the pile. Some techniques actually strengthen certain soils. On the other hand, the desire to maintain or improve soil properties should not dictate a method, such as overdriving, which endangers the structural integrity of concrete piles. Concrete piles should be installed so that the desired pile interaction with the soil will be developed without impairing the structural integrity of the pile.

The installation method should be integrated with the design. The designer should confirm that the piles can be installed under particular site conditions in a manner compatible with their intended function. The construction documents should limit or exclude the use of those installation methods that would be harmful. The contractor should install the piles in a way that will comply with the essential design requirements. Within these necessary limitations, the designer should allow freedom in the selection of installation methods, specifying results instead of methods where practical, so that economy is obtained and an appropriate division of responsibility is maintained.

The interrelationship of design, manufacture, and installation is vital to suitable foundation performance. Construction procedures often have profound influences on pile behavior, and even subtle departures from construction procedures established by the project documents can lead to unsatisfactory pile foundation performance or failure. Design personnel involved in field engineering and inspection during installation should be experienced with pile foundation construction, as well as familiar with the project design requirements. The designer may want to consider specifying minimum experience requirements for the piling contractor, its lead personnel, or both.

8.2—Installation equipment, techniques, and methods

8.2.1 *Pile-driving hammers*—The most common method of installing concrete piles is by means of hammer blows. Pile-driving hammers are of several different types and have rated energies from 356 ft-lb (483 J) to in excess of 1,000,000 ft-lb (1,360,000 J) per blow. The size of the hammer (rated energy) should be compatible with the pile size, length, weight, and capacity requirements. The proper selection and design of the hammer-cushion-pile system for a given set of conditions can be aided by a wave-equation analysis of the system (refer to 3.3.2.2 and 8.3, where pile-installation stresses are discussed). For example, if the capblock and pile cushioning material is held constant, a heavy ram with a relatively low-impact velocity is more desirable than a light ram with a high-impact velocity for controlling the peak stresses. This is especially true when driving long piles. Any combination of ram weight, stroke, and proper cushioning materials can be used, provided that the combination causes adequate peak force duration and magnitude to develop the required pile capacity and penetration and does not cause damaging tensile or compressive stresses.

8.2.1.1 *Drop hammers*—Drop hammers are weights that are raised and allowed to fall freely on the head of the pile. The velocity of the weight at impact is proportional to the square root of the fall height, and the pile stresses generated by the hammer impact increase with the impact velocity. The manner in which the operator releases or restrains the drop hammer during its fall has an important effect on the actual velocity at impact and thus on the effective energy delivered by the blow. A drop hammer should be controlled during the fall by guides so that the pile is struck squarely and concentrically.

For efficiency and to prevent damage to the pile, the weight of the drop hammer should be substantial in relation to the weight of the pile, on the order of one or two times the pile weight, and the fall should be kept low, on the order of 3 ft (1 m). Some authorities recommend even lower falls, particularly when driving onto rock (8.3.1.2). Higher falls are sometimes used, but these frequently result in damage to the pile. Where a given drop hammer proves inadequate, it is usually better to increase the weight of the hammer rather than the height of fall.

A special type of drop hammer is used to install compacted concrete piles (1.2.3). This is a long, cylindrical steel weight that falls freely inside a heavy steel drive casing or pipe, impacting on a plug of zero-slump concrete. Fall heights for this type of drop hammer can range up to 20 or 30 ft (6 or 9 m) during the formation of the compacted base. The predesignated minimum fall is monitored by a mark on the hammer line. As with other drop hammers, however, the end result is sensitive to operator control.

8.2.1.2 *Externally powered hammers*—Hammers powered by steam, air, or hydraulic fluid use external power sources such as boilers, compressors, or hydraulic power units to operate the hammer. For steam- or air-operated hammers, the pressure is released to the atmosphere. The pressure release in hydraulically powered hammers, however, involves recirculation of the hydraulic fluid through a closed system. Some of the most recent advances in hammer technology have been in hydraulic hammers. In addition to the advantage that they can often be operated off the hydraulic power system on the pile-driving rig, many of these hydraulic hammers also allow the hammer stroke to be carefully controlled and varied, and contain internal ram velocity monitoring devices. Externally powered hammers can be classified as single-, double-, or differential-acting, depending on how the motive fluid acts during the cycle of operation, as described in the following paragraphs.

Single-acting hammers use steam, air, or hydraulic pressure only to raise the ram. The ram is accelerated upward under the force resulting from the operating pressure acting on the bottom of the lifting piston. After rising a certain distance, generally called the stroke-to-cutoff, a trip valve is engaged that shuts off the pressure source and releases (exhausts) the pressure beneath the lifting piston. When the ram engages the trip valve, it has an upward velocity and continues to travel upward until the downward acceleration of gravity reduces the upward ram velocity to zero. The total height of the ram rise at zero upward velocity is the hammer stroke. The ram then starts its fall under the acceleration of gravity to impact the pile. Sufficient fluid pressure and volume should be supplied at the hammer piston to result in an upward ram velocity at the stroke-to-cutoff that will raise the ram to the desired hammer stroke.

External valve slide bars, which engage the trip valve, can sometimes be modified or adjusted to intentionally vary the stroke-to-cutoff distance and thus the hammer stroke (height of fall). Some hammers are equipped with mechanisms that make it possible to remotely shift the stroke-to-cutoff distance in seconds. Thus, the delivered energy can be adjusted to meet special driving conditions. When operating a single-acting hammer in a short-stroked mode, the modified stroke-to-cutoff distance fixes only the lower limit on the hammer stroke. The actual stroke developed will depend on the source pressure, and an oversupply of air or steam can lead to overstroking.

Double-acting hammers use steam or air pressure to power both the upstroke and the downstroke of the ram during the hammer cycle. When the trip valve is engaged on the upstroke to release (exhaust) the pressure beneath the lifting piston, an exhaust valve above the piston, which was open during uplift, is closed and the source pressure is diverted to the top of the piston. During the downstroke, the ram is accelerated downward by the force of the pressure acting on top of the piston, in addition to the force of gravity. Therefore, the ram velocity at impact, and hammer energy, is a function of the pressure on the top of the piston during the downstroke as well as the hammer stroke. The double-acting hammer exhausts the steam or air at both upstroke and downstroke. Double-acting hammers tend to have light rams and high speeds.

Differential-acting hammers use steam, air, or hydraulic pressure to power both the upstroke and downstroke. This type of hammer differs from a double-acting hammer in that during the downstroke, the cylinder is under equal pressure both above and below the piston, and the hammer exhausts only during the upstroke. When the trip valve is engaged on the upstroke, the exhaust valve above the piston, which is open during uplift, is closed and pressure is supplied to the top of the piston. The pressure below the piston, however, is not released as with the double-acting hammer. The area of the piston top is larger than the piston bottom area (difference equals the area of the piston rod), resulting in a net downward force from the source pressure during the downstroke. During the downstroke, the ram is accelerated by the differential downward force on the piston in addition to the force of gravity. Therefore, the ram velocity at impact is a function of the pressure on the piston during the downstroke as well as the hammer stroke. Control of the energy and ram velocity can thus be affected by the throttle.

The maximum energy that a differential hammer can deliver is equal to the total weight of the hammer, excluding the drive head, multiplied by the stroke of the hammer ram. Correct operating pressure is indicated by a slight raising of the hammer base at the start of each downward stroke. Differential hammers generally have shorter strokes than comparable single-acting hammers, resulting in faster hammer speeds, that is, more blows per minute. More rapid action of these hammers, approximately twice that of single-acting hammers, can result in a lower total driving time.

8.2.1.3 *Diesel hammers*—Diesel hammers are powered by internal combustion in which the explosion takes place under the ram near the end of its fall. Therefore, the impact or push is a combination of the ram fall and the explosive reaction. This explosive force also serves to propel the ram back up to the top of the stroke and restart the cycle. Diesel hammers develop maximum energy in hard driving. The thrust from combustion in diesel hammers is maintained over a relatively longer period than the actual impact and thus enhances pile penetration. Although diesel hammers have relatively lighter rams and longer strokes than single-acting or differential hammers, the ram velocity at impact is less than the velocity resulting from the height of fall because of the cushioning effect of air compression in the combustion chamber. Most diesel hammers have a fuel throttle adjustment for controlling the ram stroke, and thus the pile stresses during easy driving. The proper system (hammer, cushion, and pile) for particular driving conditions can be selected using a wave-equation analysis program that properly models both the combustion cycle and the impact forces of diesel hammers. The variable energy of the diesel hammer needs to be considered when establishing the production pile installation criteria, if all production piles are to be driven to the same final driving resistance.

8.2.1.4 *Vibratory hammers*—Vibratory driving, or rapid vibration of a pile, will aid penetration in certain soils, especially in granular materials such as sands and gravels. Bias weight (the addition of extra weight to the pile-hammer system) or down-crowd (the application of downward force by the pile-driving rig) may be required in addition to the weight of the pile and the vibratory driver to achieve penetration during vibration.

Vibratory hammers are either of the low- or high-frequency type. Low-frequency vibrators operate at less than 50 Hz (typically 10 to 20 Hz) and high-frequency vibrators operate up to approximately 150 Hz. High-frequency vibrators are capable of operating at the resonant longitudinal frequency of the pile, which can aid penetration in some cases.

The effectiveness of vibratory methods of installation is generally proportional to the energy transmitted. Some vibratory hammers are assembled in units so that a unit can be added to increase effectiveness. Hydraulically powered vibratory hammers can work fully submerged for driving and extracting piles below the water surface.

The connection of a vibratory hammer to the pile, usually with a clamp, is particularly critical and should be adequate and secure to prevent dissipation of energy. Vibratory hammers can be used effectively on sheet piles, H-piles, pipe piles, and on mandrels for CIP concrete piles.

8.2.2 *Weight and thrust*—Concrete piles can be installed by superimposing dead weights. This method is practical in very soft soils where large piles are set and then sunk by placing a weight on top. This technique is usually augmented by excavation from within or beneath the tip and by jetting.

