# Design, Manufacture, and Installation of Concrete Piles

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*This report presents recommendations to assist the design architect/engineer, manufacturer, field engineer, and contractor in the design and use of most types of concrete piles for many kinds of construction projects. The introductory chapter gives descriptions of the various types of piles and definitions used in this report.*

*Chapter 2 discusses factors that should be considered in the design of piles and pile foundations and presents data to assist the engineer in evaluating and providing for factors that affect the load-carrying capacities of different types of concrete piles.*

*Chapter 3 lists the various materials used in constructing concrete piles and makes recommendations regarding how these materials affect the quality and strength of concrete. Reference is made to applicable codes and specifications. Minimum requirements and basic manufacturing procedures for precast piles are stated so that design requirements for quality, strength, and durability can be achieved (Chapter 4). The concluding Chapter 5 outlines general principles for proper installation of piling so that the structural integrity and ultimate purpose of the pile are achieved. Traditional installation methods, as well as recently developed techniques, are discussed.*

**Keywords:** augered piles; bearing capacity; composite construction (concrete and steel); concrete piles; corrosion; drilled piles; foundations; harbor structures; loads (forces); prestressed concrete; quality control; reinforcing steels; soil mechanics; storage; tolerances.



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# CHAPTER 1—INTRODUCTION

## 1.0—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 of piles, and new types are still being developed. The following discussion deals with only the types of piles currently used in North American construction projects.

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 can be used untreated when the pile is positioned entirely below the permanent water table. The design of steel and timber piles is not considered herein except when they are 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. Postgrouting may be used after installation to densify the soil under the pile tip.

Concrete piles can also be 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.

## 1.1—Types of piles

**1.1.1** *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 must be designed and manufactured to withstand handling and driving stresses in addition to service loads.

**1.1.1.1** *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.

**1.1.1.2** *Prestressed concrete piles*—These piles are constructed using steel rods, strands, or wires under tension. The stressing steel is typically enclosed in a wire spiral. Nonmetallic strands have also been used, 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 on the job site.

**1.1.1.3** *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 if desirable for design purposes.

**1.1.2** *Cast-in-place concrete piles*—Generally, CIP piles involve 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 redrive 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 (known as step-taper). Pipe is also available in similar configurations, but normally is of uniform section or a uniform section with a tapered tip.

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

**1.1.3** *Enlarged-tip piles*—In granular soils, pile-tip enlargement generally increases pile bearing capacity. One type of enlarged-tip 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. 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. The precast, enlarged-tip base can be constructed in a wide range of sizes.

**1.1.4** *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.

**1.1.5** *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.

**1.1.6** *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 wood pile. Special conditions that can make such a pile economically attractive are:

- A long length is required;
- An inexpensive source of timber is available;
- The timber section will be positioned below the permanent water table; and
- A relatively low capacity is required.

Another common composite pile is a precast pile on top of a steel H-section tip with a suitably reinforced point. 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 portion, is filled with concrete, and reinforcing steel can be added as dictated by the design.

**1.1.7** *Drilled piles*—Although driven piles can be predrilled, the final operation involved in their installation is driving. Drilled piles are installed solely by the process of drilling.

**1.1.7.1** *Cast-in-drilled-hole pile*[1]—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 ensure that the concrete shaft placed does not contain separations caused by the frictional effects of withdrawing the liner.

**1.1.7.2** *Foundation drilled piers or caissons*—These are deep foundation units that often function like piles. They are essentially end-bearing units and 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, but is included in the reports of ACI 336.1, ACI 336.1R, and ACI 336.3R.

**1.1.7.3** *Auger-grout or concrete-injected piles*—These 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.

**1.1.7.4** *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. These piles are often installed through the existing foundation.

[1]Cast-in-drilled-hole piles 30 in. (760 mm) and larger are covered in the reference, "Standard Specification for the Construction of Drilled Piers (ACI 336.1) and Commentary (336.1R)."

**1.1.7.5** *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.

# CHAPTER 2—DESIGN

## 2.0—Notation

$A$ = pile cross-sectional area, in.$^2$ (mm$^2$)
$A_c$ = area of concrete (including prestressing steel), in.$^2$ (mm$^2$)
= $A_g - A_{st}$, in.$^2$ (mm$^2$) for reinforced concrete piles
$A_{core}$ = 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$^2$)
$A_p$ = area of steel pipe or tube, in.$^2$ (mm$^2$)
$A_{ps}$ = area of prestressing steel, 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$^2$)
$d_{core}$ = diameter of core section, to outside of spiral, in. (mm)
$D$ = steel shell diameter, in. (mm)
$E$ = modulus of elasticity for pile material, lb/in.$^2$ (MPa = N/mm$^2$)
$EI$ = flexural stiffness of the pile, lb-in.$^2$ (N-mm$^2$)
$f_c'$ = specified concrete 28-day compressive strength, lb/in.$^2$ (MPa)
$f_{pc}$ = effective prestress in concrete after losses, lb/in.$^2$ (MPa)
$f_{ps}$ = stress in prestressed reinforcement at nominal strength of member, lb/in.$^2$ (MPa)
$f_{pu}$ = specified tensile strength of prestressing steel, lb/in.$^2$ (MPa)
$f_y$ = yield stress of nonprestressed reinforcement, lb/in.$^2$ (MPa)
$f_{yh}$ = yield stress of transverse spiral or tie reinforcement, lb/in.$^2$ (MPa)
$f_{yp}$ = yield stress of steel pipe or tube, lb/in.$^2$ (MPa)
$f_{ys}$ = yield stress of steel shell, lb/in.$^2$ (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, lb/in.$^2$ (N/mm$^2$)
$K$ = coefficient for determining effective pile length
$\ell_e$ = effective pile length = $K\ell_u$, in. (mm)
$\ell_u$ = unsupported structural pile length, in. (mm)
$L$ = 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)
$n_h$ = coefficient of horizontal subgrade modulus, lb/in.$^3$ (N/mm$^3$)
$P$ = axial 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)
$s_u$ = 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)
$t_{shell}$ = wall thickness of steel shell, in. (mm)
$T$ = relative stiffness factor for normally loaded clay, granular soils, silt and peat, in. (mm)
$\rho_s$ = ratio of volume of spiral reinforcement to total volume of core (out-to-out of spiral)
$\phi$ = strength reduction factor
$\phi_c$ = strength reduction factor in compression
$\phi_t$ = strength reduction factor in pure flexure, flexure combined with tension, or pure tension

## 2.1—General design considerations

Improperly designed pile foundations can perform unsatisfactorily due to: 1) bearing capacity failure of the pile-soil system; 2) excessive settlement due to compression and consolidation of the underlying soil; or 3) structural failure of the pile shaft or its connection to the pile cap. In addition, 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 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 the geotechnical engineer.

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) are usually the control. 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, the reader is referred to general references on soil mechanics and pile design (for example, ASCE 1984; NAVFAC DM 7.2 1982; Peck et al. 1974; Prakash and Sharma 1990; Terzaghi et al. 1996) and bibliographies in such references. Considerations relating to Factors 4 and 5 are covered in Chapter 5, although some guidance on these factors, as well as Factor 6, is offered in this chapter in connection with the preparation of adequate technical specifications.

With reference to Factor 3, specific recommendations are given to ensure 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.

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, the pile must be structurally capable of being driven to its required bearing capacity. This necessitates having one set of structural considerations for driving and another for normal service. Usually, 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; 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 must meet structural engineering requirements under service load conditions, with consideration given to the lateral support conditions provided by the soil. Third, the soil must 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.

**2.1.1** *Subsurface conditions*—Knowledge of subsurface conditions and their effect on the pile-foundation design and installation is essential. This knowledge can be obtained from a variety of 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. Final data on the actual extent of vertical and horizontal subsoil variations at a particular site can be obtained from field observations during production driving. Subsurface information collected by the designer for use in developing the design and monitoring pile installation is frequently insufficient to ensure a successful project.

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 must 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, point-bearing piles, friction piles, or a combination of both types).

**2.1.1.1** *Point-bearing piles*—A pile can be considered point 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 point-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.

**2.1.1.2** *Friction 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 tip within the tolerable limits of settlement of the supported structure.

**2.1.1.3** *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.

**2.1.2** *Bearing capacity of individual piles*—A fundamental design requirement of all pile foundations is that they must carry the design 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. All of these methods should be used in combination with the careful judgment of an engineer qualified in the design and installation of pile foundations. 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 is dependent 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; and
- 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 determined solely by analysis and not proven by static load tests, the design factor of safety should be higher than normally used with piles subjected to static load tests.

**2.1.2.1** *Load testing*—Static pile load tests may be performed in advance of 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, 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, and in particular 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 (Section 2.1.1) 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 2.1.3 through 2.1.6 and Section 2.1.9. 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 to ensure a determination of the pile capacity within the bearing material;
- Instrumenting the pile with strain rods or gages to determine the distribution of load along the pile shaft;
- Testing piles driven both into and just short of a point-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 point-bearing;
- Casting jacks or load cells in the pile tip to determine distribution of pile load capacity between friction and point-bearing; and
- Cyclic loading to estimate soil resistance distribution between friction and point-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, static load tests on individual piles are the most reliable method available, both 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 require 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 are too large when applied to high-capacity piles.

This report does not present detailed recommendations for the various methods for load testing piles, methods, and instrumentation used to measure pile response under load test, or the methods of load test interpretation. ASTM D 1143, D 3689, D 3966, 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% 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.

**2.1.2.2** *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. The earlier attempts were based on energy methods and Newtonian theory of impact (Section 2.1.2.3). The shortcomings of dynamic pile-driving formulas have long been known (Cummings 1940), yet 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. With the advent of the computer, the one-dimensional wave-equation analysis of pile driving has become an indispensable tool for the foundation engineer (Section 2.1.2.4). Field instrumentation that measures and records shaft strain and acceleration near the pile top has become available and has spawned attempts to predict the ultimate bearing capacity using these measurements (Section 2.1.2.5).