Piles can be jacked down by hydraulic rams reacting against weights or anchors or against previously installed piles. One machine uses long-stroke hydraulic rams reacting against the heavily loaded carriage of the machine. Another machine attaches itself hydraulically to several adjoining piles and then pushes on one pile while holding onto several others, this being done progressively to move the entire group of piles down. This type of machine is used primarily to install steel sheet piles.

8.2.3 *Drive heads*—Piles being driven by impact require an adequate drive head, also referred to as helmets or drive caps, to distribute the blow of the hammer to the head of the pile. The drive head also frequently holds or retains protective material (8.2.4) to reduce the shock of the blow and spread it more evenly over the head of the pile. The driving head should be axially aligned with the hammer and the pile.

The driving head for steel pipe should fit snugly to prevent bulging and distortion to the head of the pile. Machined steel heads are beneficial when driving directly on thin-walled steel pipe. The use of drive-fit outside sleeves mounted over the top of the pipe can effectively reduce pipe distortion resulting from driving.

The driving head for precast concrete piles should not fit tightly, as this could cause the transfer of moment or torsion; however, the helmet should not be so loose as to prevent proper axial alignment of hammer and pile.

8.2.4 *Capblocks and cushions*—Capblocks, also called hammer cushions, are used between the drive head and the hammer ram to protect both the pile and hammer from damage that can be caused by direct impact. The capblock, however, should effectively transmit the hammer energy to the pile without excessive loss of energy. The important properties of capblock materials are their elastic and energy-transmission properties (modulus of elasticity, coefficient of restitution, and dimensions), and the stability of those properties under the high stresses and heat buildup that occur with repeated hammer blows.

Many different materials are used for capblocks. A common type of capblock is a hardwood block with grain parallel to the pile axis seated in a tight-fitting steel enclosure. Hardwood blocks have the advantage of a low modulus of elasticity and coefficient of restitution that softens or modulates the hammer blow, reducing the pile stresses and lengthening the force duration. Hardwood blocks have the disadvantages of becoming crushed and burned out, requiring frequent replacement, and having variable elastic properties during driving. Where a soft capblock is needed to control pile stresses during driving, and its disadvantages are not critical, a wood capblock can be effective.

Capblocks of alternating aluminum and micarta (the trademark for a material generically described as a phenolic resin-canvas laminate) layers are also common. These transmit energy better than hardwood, maintain nearly constant elastic properties, and have a relatively long life. Capblocks of numerous other materials, such as various resins and plastics, rubber, plywood, wire rope coils, compressed wire, and compressed paper, are available to suit a variety of pile types and driving conditions. Capblocks are often composed of layers of these various materials alternating with aluminum disks that increase the radial strength of the composite block and help dissipate the heat generated in the cushion.

Pile cushions are used between a concrete pile and the driving head and are generally required for all types of precast piles to distribute the hammer blow, protect the pile head, and control driving stresses in the pile. They are usually laminated, consisting of softwood or hardwood boards or plywood, although other materials have been used. The required thickness of cushioning material varies with the job conditions. The effect of cushion properties and cushion thickness on pile stresses and energy transmission can be evaluated by a wave-equation analysis for the driving conditions involved (3.3.2.2).

8.2.5 *Mandrels*—Thin steel pile shells are frequently driven by steel mandrels that transmit the hammer blow uniformly to the soil and prevent the shell from collapsing as it is driven through the soil. Many types of mandrels are used. One type engages closely spaced drive rings or steps in the shell. Others expand pneumatically, mechanically, or hydraulically to grip the shell at numerous points along the pile. Mandrels are generally designed for repeated use, which results in heavy wall thicknesses. The resulting high axial stiffness of the mandrel permits the shells to be driven to higher capacities than would be permitted by the axial stiffness of the shells alone.

Properly designed mandrels have proven very effective in obtaining penetration of the pile tip through hard soil layers and obstructions. When hard materials and obstructions result in shell collapse or tears that admit water and fine sands and prevent proper concreting of the shell or extraction of the mandrel, however, the use of the mandrel may be uneconomical. Mandrels should prevent distortion of the shell and resist bending and doglegging within limits set by the design engineer.

Certain types of CIS concrete piles are constructed by driving mandrels without shells, and then placing concrete though the mandrel core as the mandrel is withdrawn. Shoes or tips of such mandrels may be expendable and remain in place as the tip of the pile. When the shoe is designed for removal, it should be designed so as not to unduly disturb or disrupt the concrete during withdrawal.

Mandrels have been used to drive pipe piles by engaging only the bottom of the pipe, thus pulling the pipe downward. A special tip detail is required at the bottom drive point to take the concentrated mandrel force and distribute it to the pipe wall. The customary tip-closure plate for top-driven pipe piles is generally inadequate for this purpose. Pipe piles have also been driven with mandrels that simultaneously engage the closure plate or a plug at the pile tip and the top of the pipe. In this case, the pipe and mandrel lengths should be carefully matched so that the driving force is not transmitted primarily through the pipe top.

8.2.6 *Jetting*—A distinction is made between prejetting and jetting. Prejetting takes place before the pile is inserted into the ground, whereas jetting takes place during the insertion of the pile into the ground. Jet spudding, or prejetting, is the technique of installing a weighted water jet at the pile location to break up hard layers and cemented strata. The jet is then withdrawn and the pile installed in the same location. This prejetting can also temporarily suspend or liquefy the soils, which reduces the resistance to pile penetration. In soils containing boulders, cobbles, or large gravel, prejetting or jetting can segregate these coarse materials to the bottom of the jetted hole, making it difficult to drive the pile through them.

The use of external or internal jets during pile installation can also assist pile penetration. Jetting, with either external or internal jets, reduces skin friction in sands and sandy materials. The water flows up along the pile, reducing the friction on the pile sides. When sinking a pile with a single external jet, the pile tends to move toward the jet. Therefore, jets are often grouped in pairs or as a ring to provide uniform distribution of water around the pile. Internal jets in some instances have multiple nozzles to distribute the water around the pile. The effect of jetting on pile alignment is particularly a problem with batter piles and requires special attention.

The influence of jetting on the long-term soil properties and the consequent interaction of soil and pile after installation should be considered. Jetting is usually stopped before the final tip elevation is reached so that the pile can be driven the last few feet into undisturbed material. Most granular soils will be reconsolidated after jetting stops and the driving of the pile with a hammer augments this consolidation. A certain number of blows of the hammer should be specified, as well as a minimum distance for the pile to be driven after jetting stops, to achieve the desired consolidation and the avoidance of any deleterious effect on previously driven piles. Jetting should not be done below the tips of previously driven piles. The effect of jetting on adjacent piles and structures should be considered.

In general, simultaneous jetting and driving of precast or prestressed concrete piles is undesirable. This is particularly true when the jetting is taking place below the pile tip, which is likely to result in low tip resistance and high tension-stress reflections. Special precautions, such as restrictions on the depth to which the jets can be operated while driving and hammer-energy restrictions, should be taken if concrete piles are to be driven while jetting is taking place so that the driving stresses are not excessive. When driving of precast or prestressed concrete piles commences after the completion of jetting, the pile should be seated using a low hammer energy to develop a reasonable tip resistance before the full driving energy is used.

The use of high-pressure internal jets in hollow-core concrete piles can burst the pile if the jet pipe breaks during installation, either from the high jet pressure or from high pressures generated by water-hammer effects (8.3.1.5) during subsequent driving. External jetting is preferred for hollow-core concrete piles. If internal jetting is necessary, it may be desirable to switch to a pile of solid cross section.

8.2.7 *Predrilling*—Predrilling is an effective technique to facilitate pile installation in many soils, such as those containing hardpan, cemented strata, hard clay, or dense compacted sand. Dry predrilling can be done with either a continuous-flight auger or a drill shaft with a short-flight auger. When drilling through clay, the clay soils may provide sufficient strength to maintain the hole stability. In plastic soils that stick to the auger flights, drilling can often be facilitated by adding water or air through the drill stem to break up the soil and carry it to the surface.

Wet-rotary drilling has been used to excavate deep holes where the power required for augering would be excessive. It is particularly suited to plastic soils that would stick to the auger flights and to soils that would collapse unless the hole remains filled with fluid. In wet-rotary drilling, a pipe drill stem with various types of spade or fish-tail bits replaces the auger. Water or drilling-mud, usually a bentonite slurry, is circulated through the drill stem to carry the cuttings to the surface and to keep the hole open. The large quantity of slurry produced can be a serious problem, and its disposal should be planned for in advance.

Predrilling is generally a more controllable form of pre-excavation than jetting, with less potential for detrimental effects on adjacent piles or structures and the frictional capacity of the predrilled pile. Depth, diameter, fluid pressure, and drill time are among the variables that should be controlled to limit the effects of predrilling on pile capacity. The possible effect of predrilling on adjacent piles and structures should be considered.

8.2.8 *Drilling open-ended pipe piles*—In attempting to install piles through certain types of soils, such as those containing boulders, a combination of driving and drilling is often the most practical method. Alternating driving with drilling inside the pipe is used to advance the pile. Deformation of the pile tip should be prevented. The tip can be reinforced or a special steel shoe can be used. Driving should preferably be performed with a high-blow-rate hammer or vibrator.

When installing open-ended pipe to rock, a socket can be drilled in the rock after the pipe is seated. Reseating the pipe after drilling the socket is almost always necessary.

Excavating from within pipe piles, especially those of a larger diameter, can be performed with air-lift pumps. Alternatively, the material can be blown out with high-pressure air or a combination of steam and water suddenly injected below the soil plug. A deflector temporarily attached to the head of the pile will often be useful for controlling the geyser of water and soil ejected during the blowout operation. Drilling within the driven pipe pile is an effective way of removing the soil.

Pumping from within the interior of hollow piles when installing them in sands or silts can cause soil material to flow under the tip, thus creating a quick condition and aiding sinking. Alternatively, the water level can be brought to a much higher level inside the pile than outside, and sudden release would wash out material under the tip. This last process is difficult to control and seriously disturbs the adjacent soil.