Although the development of the wave-equation analysis and methods based on strain and acceleration measurements represents a vast improvement over the fundamentally unsound dynamic formulas, these refined methods are not a reliable substitute 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).

In spite of their short comings, 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 the capacity predictions (Schmertmann and Crapps 1994).

**2.1.2.3** *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 (Sections 2.1.2.4 and 2.1.2.5). Under certain subsoil conditions, penetration resistance as a measure of pile capacity can be misleading in that it does not reflect such soil phenomena as relaxation or freeze (Section 2.4.5), 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 in. (mm)] for a given load or to determine the load capacity based upon 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 successfully used 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.

**2.1.2.4** *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).

Over the past 30 years, 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; Raamot 1967; 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, but research in this area has improved the ability of modern programs to perform analysis for this type of hammer (Davisson and McDonald 1969; Goble and Rausche 1976, 1986; Rempe 1975; Rempe and Davisson 1977).

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 (in. or mm) of the pile. The reciprocal of the set is the driving resistance, usually expressed in hammer blows per in. (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, dimensions, weight, and length; soil quake and damping factors; percentage of pile capacity developed by friction and point bearing; and the distribution of frictional resistance over the pile length. With diesel hammers, the model must deal with the effects of gas force on the hammer output and the steel-on-steel impact that occurs as the ram contacts the anvil.

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 (Section 2.1.1.3). Although wave-equation analysis is based on the fundamentally sound theory of one-dimensional wave propagation, it is still empirical. The primary empirical content are 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, providing proper consideration is given to the possible effect of soil freeze or relaxation (Section 2.4.5). 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 test. 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 (Section 2.1.2.5) 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 driveability 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 driving stress levels. For precast piles, the analysis is most 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 driveability 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).

**2.1.2.5** *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 restruck. Procedures for making the measurements and recording the observations are covered in ASTM D 4945.

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 analysis (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 D 4945; 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 D 4945).

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 must 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 very useful design tool that helps provide information leading to the necessary design decisions (Section 2.1.2.4). 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).

**2.1.2.6** *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 point. Such analyses, insofar as possible, should reflect the effects of pile taper, cross-sectional shape (square, 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). When pile length is selected on the basis of experience or static-resistance analysis, static load tests should be performed to verify such predictions.

**2.1.2.7** *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 report. 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 (Section 2.1.9).

**2.1.3** *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 (Section 2.1.4) 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.

**2.1.4** *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, minimize tip interference;
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 occurs;
I. Where there is a mixture of vertical and batter piles; and
J. Where densification of granular soils can occur.

Special installation methods can be used as an alternative-to increasing pile spacing. For example, predrilling for Cases B, E, and H above, or staggered installation sequence for Case C. 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.

**2.1.5** *Stability*—All piles or pile groups should be stable. For normal-sized piling, stability will be provided by pile groups consisting of at least three piles supporting an isolated column. Wall or strip footings not laterally supported should be carried 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.

**2.1.6** *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.

**2.1.7** *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. Saul (1968), Hrennikoff (1950), and Reese and 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.

**2.1.8** *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 point bearing (soil-resistance distribution). The distribution of load can be approximated by theoretical analysis, special load-test methods, or properly instrumenting 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 point-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 (Sections 2.1.9.1 and 2.2.2.2), 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.

**2.1.9** *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 with the passage of 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 ground water, 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.

**2.1.9.1** *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.

Moreover, if new fill or other superimposed loads are placed around the pile foundation, consolidation of the subsurface soft soils can occur and the positive skin friction over the upper portion of the piles can be reversed completely, causing negative skin friction (or downdrag) and an increase in the total load that will be carried by the piles (Section 2.2.2.2). 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.

Possible long-term settlements due to the consolidation of compressible strata located beneath, or even at 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.

**2.1.9.2** *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 be designed accordingly.

**2.1.9.3** *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.

**2.1.9.4** *Groundwater*—The design should consider the possible effects of groundwater fluctuations on the long-term performance of pile foundations. Lowering at 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 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; this could adversely affect the pile-foundation performance. Under such conditions, take steps 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, take whatever precautions are necessary to maintain a fairly constant moisture content in the soils. If swelling of the soil could occur 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.

**2.1.9.5** *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 must have adequate capacity 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 to resist buckling (Section 2.3.4).

**2.1.10** *Lateral capacity*—Lateral forces on piles will depend on the environment and function of the supported structure, and can be produced by wind, waves, ships, ice action, earth pressures, seismic action, or mechanical causes. Batter piles are frequently used to resist lateral loads (Section 2.1.7).

The ability of vertical piles to resist lateral loads depends upon such things as 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; and the existence and magnitude of axial loads. Group-effect limitations are more severe for laterally loaded piles than for those with axial loads only (Davisson 1970b).

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.

**2.1.11** *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 pickup 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.

## 2.2—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 must be able to resist the service (unfactored) 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 318-95 or other controlling codes should be considered.

**2.2.1** *Temporary loads and stresses*

**2.2.1.1** *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% of the weight of the pile to allow for impact. Pickup and blocking points should be arranged and clearly marked so that all stresses are within the allowable limits and cracking does not occur (Chapters 4 and 5).

**2.2.1.2** *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 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 (Section 2.1.2.4). During construction, dynamic measurements can also provide useful information for evaluating driving stresses (Section 2.1.2.5).

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

**2.2.1.4** *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. The magnitude of the ground motion transmitted to the structure, and thus the loads applied to the foundation, depend on the subsoil conditions, the method of transfer of load from pile to soil (whether friction or point bearing), and the type of construction and the connection between the structure and the foundation. The magnitude of the loads transmitted back to the piles by the structure depends on the extent of the vibrations of the structure and the weight and flexibility of the structure.

The lateral load or base shear at the pile head results from the inertia of the structure at the start of earthquake vibrations and the momentum of the structure as it is moved laterally. The actual value of the base shear is a function of the magnitude of the earthquake, the degree of seismicity of the geographical area, and the fundamental period of the structure at a reasonable, actual in-service mass of the structure. To help distribute the base shear in a building, individual pile caps are often interconnected with reinforced concrete struts capable of withstanding the horizontal force resulting from an earthquake, both in compression and tension.

During an earthquake, uplift and compression loads can be exerted on the pile foundation as the structure tends to overturn. Batter piles supporting bulkheads of wharves have suffered great distress because they tend to resist all of the horizontal force in the structure, leading to failure of either the pile or the pile cap supported by the pile. Longer, more flexible batter piles have performed better. Other pile failures have occurred because of poor connection details between the piles and the cap, lack of adequate strength and rotational ductility in the pile section, and because of faulty analyses.

Design and detailing of piles to resist seismic forces and motions are discussed in Section 2.3.6.

**2.2.2** *Permanent loads and stresses*

**2.2.2.1** *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 (Section 2.1.8).

Some tension forces can be fairly 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 ensure that the most critical pile forces in both tension and compression are identified.

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. For evaluating bending and shear stresses in piles due to horizontal loads or 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, 1964b, 1965; Davisson 1970b; NAVFAC DM-7.2 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).

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 Section 9.5 of ACI 318-95 (effective moment of inertia) or Sections 10.11 to 10.13 (approximate evaluation of slenderness effects), unless a more refined analysis is used.

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). 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 define accurately 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 influence of a nonlinear soil resistance-deflection relationship can also be determined using nondimensional solutions in an iterative procedure (Prakash and Sharma 1990).

In some structures, second-order deflection ($P$-$\Delta$) effects can become important. In such cases, the foundations must be designed to resist the increased forces associated with these effects.

**2.2.2.2** *Negative skin friction*—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 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). Negative skin friction loads should be considered when evaluating both the soil bearing capacity and the pile shaft strength requirements. If it is necessary to use batter piles under conditions where negative skin friction can develop, the designer must consider both the additional axial negative skin friction loads and the additional bending loads from the weight of the settling soil and drag forces on the pile sides.

## 2.3—Structural strength design and allowable service capacities

**2.3.1** *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 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 in some cases.

Historically, the structural 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* (1994) and the *BOCA National Building Code* (1993) limit the allowable concrete compressive stress for CIP concrete piles to $0.33 f_c'$ and provide provisions for the allowable stress to be increased by concrete confinement (up to a maximum value of $0.40 f_c'$) provided required conditions are met. Similarly, both of these codes limit the allowable compressive stress on prestressed concrete piles to $(0.33 f_c' - 0.27 f_{pc})$. These allowable unit stresses were first published around 1970 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 Section 2.2. Concrete piles must have adequate structural capacity for all modes and combinations of loading that they will experience. For combined flexure and thrust loadings, the structural adequacy can be evaluated more readily through the use of moment-thrust interaction diagrams and strength-design methods.

This section recommends provisions for ensuring that concrete piles have adequate structural capacity based on strength-design methods. Recommendations are provided in Sections 2.3.2, 2.3.4, and 2.3.5 for the direct use of strength-design methods. Because of the historical use of allowable capacities and stresses in piling design, however, recommendations are also provided for allowable axial service capacities for concentrically loaded, laterally supported piles. The allowable service capacities $P_a$ recommended in Section 2.3.3 are intended specifically for cases in which the soil provides full lateral support to the pile and where the applied forces cause no more than minor bending moments resulting from accidental eccentricities. Piles subjected to larger bending moments or with unsupported lengths must be treated as columns in accordance with ACI 318-95 and the provisions given in Sections 2.3.2, 2.3.4, and 2.3.5 of this report.