8.2.9 *Spudding and driving through obstructions*—Spudding is the use of a shaft or mandrel to force a hole through overlying fill, trash, riprap, or boulders to make it possible to install a pile. A precast concrete pile often makes an excellent spud in itself and need not be withdrawn. Prestressed concrete piles have been successfully driven through riprap, miscellaneous fill, and coral layers where even steel piles deform, but during such driving, the pile should not be restrained or excessive bending will result. The nature and extent of the obstructions will dictate the best method to install the piles. Where shallow obstructions are pervasive and onerous, such as a boulder-laden stratum, it is often most advantageous to pre-excavate the obstructions. Pile tip protection may be necessary in some cases (8.7.2 and 8.7.3). The nature and extent of the obstructions will dictate the best method.

When driving closed-end pipe piles, it is often possible to first drive the pile to the obstructed level and place concrete for some or all of the pile length, and then redrive the pile after the concrete has achieved a suitable strength. This process significantly improves hammer energy transmission and minimizes the potential for pile damage. Redriving should only be done after it has been determined, by wave-equation analysis, for example, that the concrete stresses during driving are tolerable. Redriving can be done by directly driving on the top of the concrete or by the use of a mandrel extending to the top of the concrete. A pile cushion on top of the concrete will generally be required to distribute the impact evenly to the concrete. In either case, consideration should be given to the effect of this hard driving on any contiguous structures, streets, or utilities.

8.2.10 *Followers*—Frequently a pile will need to be driven in a hole or through overburden to a cutoff elevation below the level on which the driving rig is operating and beyond the level that the hammer can reach. When the use of hammer lead extensions is not feasible, a common technique to complete the driving below the hammer reach is to use a pile follower between the drive head and the pile head. A follower is a structural member, generally made of steel, which is designed to be sufficiently rigid to transmit the hammer energy to the pile. Because followers are generally subjected to repeated use similar to mandrels, the allowable driving stresses in followers are usually selected conservatively. Followers should have guides or other means adapted to the leads so that the hammer, follower, helmet, and pile are maintained in good alignment.

Consideration should be given to the effect of the follower on the driving criteria of piling installed with a follower. Specifications frequently prohibit the use of followers because they can influence the driving characteristics of the system. Proper use of a follower, however, is a matter of design. The follower should be designed and constructed so that it will be able to withstand dynamic driving stresses and allow adequate transmission of hammer energy to the pile. The wave-equation analysis (3.3.2.2) can be used to assess the effect of the follower on the pile-driving characteristics and also the pile and follower stresses.

8.3—Prevention of damage to piling during installation

8.3.1 *Damage to precast or prestressed piling during driving*—Cracking or spalling during driving of reinforced or prestressed concrete piles can be classified into six types:

1. Spalling of concrete at the pile head due to high compressive stress.
2. Spalling of concrete at the pile tip due to hard driving resistance at the tip.
3. Transverse cracking or breaking of the pile due to tensile stress reflections from the tip or head of the pile.
4. Spiral or transverse cracking due to a combination of torsion and reflected tensile stress. This type of cracking is sometimes accompanied by spalling at the crack.
5. Spalling and cracking due to a combination of compression or tension reflections and bending stress resulting from pile curvature.
6. Longitudinal splits of hollow piles due to internal radial pressures.

8.3.1.1 *Pile-head spalling*—Spalling of concrete at the pile head is caused by high or irregular compressive stress concentrations. This type of damage can be caused by the following:

1. Insufficient pile cushioning material between the drive head and the concrete pile, resulting in a very high compressive stress on impact of the hammer ram.
2. The top of the pile is not square or perpendicular to the longitudinal axis of the pile, resulting in an eccentric hammer blow and high stress concentrations.

3. Improper alignment of the hammer and pile, resulting in an eccentric hammer blow that causes high stress concentrations.

4. Impact on longitudinal reinforcing steel protruding above the pile head, resulting in high stress concentrations in the concrete adjacent to the reinforcement.

5. Lack of adequate transverse reinforcement (spiral confinement) at the pile head.

6. The top edges and corners of the concrete pile are not chamfered, causing the edges or corners to spall.

7. Fatigue failure of the concrete under a large number of hammer blows at a high stress level.

8.3.1.2 *Pile-tip spalling*—Spalling of concrete at the point of the pile can be caused by high driving resistance. This type of resistance can be encountered when founding the pile point on bedrock or other highly resistant strata. Also, piles seldom interface evenly with the rock, resulting in eccentric loading and higher-than-average stresses. Compressive stress at the pile tip when driving on bare rock can theoretically be twice the magnitude of the compressive stress produced at the pile head by the hammer impact. Under such conditions, overdriving of the pile and particularly high ram velocities should be avoided. In the more normal cases where there is soil overlying the rock, tip stresses will generally be of the same order of magnitude as the head stresses. Prolonged driving at high blow counts and high tip-stress levels can also lead to concrete fatigue failure at the tip. Like the pile head, the pile tip should be provided with adequate transverse reinforcement (spiral confinement).

8.3.1.3 *Transverse cracks*—Transverse-tension cracking of a pile due to reflected tensile stress is a complex phenomenon. It can occur in the upper end, midlength, or lower end of the pile. It usually occurs in piles 50 ft (15 m) or more in length. It can occur when the tip resistance is low during driving, such as driving in very soft soils or when jetting or predrilling has reduced the soil resistance at the pile tip. It also can occur, although rarely, with light hammers when resistance is extremely hard at the point, such as driving on solid rock.

A compressive stress is produced when a hammer ram strikes the pile head or cushion. This compressive stress in concrete piles travels as a wave down the pile at a velocity of approximately 12,000 to 15,000 ft/s (3700 to 4600 m/s). The peak magnitude of the stress wave depends on the ram properties (weight, shape, material), impact velocity, cushioning, pile material (modulus of elasticity, wave velocity), and soil resistance. Because the stress wave travels at a constant velocity in a given pile, the length of the stress wave depends on the duration that the hammer ram is in contact with the cushion or pile head. A heavy ram will stay in contact with the cushion or pile head for a longer time than a light ram, thus producing a longer stress wave. If a ram strikes a thick or soft cushion, it will also stay in contact for a longer period of time, resulting in a longer stress wave.

The compressive-stress wave traveling down the pile can be reflected from the point of the pile as either a tensile- or compressive-stress wave, depending on the soil resistance at the point, or can pass into the soil. If little or no soil resistance is present at the pile point, the compressive-stress wave will be reflected back up the pile as a tensile-stress wave. At any given time, the net stress at a point in the pile is the algebraic sum of the compressive-stress wave traveling down the pile and the reflected wave traveling up the pile. Whether or not a critical tensile stress sufficient to crack the pile will result depends on the magnitude of the initial compressive stress, the length of the stress wave relative to the pile length, and the nature (tension or compression) of the reflected wave. A long stress wave is desirable to minimize the possibility of damaging the pile.

If significant resistance exists at the pile point, the initial compressive stress wave traveling down the pile will be reflected back up the pile as a compressive stress wave. Tensile stresses will not occur under these conditions until the reflected compressive stress wave traveling up the pile is reflected from the free pile head as a downward-traveling, tensile-stress wave. It is possible for critical tensile stresses to occur near the pile head in this case, such as when driving onto rock with a very light hammer ram weight.

In summary, tensile cracking of precast piles can be caused by the following:

1. Insufficient cushioning material used between the drive head and the concrete pile, resulting in a stress wave of high magnitude and short length. The use of an adequate softwood cushion is frequently the most effective way of reducing driving stresses. Stress reductions on the order of 50 percent can be obtained with new, uncrushed cushions. As the cushion is compressed by hard driving, the intensity of the stress wave increases. Therefore, the use of a new cushion for each pile is recommended.

2. High ram velocity, which produces a stress wave of high magnitude.

3. Critical tensile-stress reflections resulting from little or no tip resistance. This condition is most critical in long piles, 50 ft (15 m) or more in length. This is possible when driving in soft soils, through a hard layer into an underlying softer layer, or when the soil at the tip has been weakened by jetting or drilling. Most commonly, these critical tensile stresses occur near the upper-third point, but they can occur at midlength or lower.

4. Critical tensile stresses resulting from the short wave produced when driving against very high tip resistance with a relatively light hammer ram weight.

8.3.1.4 *Diagonal cracks*—Diagonal tensile stress resulting from a twisting moment applied to the pile can cause pile failure, generally appearing as spiral or transverse cracking. If reflected tensile stresses occur during driving and combine with diagonal tensile stress due to torque, the situation can become even more critical. Torsion on the pile can be caused by the drive head fitting too tightly on the pile, preventing it from rotating slightly due to soil action on the embedded portion of the pile, and excessive restraint of the pile in the leads and rotation of the leads.

8.3.1.5 *Internal-bursting cracks*—Internal radial pressures in both open- and close-ended hollow precast piles lead to tension in the pile walls and can cause bursting. Longitudinal splits due to internal bursting pressures can occur with open-ended hollow precast piles. When driving in extremely soft, semifluid soils, the fluid pressure builds up and a hydraulic-ram effect occurs. This can be prevented by providing vents in the walls of the cylinder pile or by cleaning or pumping periodically. This can also occur when the pile head is driven below water, in which case substantial venting should be provided in the driving head.

Soil plugs can form inside the pile and exert a splitting action when driving open-ended precast piles. The plug can be broken up during driving by careful use of a low-pressure jet inside, but the most practicable remedy appears to be the provision of adequate transverse reinforcement in the form of spirals or ties in the plug-forming zone. Solid tips will eliminate some of the problems with fluid or soil pressures but may not be compatible with other installation requirements, such as requiring piles to be open-ended to facilitate access below the tip.

Internal jets can sometimes cause bursting, particularly in hollow-core piles with a closed tip and head. If the jet breaks during driving, water pressure in the core chamber can result in tangential stresses in the pile wall that exceed the concrete tensile strength. Vents will prevent this if they are located so as not to plug during driving. Furthermore, venting at the top of a hollow precast pile will prevent a potential long-term buildup of internal gas pressure.

Freezing of free water inside the pile cavity can also cause pile breakage. Drain holes through the pile wall should be provided at the groundwater line and the pile filled with free-draining material. For piles standing in open water, a concrete plug should be placed from the lowest freeze depth to above the high water level. Drain holes should be located just above the surface of the plug. Alternatively, the entire pile can be filled with concrete.