Foundation piles behave similar 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

**Table 2.1—Recommended compressive strength reduction factors $\phi_c$**

| Pile type | Compressive strength reduction factor $\phi_c$ |
|---|---|
| Concrete-filled shell, no confinement | 0.65 |
| Concrete-filled shell, confinement* | 0.70 |
| Uncased, plain or refinforced concrete† | 0.60 |
| Precast reinforced concrete or cast-in-place reinforced concrete within shell | 0.70 |
| Pretensioned, prestressed reinforced concrete | 0.70 |
| Concrete-filled steel pipe | 0.75 |

*Shell of 14 gage minimum thickness (0.07474 in. [1.9 mm]), shell diameter not over 16 in. (400 mm), for a shell yield stress $f_{ys}$ of 30,000 lb/in.$^2$ (210 MPa) minimum, $f_c'$ not over 5000 lb/in.$^2$ (35 MPa), noncorrosive environment, and shell is not designed to resist any portion of axial load. The increase in concrete strength due to confinement shall not exceed 54%.

†Auger-grout piles, where concreting takes place through the stem of a hollow-stem auger as it is withdrawn from the soil, are not internally inspectable. The strength reduction factor of 0.6 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 has to 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.

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 (Section 2.2.1.2) 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 lb/in.$^2$ (35 MPa) and that greater strengths are 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.

**2.3.2** *Strength design methods*—The provisions for strength design of concrete piles given herein were developed using strength design principles from ACI 318-95, although no attempt has been made to completely follow the column design requirements of ACI 318-95.

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 determined using the loading factors and combinations of service loading as stipulated in ACI 318-95 Section 9.2. The *design strength* of the pile is computed by multiplying the *nominal strength* of the pile by a *strength reduction factor* $\phi$, which is less than 1. The nominal strength of the member should be determined in accordance with the recommendations of ACI 318-95.

The strength reduction factors $\phi$ recommended herein for various types of loading conditions generally follow ACI 318-95, except that strength reduction factors for compression $\phi_c$ have been determined by the committee for the pile member types not covered by ACI 318-95. Recommended strength reduction factors for various forms of loading, as well as additional recommendations, are provided in Sections 2.3.2.1 through 2.3.2.7. Further recommendations for the use of the strength design method with piling are provided in Sections 2.3.4 and 2.3.5.

**2.3.2.1** *Compressive strength*—The recommended compressive strength reduction factors $\phi_c$ for various types of concrete piles are presented in Table 2.1. These reduction factors have been determined 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 shall be designed for the eccentricity corresponding to the maximum moment that can accompany the loading condition, but not less than an eccentricity of 5% 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 dependence of the member strength on the strength of the concrete actually attained in the member and is set at 0.60 for uncased piles. In some soil types, local experience may indicate that lower values of $\phi$ are prudent. Davisson et al. (1983) provide an extensive discussion of these design factors.

**2.3.2.2** *Flexural strength*—For concrete piles subjected to flexure without axial load or flexure combined with axial tension, the strength reduction factor $\phi_t$ is 0.9. This value corresponds to the ACI 318-95 strength reduction factor for these particular loading conditions. For piles subjected to flexure combined with axial compression, the recommended compressive strength reduction factor $\phi_c$ given in Table 2.1 should be used accordingly.

For reinforced concrete piles, prestressed concrete piles, or concrete-filled pipe piles subjected to flexure and low values of axial compression, the $\phi$ can be increased from the recommended compression value $\phi_c$ to the value of 0.9 for flexure without axial load $\phi_t$ in accordance with the procedures given in Section 9.3.2 of ACI 318-95.

**2.3.2.3** *Tensile strength*—Concrete piles subjected to axial tension (uplift) loads should be designed for the full tension load to be resisted by the reinforcing steel (Section 2.5). The strength reduction factor $\phi_t$ value used for this loading condition should be 0.9.

**2.3.2.4** *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 Chapter 10 of ACI 318-95. The $\phi$ in Sections 2.3.2.1 and 2.3.2.2 of this report and the loading factors and combinations in accordance with Chapter 9 of ACI 318-95 should be used. Under no circumstances should the axial compression capacity exceed the capacity corresponding to an eccentricity of 5% of the diameter or width of the pile.

### Table 2.2—Allowable service capacity for piles with negligible bending*

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

*Based on an eccentricity of 5% of pile diameter or width, and an assumed average load factor of 1.55. In cases of very high live or other loadings such that the average load factor exceeds 1.55, the allowable capacity equations should be reduced accordingly.

†Shell of 14 gage minimum thickness (0.0747 in. [1.9 mm]), shell diameter not over 16 in. (400 mm), for a shield yield stress $f_{ys}$ of 30,000 lb/in.2 (210 MPa) minimum, $f_c'$ not over 5000 lb/in.$^2$ (35 MPa) noncorrosive environment, and shell is not designed to resist any portion of axial load. The allowable load $P_a$ shall not exceed $0.40 f_c' A_c$.

‡Auger-grout piles, where concreting takes place through the stem of a hollow-stem auger as it is withdrawn from the soil, are not internally inspectable. 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 has to 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% 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 lb/in.$^2$ (35 MPa), or for piles with axial reinforcement areas (as a percentage of the gross pile area) greater than 3% for round piles or greater than 4.5% for square piles, the eccentricity factor should be evaluated from a nominal strength moment-thrust interaction diagram and the allowable capacity equation adjusted accordingly.

Many of the design aids for reinforced concrete columns (CRSI 1996; ACI 1990) 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 $\phi$. Fully understanding any assumptions made in the preparation of the design aids, especially the inclusion or exclusion of the $\phi$, is imperative. PCI (1992, 1993) 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).

The assumptions made for the analysis of concrete-filled pipe are worthy of noting. For the analysis of concrete-filled pipe under combined bending and compression, 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 must 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 anchorage are required in this scenario.

For the case in which a concrete-filled shell is counted on for confinement, the shell is effective in increasing the concentric compression capacity but adds nothing to the bending capacity, which significantly increases the sensitivity of the member to eccentricity of load. If it is necessary to construct the moment-thrust interaction diagram to address eccentricities for concrete-filled shell piles with confinement, constructing the interaction diagram by the procedures in Davisson et al. (1983) is recommended.

**2.3.2.5** *Shear strength*—Piles that have significant bending moments will often have significant shear forces. Provisions in Chapter 11 of ACI 318-95 should be followed when designing shear reinforcement. Special attention is required when piles have both significant tension and significant shear forces. A strength reduction factor of 0.85 should be used for shear in reinforced concrete piles, prestressed concrete piles, and pipe piles. For nonreinforced piles, the strength reduction factor for shear used should be 0.65.

**2.3.2.6** *Development of reinforcement*—Development of stress in embedded reinforcement (bond) should correspond to the information given in Chapter 12 of ACI 318-95.

**2.3.2.7** *Prestressed piles*—Prestressed piles designed by strength-design methods also require serviceability checks to ensure 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 Section 2.3.3.3 of this report.

**2.3.3** *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 Section 2.3.2.1, 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% eccentricity to the nominal axial strength under concentric loading. The allowable axial service capacity equations in Table 2.2 are based on eccentricity factors taken from a Federal Highway Administration report (Davisson et al. 1983) and a PCA report (PCA 1971) in which the general shapes of moment-axial force interaction diagrams for various types of piles were studied in detail.

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 2.2 assume a combined average load factor of 1.55, based on an average of the ACI 318-95 load factors for dead and live load (assuming the dead load is equal to live load), which is generally a

### Table 2.3—Allowable service-load stresses in prestressed piles*

| Loading condition | Permanent, lb/in.$^2$ | Temporary, lb/in.$^2$ |
|---|---|---|
| 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 | $0.45 f'_c$ | $0.6 f'_c$ |
| Flexure plus tension† | | |
| Concrete tension | 0 | $3\sqrt{f'_c}$ |
| Concrete compression | $0.45 f'_c$ | $0.6 f'_c$ |

*Units for allowable stresses and $f'_c$ in the equations in this table are lb/in.$^2$ (1 lb/in.$^2$ = 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:

| Equation in terms of lb/in.$^2$ | Equation in terms of MPa |
|---|---|
| $3\sqrt{f'_c}$ | $(\sqrt{f'_c})/4$ |
| $6\sqrt{f'_c}$ | $(\sqrt{f'_c})/2$ |

†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, $f_{ps}A_{ps} \geq 1.2\,(f_{pc} + 7.5\sqrt{f'_c})A_c$, $f_{pc}$, and $f_{ps}$ are in lb/in.$^2$ units.

conservative assumption. If the controlling loading case is dominated by very high live or other loadings, such that the actual average load factor exceeds 1.55, the allowable capacity equations indicated herein 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). Piles subjected to larger bending moments or with unsupported lengths must be treated as columns in accordance with ACI 318-95 and the provisions in Sections 2.3.2, 2.3.4, and 2.3.5 of this report.

**2.3.3.1** *Concentric compression*—The allowable axial compressive service capacity for laterally supported solid concrete piles can be determined by the equations given in Table 2.2. These equations were developed based on the procedures described in Section 2.3.3 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 must be analyzed and designed using a moment-axial force interaction design method, with a minimum eccentricity of 5% of the pile diameter or width, as described in Section 2.3.2.

**2.3.3.2** *Concentric tension*—Concrete piles subjected to axial tension (uplift) loads are designed for the full tension load to be resisted by the steel (Section 2.5). The allowable tension service capacity for reinforcing steel is

$$P_{at} = 0.5 f_y A_{st} \tag{2.1}$$

### Table 2.4—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% 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 upon complete fixity and should be adjusted depending on the actual degree of fixity (Davisson 1970b; ACI 318-95; Joen and Park 1990, PCI 1993.)

For prestressed concrete piles where the full tension load is to be resisted at the pile head by unstressed strands extended into a footing or cap, the allowable tension service capacity is

$$P_{at} = 0.1 f_{pu} A_{ps} \tag{2.2}$$

**2.3.3.3** *Special considerations for prestressed piles*—Prestressed piles must have serviceability checks applied to ensure that their service-load behavior is adequate, in addition to the limiting capacities described in Section 2.3.2. The allowable service-load stress limits given in Table 2.3 should be determined using concrete compressive strengths $f'_c$ corresponding to the age of the concrete under consideration.

**2.3.4** *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 designed to resist buckling under the imposed loads (Section 2.1.6). The effects of length on the strength of piles should be taken into account in accordance with Sections 10.10 and 10.11 to 10.13 of ACI 318-95. Whereas Sections 10.11 to 10.13 give an approximate method suitable for $K\ell_u/r$ < 100, Section 10.10 describes the requirements for a rational analysis of the effects of length.

The effective pile length $\ell_e$ is determined by multiplying the unsupported structural pile length $\ell_u$ by the appropriate value of the coefficient *K* from Table 2.4 or from Chapter 10 of ACI 318-95. For cases in which the top of the pile is free to translate, the coefficient *K* requires careful consideration and should exceed 1.0.

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. Furthermore, for many structures using unsupported pile lengths, 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 proves otherwise.

If the structural length $\ell_u$ of an unsupported concrete pile is not confined in a steel pipe or shell with a minimum wall thickness of 0.1 in. (2.5 mm) or spirally reinforced, the capacity determine on the basis of strength design should be reduced by 15%.

The structural length $\ell_u$ as defined here 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 $\ell_u$ would be equal to the length of pile above the ground surface $L_u$ plus the depth $L_s$.

$$\ell_u = L_u + L_s \tag{2.3}$$

The depth below the ground surface to the point of fixity $L_s$ can be estimated by Eq. (2.4) for preloaded clays, or by Eq. (2.5) for normally loaded clay, granular soils, silt, and peat.

$$L_s = 1.4R \quad \text{where } R = \sqrt[4]{\frac{EI}{k}} \tag{2.4}$$

$$L_s = 1.8T \quad \text{where } T = \sqrt[5]{\frac{EI}{n_h}} \tag{2.5}$$

The total length of the portion of the pile embedded in the soil must 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 must be greater than $2R$ (that is, $L_u > 2R$) or $T$ (that is, $L_u > T$) for Eq. (2.4) and (2.5) 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. (2.4) and (2.5), 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 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 2.5. These values also apply to submerged soils.

**2.3.5** *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 (Section 2.3.2) or the allowable service capacities determined in Section 2.3.3 should be reduced by 10 %. The capacity is reduced further by a reduction factor depending on both the $\ell_u/r$ ratio and the head and end conditions (Section 2.3.4). For unsupported piles not spirally reinforced, a further 15% reduction in capacity is recommended (Section 2.3.4).

**2.3.6** *Seismic design of piles*—In areas of seismic risk, designing piles or other structural members on the basis of strength alone is not adequate. These members must also possess adequate ductility, and more importantly, ductility under fully reversed moment conditions. Ductility can be defined in various ways, but it 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 is a measure of toughness.

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 at the interfaces between soil layers with significantly differing stiffnesses. An adequate description of analysis methods suitable for the computation of these concentrated rotations is beyond the scope of this report, but it is important that soil-structure interaction be properly accounted for in such an analysis. Failure to include soil-structure interaction in such an analysis can lead to unrealistically large curvature and rotation requirements for the piles.

Most reinforced and prestressed concrete structural members have some inherent ductility, but this is often inadequate for seismic response and analysis purposes unless special measures are taken to enhance it. Ductility is a function of many factors. It 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 confinement reinforcement is the spiral required in spirally reinforced concrete columns according to Eq. (10-6) of ACI 318-95, often referred to as an ACI Spiral. Experience from past earthquakes and from laboratory tests demonstrates that this spiral provides significant ductility in flexural modes, and that it also provides a major shear-strength contribution. Although this spiral leads to ductile members, the selection of the spiral ratio and bar area and spacing is unrelated to flexural or shear requirements but rather is related to axial compression considerations. Major improvements in ductility can be obtained with lighter spirals than the ACI Spiral. Because the requirement was explicitly derived for circular spirals, it does not address the requirements for square or rectangular longitudinal reinforcement arrangements. Other more empirical expressions have been developed for these cases.

This report does not recommend the use of the ACI Spiral in foundation piles for purposes of achieving flexural ductility, but the requirements are repeated here to provide a basis

### Table 2.5—Values of $n_h$

| Soil type | $n_h$, lb/in.$^3$ | 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).

of comparison with recommendations that follow. Eq. 10-6 of ACI 318-95 is expressed, with slightly modified notation, in Eq (2.6).

$$\rho_s = 0.45\frac{f'_c}{f_{yh}}\left(\frac{A_g}{A_{core}} - 1\right) \tag{2.6}$$

where

$f'_c$ = compressive strength of concrete;

$f_{yh}$ = yield stress of spiral reinforcement;

$A_g$ = gross area of member cross section; and

$A_{core}$ = area of core of section, to outside diameter of the spiral.

The spiral steel ratio $\rho_s$ is a volume ratio relating the volume of steel in the spiral to the volume of concrete contained within the spiral

$$\rho_s = \frac{4\,A_{sp}}{d_{core}\,s_{sp}} \tag{2.7}$$

where

$A_{sp}$ = area of wire or bar used in spiral;

$d_{core}$ = diameter of core of section to outside diameter of spiral; and

$s_{sp}$ = spacing or pitch of spiral along length of member.

Although an ACI Spiral provides excellent ductility, it is extremely difficult to provide the resulting amount of spiral reinforcement in many practical cases, such as square piles with longitudinal reinforcement arranged in a circular pattern. This difficulty arises because the area ratio $A_g/A_{core}$ is unfavorable for square members containing round spirals and becomes especially unfavorable for small members. High concrete strengths also lead to large steel $\rho_s$ requirements. It is not desirable to have the pitch too small because it makes concrete placement very difficult 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.

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 Chapter 21 of ACI 318-95 with a lower limit to ACI 318-95 Eq. (10-6) of

$$\text{Minimum } \rho_s = 0.12\frac{f'_c}{f_{yh}} \tag{2.8}$$

The minimum $\rho_s$ requirement of Eq (2.8) governs when the ratio of $A_g/A_{core}$ becomes less than approximately 1.27, which occurs only in large columns.

Although the ACI Spiral is widely adopted for column design, its adoption for piling is less universal. For example, the Uniform Building Code (1994) adopts the ACI Spiral but limits the spiral steel ratio so that it need not be larger than $\rho_s = 0.12 f'_c/f_{yh}$ for nonprestressed concrete piling in zones of high seismic risk.

The PCI Committee on Prestressed Concrete Piling (1993) recommends minimum spiral steel ratios for members with round steel patterns and minimum steel areas for members with square steel arrangements for regions of high seismic risk. These recommendations are repeated herein and are endorsed by ACI Committee 543 for application to both prestressed and reinforced concrete piling in regions where seismic resistance is required. The terms used herein to describe seismic risk (low, moderate, and high) are used in the same context as these terms are used in Chapter 21 of ACI 318-95.

**2.3.6.1** *Regions of low to moderate seismic risk*—In regions of low to moderate seismic risk, lateral reinforcement should meet the following steel ratio

$$\rho_s = 0.12\frac{f'_c}{f_{yh}} \geq 0.007 \tag{2.9}$$

with two limits on materials

$$f'_c \leq 6000 \text{ lb/in.}^2 \text{ (40 MPa); and}$$

$$f_{yh} \leq 85{,}000 \text{ lb/in.}^2 \text{ (585 MPa).}$$

**2.3.6.2** *Regions of high seismic risk*—In regions of high seismic risk, the following minimum amounts of confinement reinforcement are recommended:

- *Reinforcement of circular ties or spiral*

$$\rho_s = 0.25\frac{f'_c}{f_{yh}}\left(\frac{A_g}{A_{core}} - 1\right)\left(0.5 + 1.4\frac{P_u}{A_g f'_c}\right) \tag{2.10}$$

but not less than

$$\rho_s = 0.12\frac{f'_c}{f_{yh}}\left(0.5 + 1.4\frac{P_u}{A_g f'_c}\right) \tag{2.11}$$

where

$P_u$ = factored axial load on pile;

and with two limits on materials

$$f'_c \leq 6000 \text{ lb/in.}^2 \text{ (40 MPa); and}$$

$$f_{yh} \leq 85{,}000 \text{ lb/in.}^2 \text{ (585 MPa).}$$

- *Reinforcement of square spiral or ties*

$$A_{sp} = 0.3\,s_{sp}\,h_c\frac{f'_c}{f_{yh}}\left(\frac{A_g}{A_{core}} - 1\right)\left(0.5 + 1.4\frac{P_u}{A_g f'_c}\right) \tag{2.12}$$

but not less than

$$A_{sp} = 0.12 s_{sp} h_c \frac{f'_c}{f_{yh}}\left(0.5 + 1.4\frac{P_u}{A_g f'_c}\right) \qquad (2.13)$$

where

$h_c$ = 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 lb/in.$^2$ (480 MPa).

The formats of the equations for high seismic risk regions, but not the numerical constants, follow research conducted in New Zealand (Joen and Park 1990) and the New Zealand Standard Code of Practice for the Design of Concrete Structures (1982).

**2.3.6.3** *Needed research*—Most of the reversed bending tests of piles have been conducted on octagonal pretensioned members (Banerjee et al. 1987). Other tests, including 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. 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.

**2.3.6.4** *Vertical accelerations*—Experience from the 1994 Northridge earthquake in California reveals that at and 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. 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 an earthquake.

## 2.4—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 5). Others relate to the subsoil conditions or the qualifications of the pile contractor. Obviously, the engineer cannot allow for all contingencies in his 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.

**2.4.1** *Pile-head location tolerances*—Some tolerance should be allowed between the as-installed position of the pile head and the required plan 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% 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.

Sometimes piles driven off 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 (Section 5.3.5).

**2.4.2** *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 for embedded piles driven through sandy soils or soft clays;
- Four percent for embedded piles driven through difficult soils of nonuniform consistency, boulder-ridden soils, or batter piles driven into gravel; and
- A maximum of 2% of the total pile length in marine structures that have over half the pile length in water rather than soil.