The lateral pressures during placement of concrete inside of hollow cylinder piles can also lead to longitudinal splitting forces for deep plugs. Therefore, when casting plugs inside such piles, the circumferential stress in the pile walls resulting from the lateral pressures of the fresh concrete should be considered. In some instances, precast plugs that are grouted in place have been used to overcome this problem.

Some prestressed concrete piles are fabricated with flexible metal conduit in the pile head for grouting dowels after driving. Free water inside these flex-tubes should be prevented in areas where freezing can occur.

8.3.1.6 *Allowable cracks*—Precast or prestressed concrete piles with minor cracks may be acceptable in some cases. In the event of more serious damage, it may be possible to implement suitable pile repairs. The nature and extent of these cracks (number, location, and alignment), the pile environment (saltwater and corrosive soils), and the modes of loading to be resisted by the pile should be evaluated together to determine whether a replacement pile is necessary.

8.3.2 *Good driving practice for prestressed or precast concrete piles*—Some rules of thumb for good driving practice for precast concrete piles can be summarized as follows:

a) Use adequate cushioning material between the hammer drive head and the concrete pile. Three or 4 in. (75 or 100 mm) of softwood cushioning material may be adequate for piles 50 ft (15 m) or shorter with reasonably high tip resistance. Softwood cushion thicknesses of 6 to 8 in. (150 to 200 mm), or even thicker, are likely to be required when driving long piles against low tip resistance. A new cushion should be provided for each pile. The wood cushioning should be replaced when it becomes highly compressed, charred, or burned during driving of a pile. If it is necessary to change wood cushioning toward the end of driving, then driving should continue until the new cushioning has been adequately compressed before observing the final set. The use of an adequate cushion is usually a very economical means of controlling driving stresses;

b) Reduce driving stresses, when possible, by using a heavy ram with low impact velocity (short stroke) to obtain the desired driving energy rather than a light ram with a high impact velocity (long stroke). Driving stresses can also be reduced by using proper hammer cushioning (cap-block) materials;

c) Reduce the ram velocity (stroke) during early driving and when light soil resistance is encountered to avoid critical tensile stresses. This is very effective when driving long piles through very soft soil;

d) If predrilling or jetting is permitted in placing the piles, the pile point should be well seated with reasonable soil resistance at the point before full driving energy is used;

e) Avoid jetting near or below the tip of the pile where this can wash out a hole ahead of the pile or produce low resistance at the tip. In many sands, it is preferable to drive with larger hammers or to greater driving resistance rather than to jet and drive simultaneously;

f) The drive head should fit loosely around the pile top so that the pile can rotate within the drive head. The drive head should not, however, be so loose as to permit improper alignment of hammer and pile;

g) The pile should be straight and not cambered because of uneven prestress, poor manufacturing or storage methods, or both. High flexural stresses can result during driving of a crooked pile;

h) The top of the pile should be perpendicular to the longitudinal axis of the pile and strands or reinforcement should not protrude from the head;

i) Use adequate spiral reinforcement throughout the pile, particularly near the head and tip;

j) Use a level of prestress adequate to prevent cracking during transport and handling and to resist reflected tensile stresses during driving. The minimum effective prestress level after losses is normally 700 to 800 psi (4.8 to 5.5 MPa), although very short piles have been installed with lower prestress levels. Long piles, batter piles, and piles that are expected to encounter alternating dense and soft lenses or strata may require higher prestress values, with effective prestress levels of 1000 to 1200 psi (6.9 to 8.3 MPa) frequently being used. Where bending resistance is a service requirement, higher values of prestress up to $0.2f_c'$ or more have been used without difficulty. Prestress values required to accommodate driving stress conditions should be determined by wave-equation analysis (3.3.2.2) or other acceptable means;

k) The pile should be properly cured for the anticipated driving conditions. Breakage can occur at pile heads and other locations during hard driving of a pile cast only a few days previously. Although adequate compressive strength can be developed in a few days by steam curing, the tensile strength and modulus of elasticity may increase more slowly. Whenever possible, piles should be at least 2 weeks old at the time of driving unless driving conditions are not difficult; and

l) Use appropriate techniques to prevent the development of internal pressures in hollow-core and cylinder piles (8.3.1.5).

8.3.3 *Bulging and distortion of heads of steel pipe*—This can be minimized by having the head of the pipe true, square, and even (preferably saw cut), and by using a tightly fitting driving head. For steel pipe, torsion is not a problem so that a tightly fitting drive head is permissible and helpful in preventing bulging.

8.3.4 *Dogleg and bent piles*—Axial alignment of a pile can be difficult to control in certain soils, particularly if boulders are encountered. The deflections can take the shape of long bends, sharp bends, or even breaks. The use of a pile or pile-mandrel combination of appropriate stiffness will help combat this driving problem. A flat pile tip generally causes less deflection than conical or pointed ones. Splices and joints should be strong enough to resist bending during driving, and adjoining pile sections should be accurately aligned.

Axial alignment can be verified by internal inspection in CIP pile shells and pipes after they are driven. This is also true of some hollow-core concrete piles and solid piles equipped with an inspection duct. This inspection should also verify the pile's internal cross-sectional area and that the full length of the pile can be properly concreted. A pipe pile with a long bend or dogleg is generally acceptable if any part of the pile tip is visible from the top. If this is not the case, electronic inclinometer measurements or other methods can be used to determine the geometry of the pile. If there are many such piles, it may be desirable to select the worst case for load testing to establish the maximum sweep that can be tolerated for the required capacity. Load tests on piles with long sweeping bends and doglegs have indicated substantial capacity resulting from the stiffness of the pile and lateral restraint from passive soil pressures.

8.3.5 *Misalignment of piles*—Specifying an axial-alignment tolerance as a percentage of actual length is common. The frequently specified tolerance of 2 percent can usually be met in relatively uniform soils with good equipment and good construction practice. In nonuniform or boulder-ridden soils, however, it is often impossible to prevent some piles from exceeding this tolerance, and larger tolerances may be appropriate (4.4.2). Excessive restriction on axial alignment often leads to an attempt to restrain the piles too much, thus introducing bending stresses that can be more detrimental. Proper initial alignment of the pile is important. The hammer should be guided in the leads so that the pile is struck squarely and concentrically. Proper alignment of the pile-driver leads and stable support for the pile-driving rig are essential.

Predrilling or spudding a starting hole can be helpful if material near the surface tends to deflect the pile. Alternately, it may be necessary to excavate and remove this material before starting pile installation. In boulder-laden soils, a boulder can fall into the hole as the spud is withdrawn, making the spudding ineffective or detrimental, in which case it may be better to drive the pile directly.

Piles exceeding the specified tolerance should be reviewed by the engineer for net horizontal forces, interference with adjacent piles, and the restraining effect of the pile cap as well as other groups structurally connected to the group having the misaligned piles.

8.3.6 *Distortion of piles*—Pile distortion can be produced during installation when driving past or through obstructions or boulders. For casings driven without a mandrel, the use of a heavier wall and a reinforced shoe will help. For shells driven with a mandrel, the use of a heavier shell thickness can help. The type of mandrel is important; while distortion will be minimized during driving if the mandrel grips the sides of the shell firmly, it should retract sufficiently to permit its withdrawal.

Shell and thin-walled pipe piles are subject to local buckling and collapse during driving and after the mandrel is withdrawn as a result of soil or hydrostatic pressures, or both, or as these pressures increase while driving adjacent piles. The use of thicker materials will prevent damage during driving. Collapse while driving adjacent piles can be prevented by using thicker pipe or shell, increasing the circumferential strength with corrugations, temporarily placing pipes having a marginally smaller diameter into the driven piles, or temporarily filling the pile with water. In very severe cases, the sequence of driving can be adjusted by placing and curing the concrete in the susceptible piles before driving the adjacent piles.

A similar phenomenon can take place with CIS piles. The installation of an adjacent pile can displace material into the fresh concrete before it has attained sufficient strength. This danger is more frequently associated with relatively incompressible, cohesive soils. The spacing of the piles is, of course, important. Using an accelerating admixture should help to reduce the time of exposure; this can then be coupled with a controlled sequence of driving. Uncased piles are much more vulnerable to distortion than cased piles.

8.3.7 *Distortion of pile tips*—Distortion of pile tips can occur as the tip encounters hard or irregular material, such as boulders. Reinforcement of the tip by a thicker bottom plate is recommended. When a mandrel is used, it should fit the tip uniformly and snugly. In some cases, prefilling or precasting of the tip section with concrete can minimize distortion. If the distortion is caused by surface rubble, pre-excavation of the obstructions may be more appropriate.

8.3.8 *Enlarged-tip piles*—When enlarged-tip piles are driven through certain soils, it may be necessary to take special measures to reestablish the lateral support of the soil around the pile shaft or to reinforce the pile shaft for column action. The annular space created by the enlarged tip might be filled in by the driving of adjacent piles, except that frequently such piles are used with relatively high capacities, resulting in the use of single piles or two-pile groups for each column. Any annular space should be filled with granular soils. If jetting or predrilling is necessary to achieve penetration of the enlarged tip, the possible loss of lateral support deserves special attention.

8.3.9 *Pile heave and flotation*—Refer to 3.3.8 for a discussion of pile heave and flotation, which can influence pile installation methods.

8.4—Handling and positioning during installation

Piles should be handled and positioned to obtain the proper pile location and alignment (vertical or batter) without impairing the pile's structural integrity.

8.4.1 *Handling*—Piles should be picked up so as to not cause local bulging or deformation, or induce excessive bending. Precast piles should be picked up and handled so as to avoid tensile cracks and any impact damage (4.2.1.1 and 7.7).

8.4.2 *Positioning*—Correct positioning requires accurate initial setting of the pile. Removal of near-surface obstructions will facilitate accurate positioning. Where accuracy of position is critical, a template, a predrilled starter hole, or both, can be useful. If techniques such as prejetting or predrilling are used, proper position control should also be exercised in making such pre-excavations.

Pile position is largely established when the pile is initially set. Attempts to correct position after driving has commenced often result in excessive bending and damage to the pile. Correction of position of piles during or after installation without risking damage usually requires extensive jetting along the pile length. This can cause undesirable weakening of the soil or other problems.

Reference stakes offset from the proper pile location before the start of driving will assist resetting the pile if significant movement is observed before the pile has penetrated too far. These stakes can also be used to determine pile drift from design location after the completion of driving, thereby making it possible to offset the placement of other piles in the group and limit group eccentricity in the pile cap. The use of such reference stakes to certify as-driven pile locations is not recommended; this should be done by a separate and independent survey after all piles in a group are driven.

8.4.3 *Control of alignment*—As with positioning, properly applied control of alignment should be exercised before driving begins. The driving rig should have stable support so that alignment of the leaders and pile does not shift during installation. If techniques such as prejetting or predrilling are used, proper alignment control should also be exercised in making such pre-excavations.

8.4.3.1 Both the driver leaders and the pile should be properly aligned to the required pile orientation (vertical or batter) before driving starts. Vertical pile alignment should be checked by means of a carpenter's level. Batter piles should be set with an appropriate template and level. Once the driving starts, the hammer blow should be delivered essentially axially, and excessive sway of the leaders prevented.

8.4.3.2 Pile support in the leaders should be provided where necessary for long piles. Batter piles should be supported to reduce gravity bending to acceptable limits; the use of rollers in the leaders is one such method. Slender vertical piles may require guides at intervals to prevent buckling under the hammer blow.

8.4.3.3 Use of a telescoping extension in the leaders may be required to prevent excessive bending and buckling of the pile length below the leaders when driving a long, unsupported length of pile below the bottom of the leaders, especially with batter piles.

8.4.4 *Protection against bending*—After installation in water, the pile should be protected against excessive bending from waves, current, dead weight (in case of batter pile), and accidental impact. Staying and girting should be used until the pile is finally tied into the structure. Pile heads should be stayed to eliminate bending; this is particularly relevant to batter piles where the head should be supported to overcome the dead weight. Frequently, when driving in deep water, a batter pile should be stayed before it is released from the hammer.

8.4.5 *Pulling into position*—The heads of piles, even in water, cannot be pulled into position without inducing bending. Many piles have been severely damaged structurally, even with relatively low pulling forces, because of the long lever arm available in many underwater installations. The designer should control the pulling force by specifying the maximum pull allowed at the top of the pile or the maximum allowable deflection (4.4.1).

8.5—Reinforcing steel and steel core placement

8.5.1 *General*—Required reinforcing steel should be placed in accordance with design drawings and be free of foreign material that will impair its bond. Preassembly into cages, with adequate spacer bars, will facilitate accurate placement. Bars should be well tied. Sufficient bars should be provided to give a frame or truss action if the cage is to be handled. Lateral ties can impede concrete placement (8.6.7); therefore, they should be of a size and spacing that minimizes placement problems.

In CIP or CIS piles, stopping all reinforcing bars at one elevation can create a plane of weakness (4.5.3). Some designers prefer to extend one or more bars toward the tip in the CIS or CIP piles with very thin shells to provide pile continuity.

8.5.2 *Dowels*—Dowels can be used to connect the pile head to the pile cap or structure above and to resist forces or movements at the head of the pile. For CIP piles, dowels can be held in position as normal reinforcement during placement of the concrete, or placed by inserting (vibrating) into the freshly placed concrete.

Dowels in precast piles can be partially embedded in the pile head and left protruding. In this case, the driving head will be constructed to have corresponding holes with enough play to prevent torsion or bending in the pile. Dowels can also be fully embedded, with the top portion being exposed after the pile is driven. Dowels can also be inserted into preformed holes cast in the pile or in holes drilled after the pile is driven. For formed holes, a flexible metal conduit is often used and can be left in place. If removable cores are used to form the holes, the parting compound used should be removed by flushing or other means so as not to impair the bond. Dowels inserted in preformed or drilled holes are grouted with either cement or epoxy grout. Dry packing is not recommended. Admixtures that reduce the shrinkage of cement grout are beneficial. Strands extending from prestressed piles can provide adequate doweling in many cases. Sufficient embedment length should be provided.

8.5.3 *Steel cores*—Steel cores, where specified, usually consist of reinforcing steel bundles, H-pile cores, or steel rail sections. Spacers or guides should be used so that the core is centered for the full length of embedment.

8.6—Concrete placement for CIP and CIS piles

A CIP or CIS pile is not complete until the concrete has been properly placed. Concrete placement operations for such piles are just as important to the successful completion of the pile as the driving or drilling of the pile. Concrete materials and placement methods are often dictated by field conditions and should be selected to prevent the development of voids and segregation of the coarse aggregates during concrete placement. Concrete placement methods should result in a uniform quality of concrete for the full design cross section throughout the length of the pile. As placed, the concrete should develop the required strength.

If the concrete is not properly placed, pile defects can develop that could cause the proposed structure to settle excessively. Some concrete defects that can develop in CIP and CIS piles are:

- Voids resulting from entrapped water, water migration, or incomplete concreting caused by arching, blockages, or shell collapse.
- Weak zones resulting from soil inclusions, foreign object inclusions, or a reduced pile cross section.
- Aggregate pockets resulting from coarse aggregate segregation during placement, or erosion of cement paste and fines by water migration.
- Weak concrete zones resulting from bleeding mixtures, excessive water present during concrete placement, and segregation.
- Separations, breaks, or displacements caused by surrounding construction activities, such as pile heave or lateral displacement caused by adjacent driving, lateral pressures and displacements from adjacent construction traffic, and lateral pressures and displacements related to adjacent excavations or fills.

Sometimes, the presence of potential defects is indicated during construction by:

- A drop in the concrete level at the pile head after concrete placement.
- Water seepage to the pile head from somewhere below.
- Excessive accumulations of laitance at the pile head.
- Excessive variation between the theoretical placement volumes and delivered concrete volumes.
- Pile load test failures or excessive settlement.
- Observations of obvious improper concreting procedures for the particular conditions.

The prevention of concrete defects and the identification of conditions conducive to their development in CIP and CIS piles require proper pile inspection before concrete placement, proper concrete materials, proper placement procedures, and experienced pile concreting personnel. Close coordination and cooperation between pile inspection and concrete placement personnel is required.

8.6.1 *Factors affecting placement*—The placement of concrete in CIP and CIS piles is affected by several factors, such as:

- *Soil and pile-installation conditions*—Pile spacing, installation sequence, pre-excavation methods, and soil conditions can affect the concrete placement techniques, as these items influence the potential soil pressures, leading to casing collapse with CIP piles and soil intrusion with CIS piles. The soil conditions also influence the pile lengths required and the potential for sweeps or doglegs that affect placement.
- *Pile configuration*—The potential for concrete segregation, arching, pile damage, and groundwater inflow are affected by the geometrical properties of the casing: diameter; wall thickness; pile shape (straight-sided, tapered, stepped); interior roughness (smooth, corrugated, fluted); frequency and configuration of joints; pile lengths; pile inclination (vertical versus battered); and pile straightness (straight, gentle sweeps, sharp sweeps, and doglegs). Therefore, these geometrical properties influence the selection of the placement procedures and materials.
- *Reinforcement*—The presence of reinforcing steel influences the placement techniques because the length; location; clearance; and spacing of longitudinal steel, lateral spiral or ties, and spacers holding the reinforcement in its design location can constrict flow and contribute to segregation and arching during concrete placement. The bar spacing and clearance should be considered in determining the maximum aggregate size and the vibration or rodding requirements to provide concrete flow through and around the reinforcement.
- *Condition of pile*—The conditions of the pile, such as presence of water, soil, or other debris, and ruptures and leaks, affect the techniques that are required to clean the pile in preparation for concreting. If the inflow of groundwater into the pile cannot be controlled, it may dictate the use of special underwater placement techniques such as tremie or pump placement.
- *Concrete mixture proportioning*—The design mixture properties, such as slump, ratio of coarse-to-fine aggregate, maximum coarse aggregate size, *w/cm*, cement factor, and admixtures, affect the workability and cohesiveness of the mixture and the quality of the placed material. When selecting or establishing the design mixture, the placement techniques and desirable

mixture properties to combat the obstacles listed in the preceding four items should be considered.

8.6.2 *Inspection before concreting*—After a CIP pile is driven, it should be inspected to be certain that it has not been closed in or partially filled by soil movements or pressure. Such inspections would also reveal the presence of any foreign material or excessive amounts of water, as well as any detrimental damage to any casing used. Such inspections should include not only visual observations with a mirror or high-intensity light, but also quantitative verification of inside length and diameter, and the depth of any water, soil, debris, or other obstructions to concrete placement that are present. Leaky, damaged, or otherwise obstructed piles that cannot be dewatered and cleaned adequately to permit proper concrete placement should be identified so that replacement piles, if necessary, can be driven while the driving rig is still nearby.

If there will be a delay before the pile is concreted, as is frequently the case, the piles should be covered for protection from inflow of surface water, soil, pre-excavation spoil, and other debris until the concreting takes place. The pile should then be reinspected immediately before concrete placement. When concrete placement is occurring at the same time as pile installation, it is generally impossible for a single inspector to properly inspect both operations. It is essential in such cases that the inspection and construction crews for both operations are properly staffed with qualified personnel.

8.6.3 *Leaking of piles*—Leaking of pipe or shells is an indication of a rupture or unsealed joint(s). Leaky piles should always be checked for distortion, collapse or separation, and the presence of soil or debris. If water, soil, or debris is present in the pile, the soil and debris should be thoroughly cleaned out; the water should be drawn down to an acceptable level, normally 2 in. (50 mm) maximum depth; and the pile should be reinspected before it can be accepted for concreting. Various methods are available to remove this material, such as by internal jet, airlift, compressed-air blowout, and pumping. In severe cases of water inflow not accompanied by soil inflow, it may be possible to concrete the pile by tremie methods. These require care, skill, control, and experience, and should be permitted only under qualified supervision.

8.6.4 *Concrete mixture proportions*—Concrete mixture proportions for CIP piles should be designed to have adequate workability and flow characteristics so that the concrete can be placed under the particular conditions and develop the required strength. For conventional structural-grade concrete placed in the dry, slumps of 4 to 6 in. (100 to 150 mm) are usually desirable. The concrete mixture should contain a cement content of at least 564 lb/yd^3 (335 kg/m^3) and the maximum aggregate size should usually be limited to 3/4 in. (19 mm). The mixture should not bleed excessively. Bleeding is affected primarily by the properties of the cement and the physical properties of the fine aggregate. Cement-rich mixtures are less prone to bleeding than lean ones. The use of admixtures can be beneficial in obtaining the desired workability and nonbleeding characteristics (Chapter 6).