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% 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.

**2.4.3** *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 (for example, 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 (not 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 ensure durability (Chapter 3).

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 ensure that their corrosion resistance is adequate.

**2.4.4** *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 manufacture, transport, and handling inconvenient field splices will be part of the original design. Some piles have standard stock lengths and splicing is a part of their normal manufacture and usage (sectional precast piles). These 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. Aboveground 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 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, 1974b), Gamble and Bruce (1990), and Venuti (1980) report on the behavior of several different splices, and also 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 must 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 member; and
- 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.

**2.4.5** *Relaxation and soil freeze*—If soil relaxation or freeze can occur, 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 (Sections 2.1.2.3, 2.1.2.4, and 2.1.2.5) will not produce valid results without modifications based on a load test or redriving results. Relaxation is evidenced by a reduction in the final penetration resistance after initial driving and could be accompanied by a loss of bearing capacity. Soil freeze has the opposite effect on pile capacity and is associated with regain of strength of soils after being disturbed during the driving process with a corresponding increase in the bearing capacity.

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 redriving some piles several hours after initial final driving to determine if the driving resistance has been maintained. Soil freeze can also be checked by redriving, 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. Retapping of piles produces more valid information if the hammer-cushion-pile system is the same as for initial driving.

**2.4.6** *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 (Sections 5.1.6 and 5.1.7).

**2.4.7** *Liquefaction*—Liquefaction is usually associated with earthquake or large vibratory forces combined with liquefiable granular soils. This can result in loss of pile capacity. 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 that causes vertical ground movements will cause downdrag and possible settlement of friction piles. Piles in slopes can be subjected to large lateral loads and displacements due to liquefaction. If this phenomenon must be provided for, the pile-soil capacity should be developed below the zone of possible soil liquefaction. Liquefaction generally does not occur below a depth of 30 ft (9 m) and, at most, 50 to 60 ft (15 to 18 m). Further, it is not likely to occur within a pile group because of the soil compaction resulting from the pile driving. It can, however, occur around the perimeter of a pile group; therefore, under these conditions, the stability of the group should be evaluated. Methods of determining whether soils at a particular site can experience liquefaction (Kriznitsky et al. 1993; Ohsaki 1966; Poulos et al. 1985; Seed et al. 1983) should be used whenever there is significant seismic activity.

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

**2.4.8** *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 it 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 stretch. For laterally corrugated pile shells, 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. 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 redriving 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.

**2.4.9** *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 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) (Davission 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 in a manner 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.

**2.4.10** *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 (Sections 4.2.5 and 5.2.1.5).

## 2.5—Other design and specification considerations

The pile-foundation design should include other considerations that may relate to specific type piles or that may have to be covered in the plans and specifications to ensure that piles are installed in accordance with the overall design. Some of these considerations are closely tied to items discussed in Chapters 4 and 5.

**2.5.1** *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. If construction or inspection personnel must enter the shaft, however, the diameter should be at least 30 in. (760 mm).

**2.5.2** *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 as 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% of the gross pile section.

**2.5.3** *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 (Sections 2.2 and 2.3). 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.

**2.5.3.1** *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% 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% 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% 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% 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% nor more than 8% of the gross cross-sectional area of the pile. If after a thorough analysis of the handling, driving, and service-load conditions, the designer selects to use less than 1.5% (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 wire (ACI 318-95 Appendix E) and spaced not more than 6 in. (150 mm) on centers. The spacing should be closer at each end of the pile.

**2.5.3.2** *Reinforcement for precast prestressed piles*—Within the context of this report, longitudinal prestressing is not considered as 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 (Section 2.5.3.3). 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 (ACI 318-95 Appendix E), 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.

**2.5.3.3** *Effective prestress*—For prestressed concrete piles, the effective prestress after all losses should not be less than 700 lb/in.$^2$ (4.8 MPa). Significantly higher effective prestress values are commonly used and may be necessary to control driving stresses in some situations (see Item J in Section 5.2.2 for additional comments on the use of higher effective prestress values).

**2.5.3.4** *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. Reinforcing will also be required for lateral loading, except for very small lateral loads under conditions where the presence of concurrent axial compression loads can be ensured.

Unsupported sections should be designed in accordance with Section 2.3. 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

### Table 2.6—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 and smaller | 1.25 (35) |
| Precast-reinforced piles—marine exposure† | 2.0 (50) |
| Precast-reinforced piles—normal exposure‡ | 1.5 (40) |
| Precast-reinforced 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.
†For marine exposures, consider the following section from the Commentary to ACI 318-95 when selecting concrete materials and cover values:
"**R7.7.7—Corrosive Environments—**When concrete will be exposed to external sources of chlorides in service, such as deicing salts, brackish water, seawater, or spray from these sources, concrete must be proportioned to satisfy the special exposure requirements of Chapter 4. These include minimum air content, maximum water-cementitious materials ratio, minimum strength for normal weight and lightweight concrete, maximum chloride ion in concrete, and cement type. Additionally, for corrosion protection, a minimum concrete cover for reinforcement of 2 in. (50 mm) for walls and 2.5 in. (65 mm) for other members is recommended. For precast concrete manufactured under plant control conditions, a minimum cover of 1.5 and 2 in. (40 and 50 mm), respectively, is recommended."
‡For prestressed piles under exposure, the required cover could range from 2 to 3 in. (50 to 70 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.

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, it is recommended that not less than four longitudinal bars 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.

**2.5.3.5** *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 ensure 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. Holes through the web and flanges of the stub (vent holes) may be required to permit the escape of air and water, and thereby help ensure proper concrete placement (Sections 4.5.3.1, 5.6.2, and 5.6.3).

**2.5.3.6** *Cover for reinforcement*—The minimum recommended clear cover for any pile reinforcement is summarized in Table 2.6 for various pile types and exposure conditions.

**2.5.4** *Concrete for CIP and CIS concrete piles*—The designer should give consideration to the factors affecting concrete placement in CIP and CIS piles when preparing specifications for this kind of work. This includes such things 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 (Sections 3.1, 3.5, and 5.5).

**2.5.5** *Rock sockets for drilled-in-piles*—The design of drilled-in-piles 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 end bearing and bond to develop the required load. The combined use of end bearing and bond, however, may not be permitted by the applicable building code.

## CHAPTER 3—MATERIALS

### 3.1—Concrete

**3.1.1** *Cement*—Portland cement should conform to either ASTM C 150 (Types I, II, III, or V) or ASTM C 595 (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 Chapter 4 of ACI 318-95.

The principal consideration in the selection of cement type for sulfate resistance in ACI 318-95 is the tricalcium aluminate ($C_3A$) content. For example, concrete piles with moderate exposure to sulfate-containing soils or water (soils containing 0.1 to 0.2% 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% tricalcium aluminate, such as ASTM C 150 Type II cement or moderate sulfate-resistant blended cement (MS). Similarly, for severe sulfate exposure, use ASTM C 150 Type V cement, which contains not more than 5% tricalcium aluminate, and for very severe sulfate exposure, use ASTM C 150 Type V cement with a fly ash admixture.

Type V cement is not generally available in most sections of the country. In areas where Type V cement is not available, a comparable substitution needs to be specified (for example, Type II with tricalcium aluminate less than 8% with Type F fly ash at approximately 20% by weight, see Section 3.1.4.3).

Concrete in seawater environments, with portland cement containing 5 to 8% tricalcium aluminate, has been reported to show less cracking due to steel corrosion than cement with less than 5% tricalcium aluminate (ACI 201.2R). 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 318-95 recommendations that classify seawater as moderate sulfate exposure, even though it generally contains sulfate in excess of the moderate exposure limits.

In addition to the proper selection of the cement type, consideration of other requirements, such as water-cementitious materials ratio, strength, air entrainment, adequate consolidation, adequate cover of reinforcement, and curing are essential to producing a durable concrete structure. Additional information on concrete durability is given in ACI 201.2R.

**3.1.2** *Aggregates*—Concrete aggregates should conform to ASTM C 33. Aggregates that fail to meet this specification 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, the use of reactive aggregate in concrete piles should be avoided. The potential for an adverse reaction between the alkali of the cement and the silica in the aggregates should be evaluated (ACI 201.2R). Additional information on aggregates is given in ACI 221R and ACI 201.2R.

**3.1.3** *Water*—Water used for curing, washing aggregates, and mixing concrete for concrete piles should conform to the requirements in Chapter 3 of ACI 318-95.

**3.1.4** *Admixtures*—Specific information on admixtures is given in ACI 201.2R, ACI 212.3R, and ACI 212.4R.

**3.1.4.1**—Concrete for piles that will be exposed to freezing and thawing in moist conditions 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 C 260. 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 will reduce its effectiveness in preventing freezing-thawing damage. ACI 201.2R recommends that the entrained air content of fresh concrete be in the range of 3 to 7%, depending on the size of coarse aggregate and on the severity of exposure.

**3.1.4.2**—Water-reducing admixtures, retarding admixtures, accelerating admixtures, water-reducing and retarding admixtures, and water-reducing and accelerating admixtures should conform to ASTM C 494 or ASTM C 1017.

**3.1.4.3**—If fly ash or other pozzolans are used as admixtures, the amount recommended by ACI 211.4R can be used. Because the fly ash content affects the rate of strength development, however, practical considerations may limit the amount of fly ash used for precast-pile applications to less than permitted by ACI 211.4R. 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 C 618.

**3.1.4.4**—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 chloride that may be present in the concrete would exceed the recommended limits given in ACI 201.2R.

**3.1.4.5**—Calcium chloride should not be used as an admixture in concrete that will be exposed to severe sulfate-containing solutions as defined in Chapter 4 of ACI 318-95, and should never be used with prestressed concrete.