Concrete mixtures containing approximately 800 lb/yd^3 (475 kg/m^3) of coarse aggregate (less than half that of conventional structural concrete) and with a corresponding increase in sand and cement content have been found to produce a very workable and highly cohesive mixture with a slump of approximately 4 in. (100 mm) (Raymond International 1970; Snow 1976; Fuller 1983). These mixtures are especially useful when addressing difficult placement conditions, such as reinforced piles where the concrete has to be placed through the reinforcement cage, batter piles, and very long piles with extensive corrugations or steps. Such mixtures can be pumped, tremied, or placed by conventional methods. While these mixtures require 40 to 100 lb (18 to 45 kg) more cement per 1 yd^3 (0.8 m^3) for comparable strengths, the precharge of grout required with conventional mixtures is not generally required with reduced-coarse-aggregate mixtures (8.6.5.1). Most properly designed pump mixtures used with piles have a reduced coarse aggregate content and maximum aggregate size, resulting in high sand and cement contents and behavior similar to that of the mixtures with reduced coarse aggregate described previously.

8.6.5 *Concrete placement methods and techniques*—Concrete should not be dumped directly onto the top of the pile. If placed from the top, it should be deposited through a steep-sided funnel hopper. Concrete for CIP piles can be satisfactorily placed by tremie, bottom-dump bucket, or pumping in addition to conventional placement through a funnel at the top of the pile. The selection of proper placing methods and techniques is dictated by field conditions and available equipment.

8.6.5.1 *Dry placement*—Conventional concrete placement for dry piles consists of depositing the concrete from the top through a steep-sided funnel with a discharge spout diameter at least 2 in. (50 mm) smaller than the pile top diameter and not larger than the smallest diameter of the pile. A spout diameter of approximately 8 to 10 in. (200 to 250 mm) generally works well, although a diameter as small as 6 in. (150 mm) may be required when placing concrete through a reinforcement cage. The funnel should be centered on the pile and should be supported up off of the pile top so that the displaced air from the pile can freely escape. Immediately before concrete placement, the pile should be inspected, or reinspected, to be sure that it is free of foreign matter, including appreciable water (2 in. [50 mm] maximum depth).

When using conventional structural concrete, it is frequently specified that a small batch of rich grout (generally one part cement and two parts concrete sand, and water) be placed in the pile immediately before the concrete placement. The purpose of the grout is to partially precoat the pile sides and reinforcement with a mortar mixture and supply a charge of rich cement grout to the tip of the pile to counteract the segregation of coarse aggregate at the pile tip during the initial charge of concrete. The decision to require the use of a precharge of grout is dependent on not only the length and configuration of the pile but also other variables, such as the maximum coarse aggregate size, percentage of coarse aggregate in the mixture, and the cohesiveness of the concrete mixture. When using mixtures with reduced coarse aggregate, or other cohesive mixtures that have high cement and sand contents, a precharge of rich grout is not typically required.

Provided the concrete is placed in dry conditions using a steep-sided funnel centered on the pile as described herein, a grout precharge before concrete placement is generally not required for piles with lengths shorter than 50 ft (15 m), vertical sides, diameters greater than approximately 12 in. (300 mm), and without reinforcing cages or with cages set after the concrete has been placed to the bottom of the cage. Difficult placing conditions can cause segregation of the concrete mixture due to contact of the concrete with the sides of the pile, steps, or reinforcement during its fall. Such conditions are found in battered piles, tapered or stepped piles, heavily reinforced piles, and long piles with extensive sweeps or doglegs; for these, a precharge of grout before concreting is recommended.

The amount of grout required varies with the placing conditions. For piles up to 50 ft (15 m) long with little or no reinforcement, approximately 0.5 ft^3 (0.014 m^3) of grout is typically used. For piles that are longer, battered, tapered or stepped, heavily reinforced, or with extensive sweeps or doglegs, approximately 1 to 1.5 ft^3 (0.03 to 0.04 m^3) of grout should be used.

Concrete should be discharged into the funnel as rapidly as possible without spilling from the truck, chute, or funnel. The flow should be uninterrupted. High flow is required when filling piles, especially for the first part of the placement when the initial several feet of the pile is placed. A rapid discharge provides a larger concrete volume through which any water present at the tip will be distributed, thus minimizing the influence of the water on the concrete.

When placing conventional structural concrete mixtures (not reduced-coarse-aggregate or pumpable mixtures), the use of bottom-dump tubes is recommended for concrete placement in the lower portions of long corrugated-shell piles. These dump tubes are generally 8 to 10 in. (200 to 250 mm) in diameter, 6 to 12 ft (2 to 4 m) long, have bottom doors or flaps that can be released from the top, and a steep-sided funnel on the top of the tube. With this technique, the tube is filled with concrete and then the bottom door is tripped, sending the 2.5 to 5 ft^3 (0.07 to 0.14 m^3) charge to the pile bottom. The bottom door is relatched and the process repeated until the pile is filled to within approximately 50 ft (15 m) of cutoff elevation, where placement can be completed by continuous flow through the dump tube or through the standard, steep-sided funnel. When using this technique, the 0.5 to 1 ft^3 (0.014 to 0.028 m^3) batch of rich grout normally placed in the pile immediately before conventional concrete placement should be placed in the bottom of the dump tube on the initial charge.

8.6.5.2 *Underwater placement*—Where significant water is present in the pile, as in open-ended pipe piles, or where leakage is excessive, as can occur in shells, underwater placement is used. Underwater placement can use either tremie or pump methods. For either of these methods, the pile casing is purposely filled with water and cleaned out by flushing or other means as described in 8.6.3. Fine-grained material that remains in suspension is displaced by the tremie concrete.

For tremie placement, a smaller-diameter pipe with a plugged end is lowered to the bottom of the pile. The pipe is filled with a suitable tremie concrete mixture when resting on the bottom. The pipe is then gradually raised, keeping the tip well-embedded in the concrete and avoiding sudden shock or disturbance. When tremie placement is used, it is preferable to cast the entire pile in one placement for the full height, avoiding a cold joint.

For pump placement, a smaller-diameter pipe with a plugged end is lowered to the bottom of the pile. When resting on the bottom, it is filled under pressure with suitable pumped concrete. As the pumped concrete enters the pile shaft, the pipe should be raised gradually, keeping the discharge nozzle well embedded in the concrete.

When pump placement is used, it is preferable to cast the entire pile in one placement for the full height, avoiding a cold joint. Normally, the flow of concrete is continued until the concrete emerging from the top of the pile has the same quality as at the mixer, with no excess water. If laitance develops after the completion of concrete placement, it should be thoroughly cleaned and replaced.

If the tremie or pumping methods are used only to place a seal in the lower portion of the pile, then the surface should be carefully cleaned and laitance removed before the remaining concrete is placed. Removal of laitance is more difficult as the pile diameter decreases and if reinforcement is present at the joint.

8.6.6 *Concrete consolidation and vibration*—Mechanical vibration is generally not required in ordinary CIP piles that do not contain reinforcement, provided that proper concrete mixtures with high slumps and good workability are used. The reason for this is that the high pressures and flow characteristics of the high slump, 3/4 in. (19 mm) minus aggregate mixture, consistent with normal pile concreting practice, will lead to adequate consolidation, except in approximately the upper 5 ft (1.5 m) of the pile, where the concrete should be rodded. The upper 5 to 15 ft (1.5 to 4.5 m) of piles with reinforcement may require mechanical vibration, depending on the reinforcement spacing, maximum aggregate size, and the flow characteristic of the concrete mixture. Vibration, if necessary, can be accomplished by rodding or with an internal vibrator. Over-vibration should be avoided because it can induce excessive bleeding.

8.6.7 *Obstruction to concrete placement*—Steps in shells and reinforcing ties can cause segregation and voids unless the mixture is sufficiently fluid and workable to prevent arching. Vibration in accordance with 8.6.6 may be desirable under these circumstances.

8.6.8 *Compaction of uncased pile*—Some CIS piles use ramming during concrete placement to compact and consolidate the concrete (1.2.3). In those pile types where the casing is simultaneously withdrawn, care should be exercised to overcome pull-up effect on concrete (arching within the casing) during withdrawal of casing and provide adequate concrete so that if a weak stratum is encountered and the concrete is pushed out to fill the void, continuity of the structural column is not impaired.

Enough concrete should be provided to make up for concrete forced laterally into the soil. The techniques used for compaction and casing withdrawal should prevent separation of the column.

8.6.9 *Cast-in-drilled-hole piles*—Cast-in-drilled-hole piles 30 in. (760 mm) and larger are covered in ACI 336.1-01 and ACI 336.3R-93. The discussion of construction methods and precautions in these publications are, in general, equally applicable to the cast-in-drilled-hole piles covered herein. The placing of concrete in cast-in-drilled-hole piles as covered by this report should follow the same basic procedures as that for CIS concrete piles. For unstable soils, a temporary liner should be installed to prevent collapse of the hole or sloughing off of the soil during concrete placement. Temporary liners should also be used for deep drilled holes when the effects of concrete placement on the sides of the hole cannot be observed. When placing concrete in temporarily lined holes, the top of the concrete should be kept well above the bottom of the steel liner as it is withdrawn. Low-slump concrete should not be used to avoid the possibility of arching of the concrete in the liner and possible discontinuities in the pile shaft as the liner is withdrawn.

8.6.10 *Auger-grout or concrete-injected piles*—Auger-grout piles (1.2.7.3 and 1.2.7.4) are installed by drilling a hole to a predetermined depth with a continuous-flight, hollow-stem auger, plugged at the tip. The auger is then lifted slightly (6 to 12 in. [150 to 300 mm]) and fluid grout, or concrete, is pumped into the auger stem under sufficient pressure to eject the plug and begin forcing grout upward in the auger flights. The auger is then slowly withdrawn while continuously pumping grout under pressure to prevent collapse of the hole. The completed grout column forms a CIS pile.