**3.1.5** *Water-cementitious materials proportions*—

**3.1.5.1**—The water-cementitious materials ratio for a concrete mixture can be a reliable predictor of the strength and durability of the mixture. Guidelines for selecting appropriate water-cementitious materials ratios are given in ACI 211.1 and ACI 301. Limitations on the water-cementitious materials ratio for durability requirements are addressed in Chapter 4 of ACI 318-95.

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

**3.1.5.2**—The amount of cement in a mixture is an important variable. In general, the cement content of a concrete pile mixture should be a minimum of 564 lb/yd$^3$ (335 kg/m$^3$) to ensure durability. In aggressive environments, such as for marine usage, at least 658 lb/yd$^3$ (390 kg/m$^3$) is recommended. For conventional structural concrete, 752 lb/yd$^3$ (445 kg/m$^3$) is considered a reasonable maximum. Reduced-coarse-aggregate concrete mixtures, containing approximately 800 lb of coarse aggregate/yd$^3$ (475 kg/m$^3$) and with as much as 846 lb of cement/yd$^3$ (500 kg/m$^3$), 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 (Section 5.5.4).

The proportions of a concrete mixture 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$^3$ of cement [335 to 445 kg/m$^3$], 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% or more sand) are used for tremie placement. A plasticizing admixture can also be beneficial.

**3.1.5.3**—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 a plastic mixture and the desired workability for the most effective placement of the concrete. The durability of the finished product decreases with an increasing water-cementitious materials ratio.

**3.1.6** *Quality control*

**3.1.6.1**—Slump tests made in accordance with ASTM C 143, penetration tests made in accordance with ASTM C 360, or flow tests made in accordance with ASTM C 939 are measures of the workability of concrete mixtures. The test results of slump tests are loosely related to the total water content of the mixture. A slump test taken after the addition of a high-range water reducer (superplasticizer) no longer reflects the water content of a batch of concrete. The slump of a concrete mixture proportion 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 (see Section 3.5 for comments on monitoring fluidity of grout mixtures for auger-grout piles).

**3.1.6.2**—The presence of entrained air should be verified during placement when strength samples are obtained or when the possibility of an inappropriate air content is suspected. Entrained-air tests should be made in accordance with ASTM C 173 or ASTM C 231, as applicable. Indicators, such as the Chase meter, should be frequently calibrated for a given mixture for a specific project.

**3.1.6.3**—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 C 172 and ASTM C 31 and tested in accordance with ASTM C 39. 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 cylinders 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 tests to be done 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 auger-grout piles, strength tests of the grout are usually made on 2 in. (50 mm) cubes, in accordance with ASTM C 109. A set of six to eight cubes is typically made, with two (or three) cubes tested at 7 days, two (or three) cubes at 28 days, and the remaining cubes held in reserve for testing at a later date if required. The failure stresses for tests on cube specimens are approximately 15% higher than for tests on cylinder specimens used for determining $f'_c$.

## 3.2—Reinforcement and prestressing materials

**3.2.1** *Reinforcement*—Reinforcement steel should conform to the latest revision of ASTM A 82, ASTM A 185, ASTM A 496, ASTM A 497, ASTM A 615, ASTM A 616, ASTM A 617, ASTM A 706, or ASTM A 955, as appropriate.

**3.2.2** *Prestressing strand*—Strand used for prestressing should conform to ASTM A 416, Grade 250, or Grade 270.

**3.2.3** *Prestressing wire*—Wire used for prestressing should conform to ASTM A 421.

**3.2.4** *Prestressing bars*—High-strength steel bars used for prestressing should conform to ASTM A 722.

**3.2.5** *Epoxy-coated reinforcement*—Epoxy-coated steel has been used as lateral reinforcement (spiral or ties) in concrete piles. The committee is aware of only limited use of epoxy-coated longitudinal reinforcing bars or prestressing strand in concrete piles. In the limited instances reported to the committee, 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. The committee has not received any 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). Higher-quality concrete, with lower water-cementitious materials ratio 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. Special admixtures such as silica fume (ASTM C 1240) 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 A 775, ASTM A 882, or ASTM A 884, as applicable.

## 3.3—Steel casing

**3.3.1** *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 A 252, ASTM A 283, or ASTM A 570.

The suitability of the intended materials for welding should be predetermined. The ASTM A 252 specification does not strictly imply weldability. Other steel specifications can be used, provided that the yield, elongation, and other items are satisfactory.

**3.3.2** *Nonload-bearing casing*—Steel casing not intended for permanent load bearing in composite action with CIP concrete should meet the requirements of ASTM A 366, ASTM A 569, or ASTM A 570.

## 3.4—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 A 36, ASTM A 242, or ASTM A 572.

The thickness of steel in any part of the structural steel core shall not be less than 3/8 in. (10 mm).

## 3.5—Grout

Grout used for auger-injected piles, preplaced-aggregate piles, and drilled piles should consist of a mixture of approved cement, fine aggregate, admixtures, and water. The grout should be mixed so as 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. Under certain conditions, it may be advisable to include an admixture that will produce an expansion not exceeding 4%. Grout used to fill sheet pile interlocks should be a pumpable mixture. 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 (Section 1.1.2.6), 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 C 939 has a volume of 105.3 in.$^3$ (0.001725 m$^3$) and a discharge-tube diameter of 0.5 in. (13 mm). The discharge diameter of the standard ASTM C 939 flow cone cannot be modified. The current U.S. Army Corps of Engineers standard for measuring the flow of grout (CRD-C611) is essentially the same as ASTM C 939.

The ASTM C 939 flow cone was intended for use with grouts having efflux times of 35 s or less. When the efflux time exceeds 35 s, 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 C 939. For such grouts, ASTM C 939 recommends flowability be determined by the flow table method found in ASTM C 109,[2] using five drops in 3 s.

The grouts used with auger-grout piles are generally too thick to permit proper monitoring of the flow rate with the ASTM C 939 flow cone method. Because the current Corps of Engineers flow cone method (CRD-C611) is identical to ASTM C 939, 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 C 109).

An older U.S. Army Corps of Engineers test method for flow of grout mixtures by the flow cone method, CRD-C79-77 (U.S. Army Corps of Engineers 1977), describes a flow cone with a volume and discharge-tube diameter identical to the ASTM C 939 cone. The 0.5 in. (13 mm) discharge tube on the CRD-C79-77 flow cone, however, 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. discharge opening, has been used as an index of grout fluidity for auger-grout piles. The typical grout efflux range of 10 to 25 s used for auger-grout piles is based on observations using the 0.75 in. opening of the CRD-C79-77 flow cone (DFI 1990, 1994; 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 (DFI 1990, 1994). The continued use of the 0.75 in. 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 would 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.

## 3.6—Anchorages

Anchorage fittings should conform to Chapter 18 of ACI 318-95.

## 3.7—Splices

Materials used for splicing concrete piles should conform to the specifications listed in this chapter where possible. 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.

# CHAPTER 4—MANUFACTURE OF PRECAST CONCRETE PILES

## 4.1—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 ensure high-quality products and usually require less control and field inspection than normal on-site work. Established prestressed- or precast-concrete manufacturing plants are often certified by the Precast/Prestressed Concrete Institute (PCI) or the International Conference of Building Officials (ICBO), or both, thereby providing recognizable quality control.

Certain 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 Manual 116*. Engineers should consider requiring inspection of the prestressing plant during fabrication of the piles by personnel knowledgeable in pile fabrication.

## 4.2—Forms

**4.2.1** *General requirements*—Formwork should adhere to the requirements given in ACI 347R and Chapter 6 of ACI 318-95, except as modified herein.

[2] Note: The flow table used in ASTM C 109 is the ASTM C 230 flow table.

**4.2.2** *Type*—Forms should be suitably permanent — metal, plastic, or concrete constructed — so that the tolerances given in Section 4.6.3 can be maintained. Wood forms 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 release of the prestressing force without damage. Avoid offsets at form joints due to misalignment or an open joint. Fins or offsets in the cast pile can cause stress concentrations, which in some instances have caused shallow cracks to form in the concrete. Grinding of the form surfaces may be required to correct offsets. Leaky joints should be sealed with pressure-sensitive waterproof tape.

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.

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 designer should be assured that the method of controlling the concentricity of the mandrel, strand, spiral, and reinforcement locations is adequate for the job requirements (see Sections 4.2.5, 4.5.3, and 4.6.3 for other discussion).

**4.2.3** *End forms*—End forms or bulkheads should be rigid 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.

**4.2.4** *Chamfers and rounded corners*—All corners of square piles should be chamfered or rounded. Chamfers or radii of approximately 3/4 to 1 in. (19 to 25 mm) are commonly used. Chamfers at the pile head and tip are recommended but not generally used on hollow cylinder piles.

**4.2.5** *Hollow cores*—Hollow cores or voids in piles should be concentric with the pile center line 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 Section 4.6.3.

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. Therefore, 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 (Sections 2.4.10 and 5.2.1.5).

## 4.3—Placement of steel reinforcement

**4.3.1** *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 (Section 7.4.3 ACI 318-95). All tie wire, metal chairs, and other supports for reinforcement should have a minimum cover as given in Chapter 2, or should be of noncorrosive material or protected by a layer of noncorrosive material.

Strands and spiral reinforcement 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. Use spacer hoops (fabricated from two or three turns of spiral wire with an outside diameter equal to the inside diameter of the strand group) installed inside the strand group circle to maintain the strand pattern (or rebar 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 will also be 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. For square strand and reinforcement patterns, similar attention should be paid to ensure a concentric location throughout the pile length.

**4.3.2** *Placement of unstressed steel reinforcement*—Unstressed reinforcement should be placed in accordance with requirements of Chapter 7 of ACI 318-95. Details of reinforcing steel should conform to ACI 315.

**4.3.3** *Placement of prestressed reinforcement*—Placement of prestressed reinforcement and the application and measurement of prestressing force should conform to industry standards such as Chapter 18 of ACI 318-95 and *PCI Manual 116*.