Auger-grout piles are frequently used instead of driven piles to limit damage to adjacent structures or avoid vibrations and noise. When used improperly, however, they can cause damage to adjacent structures (Lacy and Moskowitz 1993). Additional information on the installation of auger-grout piles is provided by Neely (1990), Moskowitz (1994), Frizzi (2003), and Brown et al. (2007).

Grout should conform to the recommendations of 6.5. If concrete is used, it should contain sufficient cement, properly sized aggregates, and required admixtures to produce a rich, pumpable mixture. Oil and other rust inhibitors should be removed from mixing drums and grout or concrete pumps.

When filling the drilled hole as the auger is withdrawn, careful control is essential to prevent separation or necking of the grout or concrete shaft and to provide a shaft of full cross-sectional area. Each pile should be installed in one continuous operation. Concrete or grout should be pumped continuously, and the rate of withdrawal of the auger should be controlled so that the hole is completely filled as the auger is withdrawn. If there is evidence that the auger has been withdrawn too rapidly, it should be redrilled to the original tip elevation and the pile recast from the tip upward.

The volume of grout or concrete placed should be measured and be greater than the theoretical volume of the hole created by the auger. The top of each pile should be cast higher than the required pile cutoff elevation to permit trimming the pile back to sound grout or concrete. Unless the soil is sufficiently stable to resist the pressure from the grout or concrete shaft without lateral movement while adjacent piles are installed, the adjacent piles should not be installed until the grout or concrete has set.

If reinforcement is required, the reinforcing bars or cages should be accurately positioned, aligned, and inserted into the pile shaft while the grout or concrete is still fluid. A single reinforcing bar can be installed through the hollow stem before grouting.

8.6.11 *Drilled and grouted piles*—Drilled and grouted piles (1.2.7.5) are advanced by rotating a heavy-wall casing into the ground with wash water returning up the outside of the casing. When boulders are present, a saw-toothed bit or a disposable tricore bit is sometimes used to advance the casing. In some cases, internal rotary drills are used to advance the casing, and the return may be through the annulus between the drill shaft and the casing. As the casing reaches the planned pile depth, reinforcing steel is placed, the drilling fluid is switched to sand-cement or neat-cement grout, and the hole is filled from the bottom up as the casing is withdrawn while grout continues to be pumped. In other instances, the casing may be only partially withdrawn through a planned pile bond zone and then rotated downward into the bond zone and left in place. When left in place, the steel casing provides a large part of the structural capacity while the cement bond between the outside of the casing and the soil provides high side resistance and load transfer to surrounding soil. Regrouting is sometimes used to increase pile capacity within the bond zone using a pipe with grouting ports to inject grout at discrete levels.

When installing drilled and grouted piles, care should be taken so that a full-sized and continuous pile is produced. All soil cuttings should be removed from the casing except those that will remain in suspension and be displaced with the drilling fluid. Reinforcing steel should have sufficient spacers to hold it in position. This is especially important when installing batter piles. Grout should conform to 6.6 and the casing should not be withdrawn faster than the hole is being filled with grout.

8.7—Pile details

8.7.1 *Tips*—The tips of piles should be strong and rigid enough to resist distortion. Adequate wall thickness, reinforced as necessary, should be used for CIP pile shells. Steel tip-plates should have sufficient thickness to withstand local distortion. The connection (weldment or drive-fit assembly) between the tip plate and shell should be watertight and able to withstand repeated impact.

Pointed or wedge-shaped tips can aid penetration through overlying riprap, boulders, or miscellaneous fills, and can also be used to help penetration into decomposed rock. Such tips, however, can guide the pile off axial alignment. Blunt (rounded) tips will often accomplish the penetration through rock or surface rubble with a minimum of misalignment and point breakage. Flat tips drive straighter and truer than pointed tips.

8.7.2 *Shoes for precast piles*—When the tips of precast and prestressed concrete piles are provided with adequate transverse reinforcement (spiral confinement) and the corners of square piles are chamfered to prevent stress concentrations and spalling (7.2.4 and 8.3.1.2), special pile shoes are not generally required nor used. Cast or fabricated steel points and flat steel plates are sometimes beneficial when piles need to penetrate buried timber, riprap, or weak rock. Shoes with short dowels have also been reported to be helpful in seating precast piles on sloping rock surfaces. When shoes are used with precast concrete piles, the potential effects of shape on penetration and pile alignment are the same as with other pile types (8.3.4 and 8.7.1).

Shoes should be securely attached to the main body of the pile by anchor rods. These rods should have sufficient embedment length to develop anchorage by bond under the repeated high-stress loading that can occur under hard driving at the tip, and the anchor rods should be securely attached to the plate or shoe. Particular care should be taken to properly place and consolidate concrete in the shoe during casting. Depending on the configuration of the shoe, vent holes in the shoe may be required (7.5.3.1).

8.7.3 *Stubs for prestressed piles*—Structural steel stubs (stingers) are sometimes used as extensions from the tips of prestressed piles. Structural steel stubs most frequently consist of heavy H-pile sections, but other structural shapes, fabricated crosses, steel rail, and large-diameter dowels have also been used.

Stubs can be used to break up and penetrate hard strata, such as coral or limerock, ahead of the pile or to secure penetration of soft or weathered rock. To perform this function, the stubs should be of sufficient thickness, stiffness, and strength to prevent their own distortion. Stubs are frequently used under conditions known to be conducive to damage of structural steel shapes, such as H-piles, during driving. Therefore, structural steel stubs should generally be provided with cast or fabricated steel tips.

Stubs can be welded to steel plates that are in turn anchored to the pile. They are, however, most frequently anchored by direct embedment of the stub into the body of the precast pile. Design of the stub attachment and placement of concrete in the area of the stub require special attention (4.5.3.5 and 7.5.3.1).

8.7.4 *Splices*—During driving and under service conditions, splices should develop the requisite strength in compression, bending, tension, shear, and torsion at the splice. Splices can sometimes be located so that these requirements are minimized; direct bearing (compression) is often the only condition requiring full pile strength. Splice details are discussed in 4.4.4.

8.7.4.1 Design of welded splices in shells or precast pile joints should consider the effect of repeated impact. Welding rod and techniques used should be in accordance with ANSI/AWS D1.1/D1.1M:2010 and AWS D1.4/D1.4M:2011 and selected for impact conditions. When welded splices are used with precast piles, the effect of heat and consequent splitting and spalling near the splice needs to be overcome.

8.7.4.2 Backup plates or other suitable techniques should be used to develop full weld penetration when splicing load-bearing steel shells, especially for shells 3/8 in. (10 mm) or thicker.

8.7.4.3 When splicing precast or prestressed piles, special care should be taken to avoid a discontinuity at the point of splice, which will result in tensile failure of the pile. Doweled splices using cement or epoxy grout have been used successfully with precast piles under widely varying conditions, and accomplish continuity if properly installed (Bruce and Hebert 1974b). Adequate curing before driving is essential.

A number of manufactured splices are available to quickly and effectively splice precast or prestressed concrete piles (Bruce and Hebert 1974a; Venuti 1980; Gamble and Bruce 1990). The pile fabrication methods and forms should accommodate the specific splice that is to be used. Care should be taken in splicing to provide concentric alignment, full bearing at the interface, and the tensile adequacy of the connection. The designer should exercise control over the use of splices in precast piles and the splice design requirements (4.4.4).

8.7.4.4 Outside drive sleeves have been used successfully to splice both precast concrete and steel pipe piles. Inside sleeves can be used for steel pipe, but these sleeves are not as effective as outside sleeves for a drive fit and have to be fabricated for both pile diameter and wall thickness.

8.7.5 *Cutoff of precast or prestressed piles*—Precast or prestressed piles should be cut off at the required elevation by techniques that will prevent spalling or weakening of the concrete. The selected cutoff technique should also not damage the reinforcement when exposed reinforcement or prestressing tendons are used to connect the pile to the structure.

A circumferential cut around the pile head will permit the use of hydraulic breakers without spalling. Various mechanical and hydraulic tools are available to cut concrete piles quickly and effectively. Concrete crushing equipment is also available to break up the pile waste materials and thereby simplify the disposal process, but should not be used on the pile itself.

Clamps of timber or steel help prevent spalling. In general, explosives should not be used as a means of cutting off concrete piles.

8.7.6 *Extension of precast piles*—Extensions are used when the pile has been driven a short distance below grade. Lowering the pile cap or capital at the low pile is often the best solution if the pile top has not been driven too far below cutoff grade. Extension of the pile section itself, as reinforced concrete and with dowels into the pile, is adequate only when comparable section strengths can be obtained.

Pile sections can be spliced on for extensions, as in 8.7.4. Special care to provide for durability should be taken at the splice where the pile is subject to marine and other adverse exposure.

8.8—Extraction of concrete piles

Concrete piles can be extracted by direct pull, jetting, vibration, excavation, jacking, or a combination of these means. For piles developing their capacity primarily through friction, redriving the pile just before starting the extraction operation can aid in extraction by breaking the soil friction or freeze along the sides.

Large, expensive piles, such as cylinder piles, are occasionally pulled and reused. Pulling often introduces bending stresses that cause cracks in piles. These can be serious enough to prevent reuse or can produce discontinuities that will damage the pile on redriving (8.3.1). To minimize cracking, the slings should be arranged to pull axially. A double sling leading over an equalizing sheave and pulling on each side of the pile has been used with success. Before a pulled pile is reused, its condition should be carefully assessed (8.3.1.6).

8.9—Concrete sheet piles

Precast and prestressed concrete sheet piles, with tongue-and-groove joints, are installed like other concrete piles, with the following points being emphasized or given special consideration.

8.9.1 *Installation*

- Jetting is frequently useful and necessary. Gang jets can be useful.
- Accurate setting is essential. Falsework or guides are usually necessary.
- Tips should be beveled at the leading edge so that the pile tip drives toward the adjacent, previously driven pile.
- During driving, the head is continuously pulled in toward the previous pile.
- Tongued edge should lead where possible, as soil will otherwise wedge in the groove.
- To facilitate placing of the hammer and driving head on individual sheet piles, extending the pile head 18 to 24 in. (450 to 600 mm) with a reduced (tapered) width, to approximate a square concrete pile head is often desirable. Otherwise, the helmet and hammer can hit the adjoining sheet pile. This extension can later be cut off, exposing the strands for tying into the coping or cap.

8.9.2 *Special care*—Special care should be taken to prevent wings of the grooved edge from breaking off during driving. This can be minimized by accuracy in setting, the use of jets to assist driving, and provision of light reinforcement in the wings.

8.9.3 *Grouting of joints*—Joints can be grouted directly or by first inserting a light fabric tube. If the tube is slightly porous (burlap, for example), some bond will develop. Polyethylene and canvas tubes have been widely used. A jet can be used to first clean out the joint before grouting. Groove joints can be provided on both sides of adjoining precast and prestressed concrete sheet piles from the top of the piles to 4 ft (1.2 m) below low water or grade on the exterior side of the sheet to allow for grouting of the joints. The *PCI Design Handbook* (PCI 2005) contains details of standard prestressed concrete sheet piles.

CHAPTER 9—REFERENCES

9.1—Referenced standards and reports

American Concrete Institute

117-10 – Specification for Tolerances for Concrete Construction and Materials (ACI 117-10) and Commentary
201.2R-08 – Guide to Durable Concrete
211.1-91 – Standard Practice for Selecting Proportions for Normal, Heavyweight, and Mass Concrete
211.4R-08 – Guide for Selecting Proportions for High-Strength Concrete Using Portland Cement and Other Cementitious Materials
212.3R-10 – Report on Chemical Admixtures for Concrete
212.4R-04 – Guide for Use of High-Range Water-Reducing Admixtures (Superplasticizers) in Concrete (withdrawn)
221R-96 – Guide for Use of Normal Weight and Heavyweight Aggregates in Concrete
221.1R-98 – Report on Alkali-Aggregate Reactivity
222R-01 – Protection of Metals in Concrete against Corrosion
228.2R-98 – Nondestructive Test Methods for Evaluation of Concrete in Structures
301-10 – Specifications for Structural Concrete
304R-00 – Guide for Measuring, Mixing, Transporting, and Placing Concrete
305R-10 – Guide to Hot Weather Concreting
306R-10 – Guide to Cold Weather Concreting
308.1-11 – Standard Specification for Curing Concrete
308R-01 – Guide to Curing Concrete
309R-05 – Guide for Consolidation of Concrete
315-99 – Details and Detailing of Concrete Reinforcement
318-08 – Building Code Requirements for Structural Concrete (ACI 318-08) and Commentary
336.1-01 – Specification for the Construction of Drilled Piers
336.3R-93 – Design and Construction of Drilled Piers
347-04 – Guide to Formwork for Concrete
503R-93 – Use of Epoxy Compounds with Concrete (withdrawn)
517.2R-92 – Accelerated Curing of Concrete at Atmospheric Pressure (withdrawn)

American Society of Civil Engineers

ASCE/SEI 7-05 – Minimum Design Loads for Buildings and Other Structures

American Welding Society

ANSI/AWS D1.1/D1.1M:2010 – Structural Welding Code—Steel
AWS D1.4/D1.4M:2011 – Structural Welding Code—Reinforcing Steel

ASTM International

A36/A36M-08 – Standard Specification for Carbon Structural Steel
A242/A242M-04(2009) – Standard Specification for High-Strength Low-Alloy Structural Steel

A252-10 – Standard Specification for Welded and Seamless Steel Pipe Piles
A283/A283M-03(2007) – Standard Specification for Low and Intermediate Tensile Strength Carbon Steel Plates
A416/A416M-10 – Standard Specification for Steel Strand, Uncoated Seven-Wire for Prestressed Concrete
A421/A421M-10 – Standard Specification for Uncoated Stress-Relieved Steel Wire for Prestressed Concrete
A572/A572M-07 – Standard Specification for High-Strength Low-Alloy Columbium-Vanadium Structural Steel
A615/A615M-09 – Standard Specification for Deformed and Plain Carbon-Steel Bars for Concrete Reinforcement
A706/A706M-09 – Standard Specification for Low-Alloy Steel Deformed and Plain Bars for Concrete Reinforcement
A722/A722M-07 – Standard Specification for Uncoated High-Strength Steel Bars for Prestressing Concrete
A775/A775M-07 – Standard Specification for Epoxy-Coated Steel Reinforcing Steel Bars
A882/A882M-04(2010) – Standard Specification for Filled Epoxy-Coated Seven-Wire Prestressing Steel Strand
A884/A884M-06 – Standard Specification for Epoxy-Coated Steel Wire and Welded Wire Reinforcement
A955/A955M-11 – Standard Specification for Deformed and Plain Stainless-Steel Bars for Concrete Reinforcement
A970/A970M-09 – Standard Specification for Headed Steel Bars for Concrete Reinforcement
A996/A996M-09 – Standard Specification for Rail-Steel and Axle-Steel Deformed Bars for Concrete Reinforcement
A1008/A1008M-11 – Standard Specification for Steel, Sheet, Cold-Rolled, Carbon, Structural, High-Strength Low-Alloy, High-Strength Low-Alloy with Improved Formability, Solution Hardened, and Bake Hardenable
A1011/A1011M-11 – Standard Specification for Steel, Sheet and Strip, Hot-Rolled, Carbon, Structural, High-Strength Low-Alloy, High-Strength Low-Alloy with Improved Formability, and Ultra-High Strength
A1064/A1064M-10 – Standard Specification for Steel Wire and Welded Wire Reinforcement, Plain and Deformed, for Concrete
C31/C31M-10 – Standard Practice for Making and Curing Concrete Test Specimens in the Field
C33/C33M-11 – Standard Specification for Concrete Aggregates
C39/C39M-11 – Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens
C109/C109M-11 – Standard Test Method for Compressive Strength of Hydraulic Cement Mortars (Using 2-in. or [50-mm] Cube Specimens)
C138/C138M-10 – Standard Test Method for Density (Unit Weight), Yield, and Air Content (Gravimetric) of Concrete
C143/C143M-10 – Standard Test Method for Slump of Hydraulic-Cement Concrete
C150/C150M-11 – Standard Specification for Portland Cement
C172/C172M-11 – Standard Practice for Sampling Freshly Mixed Concrete
C173/C173M-10 – Standard Test Method for Air Content of Freshly Mixed Concrete by the Volumetric Method
C227/C227M-10 – Standard Test Method for Potential Alkali Reactivity of Cement-Aggregate Combinations (Mortar-Bar Method)
C230/C230M-08 – Standard Specification for Flow Table for Use in Tests of Hydraulic Cement
C231/C231M-10 – Standard Test Method for Air Content of Freshly Mixed Concrete by the Pressure Method
C260/C260M-10 – Standard Specification for Air-Entraining Admixtures for Concrete
C295/C295M-11 – Standard Guide for Petrographic Examination of Aggregates for Concrete
C309/C309M-11 – Standard Specification for Liquid Membrane-Forming Compounds for Curing Concrete
C403/C403M-08 – Standard Test Method for Time of Setting of Concrete Mixtures by Penetration Resistance
C494/C494M-11 – Standard Specification for Chemical Admixtures for Concrete
C595/C595M-11 – Standard Specification for Blended Hydraulic Cements
C618-08 – Standard Specification for Coal Fly Ash and Raw or Calcined Natural Pozzolan for Use in Concrete
C939-10 – Standard Test Method for Flow of Grout for Preplaced-Aggregate Concrete (Flow Cone Method)
C989-10 – Standard Specification for Slag Cement for Use in Concretes and Mortars
C1017/C1017M-07 – Standard Specification for Chemical Admixtures for Use in Producing Flowing Concrete
C1074-11 – Practice for Estimating Concrete Strength by the Maturity Method
C1240-11 – Standard Specification for Silica Fume Used in Cementitious Mixtures
C1611/C1611M-09 – Standard Test Method for Slump Flow of Self-Consolidating Concrete
D1143/D1143M-09 – Standard Test Methods for Deep Foundations under Static Axial Compressive Load
D3689-07 – Standard Test Methods for Deep Foundations under Static Axial Tensile Load
D3966-07 – Standard Test Method for Deep Foundations under Lateral Load
D4945-08 – Standard Test Method for High Strain Dynamic Testing of Deep Foundations
D5882-07 – Standard Test Method for Low Strain Impact Integrity Testing of Deep Foundations

D6066-96(2004) – Standard Practice for Determining the Normalized Penetration Resistance of Sands for Evaluation of Liquefaction Potential
D6760-08 – Standard Test Method for Integrity Testing of Concrete Deep Foundations by Ultrasonic Crosshole Testing
D7383-10 – Standard Test Methods for Axial Compressive Force Pulse (Rapid) Testing of Deep Foundations

International Code Council
IBC 1808.2.9-2006 – International Building Code

Precast/Prestressed Concrete Institute
MNL 116-99 – Manual for Quality Control for Plants and Production of Precast and Prestressed Concrete Products

U.S. Army Corps of Engineers
CRD-C6-11 – Standard Test Method for Flow of Grout for Preplaced-Aggregate Concrete (Flow-Cone Method)
CRD-C79-77 – Test Method for Flow of Grout Mixtures (Flow-Cone Method)

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Guide to Design, Manufacture, and Installation of Concrete Piles

The AMERICAN CONCRETE INSTITUTE

was founded in 1904 as a nonprofit membership organization dedicated to public service and representing the user interest in the field of concrete. ACI gathers and distributes information on the improvement of design, construction and maintenance of concrete products and structures. The work of ACI is conducted by individual ACI members and through volunteer committees composed of both members and non-members.

The committees, as well as ACI as a whole, operate under a consensus format, which assures all participants the right to have their views considered. Committee activities include the development of building codes and specifications; analysis of research and development results; presentation of construction and repair techniques; and education.

Individuals interested in the activities of ACI are encouraged to become a member. There are no educational or employment requirements. ACI's membership is composed of engineers, architects, scientists, contractors, educators, and representatives from a variety of companies and organizations.

Members are encouraged to participate in committee activities that relate to their specific areas of interest. For more information, contact ACI.

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