**4.3.4** *Dowel placement*—Dowel holes cast into pile heads should not all end at the same level. Cutoff points for dowel holes should be staggered to avoid stress risers. The same applies to added unstressed steel cast into the pile.

**4.3.5** *Detensioning prestressed strands*—Prestressing strands should be detensioned in accordance with *PCI Man-*

*ual 116*. The detensioning method should minimize any longitudinal movement of the pile in the prestress bed. Use strand-detensioning procedures that minimize unsymmetrical stresses in the cross section and avoid concrete splitting shock around suddenly detensioned strands.

**4.3.6** *Pile end conditions*—For prestressed piles, projecting strands after release 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.

## 4.4—Embedded items

**4.4.1** *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. Particular care should be used to ensure proper cover on all embedded items.

**4.4.2** *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.

## 4.5—Mixing, transporting, placing, and curing concrete

**4.5.1** *Mixing*—Mixing should conform to the general requirements in Chapter 5 of ACI 318-95. Detailed recommendations are given in ACI 304R. The water-cementitious materials ratio (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 ground water.

**4.5.2** *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.

**4.5.3** *Placing*—The placing of concrete should conform to ACI 304R and Chapter 5 of ACI 318-95, except as modified herein.

**4.5.3.1** *Long-line casting*—Precast concrete piles require use of a concrete mixture having a low water-cementitious materials ratio. 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 water-reducing agents. In these mixtures, the water-cementitious materials ratio is important, not the slump. Concrete for precast piles is usually 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 ensure consolidation of the concrete in congested areas, such as at the head of the pile, wherever additional ties or spiral reinforcing are placed, and where reinforcing steel or sleeves are used for doweling. Detailed recommendations are given in ACI 309R.

When shoes, steel stubs, or mechanical splicing attachments (Sections 2.4.4, 2.5.3.5, and 5.6) are cast at the ends of precast piles, particular care should be taken to ensure the proper placement of dense consolidated concrete around such items during casting. Holes through the web and flanges of the stub (vent holes) may be required to ensure 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.

**4.5.3.2** *Centrifugal casting*—Concrete piles manufactured by the centrifugal process should be formed and compacted by centrifugal force in a suitable machine so that the pile molds can be revolved at speeds necessary to ensure even distribution and dense packing of the concrete without the creation of voids behind the reinforcing steel.

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 must be sufficiently rigid to take 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 must be drained or expelled. Concrete slump for pretensioned piles should not exceed 1-1/2 in. (38 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.

**4.5.4** *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.

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

**4.5.5.1** *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 for required duration of curing.

**4.5.5.2** *Membrane curing*—Use and application of liquid membrane-forming compounds for curing should follow the recommendations of ACI 308. Liquid membrane-forming curing compounds should comply with the requirements of ASTM C 309. 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.

**4.5.5.3** *Accelerated curing*—Accelerated curing with low-pressure steam or other heat sources, such as hot water or hot oil lines under the form or electric 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 convective or 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 ensure an even temperature throughout the length of the bed.

A preset period of approximately 2 to 4 h 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+ h). The preset period can be determined by ASTM C 403. During the preset period, the fresh concrete should be protected from the sun or 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 (92 C). The curing should continue until the desired release strength is developed (usually 10 to 15 h).

Prestressed piles cured at high temperatures should not be allowed to cool excessively while the strands are fully anchored to the pretensioning bed as 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 phenomenon, 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 h heat period, then turning off the heat and keeping the line covered until detensioning a day or two later.

## 4.6—Pile manufacturing

**4.6.1** *Post-tensioned*—Post-tensioned piles are often manufactured in sections from 12 to 16 ft long (3.7 to 4.9 m) 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 min, 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 Chapter 18 of ACI 318-95 and Chapter 3 of this report.

**4.6.2** *Prestressing*—Minimum concrete strengths should be 3500 lb/in.$^2$ (24 MPa) for pretensioned piles at the time of stress transfer, and 4000 lb/in.$^2$ (28 MPa) for post-tensioned piles at the time of prestressing, unless higher strengths are required by the design.

**4.6.3** *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.

The permitted departure of the pile head from a plane at right angles to the longitudinal axis of the pile according to ACI 117, 1/4 in./ft of head dimension (20 mm/m), may be too large for conditions where the piles will be subjected to hard driving conditions. In such cases, the engineer may want to specify square driving heads with closer tolerances. Squarer pile ends may also be required when using mechanical splices (Section 2.4.4). The departure from a straight line parallel to the center line of the pile permitted according to ACI 117, 1/8 in. per 10 ft of length (1 mm/m), should be interpreted as the as-built straightness, including the cumulative effects of forming, curing, and long-term storage.

## 4.7—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 predesignated 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 lb/in.$^2$ (24 MPa). Impact stresses due to handling or storage should not exceed the values given in Chapter 2. For calculating handling stresses, a 50% impact factor is recommended (Section 2.2.1.1). Piles should be stored in a manner that will not result in net tensions 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, it 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 Section 3.7. Handling holes are not recommended where driving conditions result in net tension in the section.

# CHAPTER 5—INSTALLATION OF DRIVEN PILES

## 5.0—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 they 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, the reader is referred to general references on pile installation (for example, ASCE 1984; Davisson 1972b; Fuller 1983; Gerwick 1993; Gendron 1970), bibliographies in such general references, and equipment manufacturers' manuals.

The primary purpose of this chapter is to set forth general principles by which driven piling can be properly installed. Only limited recommendations for drilled piles are included herein. 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.

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, that 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 ensure that the piles can be installed in the particular site conditions in a way that will ensure their proper 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 the maximum freedom in the selection of installation methods, specifying results instead of methods where practical, so that maximum economy is obtained and a proper 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.

## 5.1—Installation equipment, techniques, and methods

**5.1.1** *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-lbs (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 (Section 2.1.2.4; also see Section 5.2 where driving 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, however, 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.

**5.1.1.1**—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 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 (Section 5.2.1.2). 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 (Section 1.1.3). This is a long, cylindrical steel weight that falls free 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.

**5.1.1.2**—Hammers powered by steam, air, or hydraulic fluid use external power sources (boilers, compressors, or hydraulic power units) to operate the hammer. For steam- or air-operated hammers, the pressure is released to the atmosphere; however, the pressure release in hydraulically powered hammers involves recirculation of the hydraulic fluid through a closed system. Some of the most recent advances 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 referred to as 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 must 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 a matter of 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 over-stroking.

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 upward stroke. 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 downward stroke. 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 (blows per min). The more rapid action of these hammers, approximately twice that of single-acting hammers, can result in easier penetration and less total time of driving in some soils.

**5.1.1.3**—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. Some diesel hammers have a fuel throttle adjustment for controlling the ram stroke, thus, and the pile stresses during easy driving. The proper system (hammer, cushion, 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.

**5.1.1.4**—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 or down-crowd 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 a frequency 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 is a special condition.

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.

The connection of a vibratory hammer to the pile, usually 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.

**5.1.2** *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.

**5.1.3** *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 (Section 5.1.4) 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 of 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.

**5.1.4** *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, must 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 build up that occur with repeated blows of the hammer.

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 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, 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 (Section 2.1.2.4).

**5.1.5** *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 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 to ensure that the driving force is not transmitted primarily through the pipe top.

**5.1.6** *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, rejetting or jetting can segregate these coarse materials to the bottom of the jetted hole, making it difficult to drive the pile through them.

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 so as 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 to ensure 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 (Section 5.2.1.5) during subsequent driving. External jetting is preferred for hollow-core concrete piles. If internal jetting must be used, it may be desirable to switch to a pile of solid cross section.

**5.1.7** *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 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 (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.

**5.1.8** *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 found useful in 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.

**5.1.9** *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.

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.

**5.1.10** *Followers*—Frequently a pile must 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 must be sufficiently rigid to ensure adequate transmission of 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 to ensure that it will be able to withstand dynamic driving stresses and allow adequate transmission of hammer energy to the pile. The wave-equation analysis (Section 2.1.2.4) can be used to assess the effect of the follower on the pile-driving characteristics and also the pile and follower stresses.

## 5.2—Prevention of damage to piling during installation

**5.2.1** *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:

- Spalling of concrete at the pile head due to high compressive stress;

- Spalling of concrete at the pile point (tip) due to hard driving resistance at the point;
- Transverse cracking or breaking of the pile due to tensile stress reflections from the tip or head of the pile;
- 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;
- Spalling and cracking due to a combination of compression or tension reflections and bending stress resulting from pile curvature; and
- Longitudinal splits of hollow piles due to internal radial pressures.

**5.2.1.1**—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:

- 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;
- 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;
- Improper alignment of the hammer and pile, resulting in an eccentric hammer blow that causes high stress concentrations;
- Impact on longitudinal reinforcing steel protruding above the pile head, resulting in high stress concentrations in the concrete adjacent to the reinforcing;
- Lack of adequate transverse reinforcement (spiral confinement) at the pile head;
- The top edges and corners of the concrete pile are not chamfered, causing the edges or corners to spall; and
- Fatigue failure of the concrete under a large number of hammer blows at a high stress level.

**5.2.1.2**—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).

**5.2.1.3**—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 long piles 50 ft (15 m) or over. 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 will depend 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.

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

- Insufficient cushioning material used between the drive head and the concrete pile, resulting in a stress wave of high magnitude and short length. Use of an adequate softwood cushion is frequently the most effective way of reducing driving stresses. Stress reductions on the order of 50% can be obtained with new, uncrushed cushions. As the cushion is compressed by hard driving, the intensity of the stress wave increases. Therefore, use of a new cushion for each pile is recommended;
- High ram velocity, which produces a stress wave of high magnitude;
- Critical tensile stress reflections resulting from little or no tip resistance. This condition is most critical in long piles, 50 ft (15 m) or longer. This is possible when driv-

ing 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; and
- Critical tensile stresses resulting from the short wave produced when driving against very high tip resistance with a relatively light hammer.

**5.2.1.4**—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 they 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.

**5.2.1.5**—Internal radial pressures in both open- and close-ended hollow precast piles lead to tension in the pile walls and can cause bursting of such piles. 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.

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 highwater 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.

**5.2.1.6**—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 (salt water, and corrosive soils), and the modes of loading to be resisted by the pile should be evaluated together to ascertain whether a replacement pile is necessary.

**5.2.2** *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 be continued until the new cushioning has been adequately compressed to ensure the reliability of the delivered hammer energy before observing the final set. 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 (capblock) 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, ensure that the pile point is 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 and desirable to drive with larger hammers or to greater driving resistance rather than to jet and drive simultaneously;

f) Ensure that the drive head fits 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) Ensure that the pile is straight and not cambered because of uneven prestress or poor manufacturing or storage methods, or both. High flexural stresses can result during driving of a crooked pile;

h) Ensure that the top of the pile is square or perpendicular to the longitudinal axis of the pile and that no strands or reinforcing protrudes 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 lb/in.$^2$ (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 lb/in.$^2$ (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 with no difficulty. Prestress values required to accommodate driving stress conditions should be determined by wave-equation analysis (Section 2.1.2.4) or other acceptable means;

k) Ensure that the pile is 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 two 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 (Section 5.2.1.5).

**5.2.3** *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 tight fit is permissible and helpful in preventing bulging.

**5.2.4** *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 in combating 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 requisite 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.

**5.2.5** *Misalignment of piles*—Specifying an axial alignment tolerance as a percentage of actual length is common. The frequently specified tolerance of 2% 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 (Section 2.4.2). Excessive restriction on axial alignment often leads to the 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.

**5.2.6** *Distortion of piles*—Pile distortion can be produced during installation when driving past or through obstructions or boulders. For casings driven without a mandrel, use of a heavier wall and a reinforced shoe will help. For shells driven with a mandrel, 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 must 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. Use of 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.

**5.2.7** *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.

**5.2.8** *Enlarged-tip piles*—When enlarged-tip piles are driven through certain soils, it may be necessary to take special measures to re-establish 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.

**5.2.9** *Pile heave and flotation*—See Section 2.4.8 for a discussion of pile heave and flotation, which can influence pile installation methods.

## 5.3—Handling and positioning during installation

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

**5.3.1** *Handling*—Piles should be picked up so as not to 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 (Sections 2.2 and 4.7).

**5.3.2** *Positioning*—Correct position can be ensured by accurately setting the pile. Removal of near-surface obstructions will facilitate accurate positioning. Where accuracy of position is critical, a template or 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 the resetting of 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.

**5.3.3** *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.

**5.3.3.1**—Both the driver leaders and the pile must 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 to ensure verticality for plumb piles. 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.

**5.3.3.2**—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; use of rollers in the leaders is one method. Slender vertical piles may require guides at intervals to prevent buckling under the hammer blow.

**5.3.3.3**—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.

**5.3.4** *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 must be supported to overcome the dead weight. Frequently, when driving in deep water, a batter pile must be stayed before it is released from the hammer.

**5.3.5** *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 (Section 2.4.1).

## 5.4—Reinforcing steel and steel core placement

**5.4.1** *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 (Section 5.5.7); 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 (Section 2.5.3). Some designers prefer to extend one or more bars toward the tip in the CIS piles or CIP piles with very thin shells to ensure continuity of the pile.

**5.4.2** *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 are best 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 must 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 must 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.

**5.4.3** *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 to ensure the core is centered for the full length of embedment.

## 5.5—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. The concrete should be placed so as to ensure 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; and
- 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 potential presence of defects are 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; and
- Observations of obvious improper inspection or concreting procedures, or both, for the particular conditions.

The prevention of concrete defects and the identification of conditions conducive to their development in CIP and CIS piles depends on 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.

**5.5.1** *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 ensure concrete flow through and around the reinforcement;
- *Condition of pile*—The conditions of the pile (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 ground water 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 (slump, ratio of coarse-to-fine aggregate, maximum coarse aggregate size, water-cementitious materials ratio, 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.

**5.5.2** *Inspection before concreting*—After a CIP pile is driven, it should be inspected to ensure 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 can be driven while the driving rig is still near by if necessary.

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.

**5.5.3** *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.

**5.5.4** *Concrete mixture proportions*—Concrete mixture proportions for CIP piles should be designed to ensure 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. Use of admixtures can be beneficial in obtaining the desired workability and nonbleeding characteristics (Chapter 3).

Concrete mixtures containing approximately 800 lb of coarse aggregate/yd$^3$ (475 kg/m$^3$) (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 must be placed through the reinforcement cage, batter piles, and very long piles with extensive corrugations or steps, or both. 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 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 (Section 5.5.5.1). Most properly designed pump mixtures used with piles have a reduced coarse aggregate content and also maximum aggregate size, resulting in high sand and cement contents and behavior similar to that of the mixtures with reduced coarse aggregate described previously.

**5.5.5** *Concrete placement methods and techniques*—Concrete should not be dumped directly into 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.

**5.5.5.1**—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 of 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 ensure that it is free of foreign matter, including appreciable water (2 in. or 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 less 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 them, 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.

**5.5.5.2**—Underwater placement is used 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 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 Section 5.5.3. 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 can 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.

**5.5.6** *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. Overvibration should be avoided because it can induce excessive bleeding.

**5.5.7** *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 Section 5.5.6 may be desirable under these circumstances.

**5.5.8** *Compaction of uncased pile*—Some CIS piles use ramming during concrete placement to compact and consolidate the concrete (Section 1.1.3). In those pile types where the casing is simultaneously withdrawn, care should be exercised to:

- Overcome pullup 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.

**5.5.9** *Cast-in-drilled-hole piles*—Cast-in-drilled-hole piles 30 in. (760 mm) and larger are covered in ACI 336.1, ACI 336.1R, and ACI 336.3R. 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 CIP 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 so as to avoid the possibility of arching of the concrete in the liner and possible discontinuities in the pile shaft as the liner is withdrawn.

**5.5.10** *Auger-grout or concrete-injected piles*—Auger-grout piles (Section 1.1.7.3) 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 in lieu 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). Neely (1990) discusses some of the factors affecting the installation of auger-cast piles. Model specifications and an inspector's guide by the Deep Foundation Institute (DFI 1990, 1994) also discuss the installation of such piles.

Grout should conform to the recommendations of Section 3.5. 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.

**5.5.11** *Drilled and grouted piles*—Drilled and grouted piles (Section 1.1.7.4) 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 to ensure a full-sized and continuous pile. 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 Section 3.5 and the casing should not be withdrawn faster than the hole is being filled with grout.

## 5.6—Pile details

**5.6.1** *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 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.

**5.6.2** *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 (Sections 4.2.4 and 5.2.1.1), special pile shoes are not generally required nor used. Cast or fabricated steel points and flat steel plates are sometimes beneficial when piles must 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 (Sections 5.2.4 and 5.6.1).

Shoes should be securely attached to the main body of the pile by anchor rods of 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 ensure the proper placement of dense, consolidated concrete in the shoe during casting. Depending on the configuration of the shoe, vent holes in the shoe may be required (Section 4.5.3.1).

**5.6.3** *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; 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 (Sections 2.4.10 and 4.5.3.1).

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

**5.6.4.1**—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 the applicable American Welding Society requirements, AWS D1.1 and AWS D1.4, 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 must be overcome.

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

**5.6.4.3**—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 must accommodate the specific splice that is to be used. Care should be taken in splicing to ensure 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 (Section 2.4.4).

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

**5.6.5** *Cutoff of precast piles 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 pavement 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.

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

**5.6.6** *Extension of precast piles*—Extensions are used when the pile has been driven a short distance below grade. Lowering of 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 Section 5.6.4. Special care to ensure durability should be taken at the splice where the pile is subject to marine and other adverse exposure.

## 5.7—Extraction of concrete piles

Concrete piles can be extracted by direct pull, jetting, vibration, excavation, jacking, or a combination theses 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 (Section 5.2.1). 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 (Section 5.2.1.6).

## 5.8—Concrete sheet piles

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

**5.8.1** *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; and
- 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.

**5.8.2** *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 reinforcing in the wings.

**5.8.3** *Grouting of joints*—Joints can be grouted directly or by first inserting a light fabric tube. If the tube is slightly porous (for example, burlap) 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.

# CHAPTER 6—REFERENCES

## 6.1—Reference standards and reports

The specifications, standards, and reports of the American Concrete Institute (ACI), the American Society for Testing and Materials (ASTM), the American Welding Society (AWS), the Precast/Prestressed Concrete Institute (PCI), and the U.S. Army Corps of Engineers are listed below with their serial designations. The listed items in this section are based on the latest editions available at the time this report was originally published. Because specifications are revised periodically, the user of this report should check with the sponsoring group to verify the most recent versions. Specific editions of specifications, standards, reports, and codes referenced in this report (such as ACI 318-95) are included in the cited references list, Section 6.2.


*American Concrete Institute*

117 Standard Tolerances for Concrete Construction and Materials

201.2R Guide to Durable Concrete

211.1 Standard Practice for Selecting Proportions for Normal, Heavyweight, and Mass Concrete

211.4R Guide for Selecting Proportions for High-Strength Concrete with Portland Cement and Fly Ash

212.3R Chemical Admixtures for Concrete

212.4R Guide for Use of High-Range Water-Reducing Admixtures (Superplasticizers) in Concrete

221R Guide for Use of Normal Weight and Heavyweight Aggregates in Concrete

222R Corrosion of Metals in Concrete

301 Standard Specifications for Structural Concrete

304R Guide for Measuring, Mixing, Transporting, and Placing Concrete

305R Hot Weather Concreting

306R Cold Weather Concreting

308 Standard Practice for Curing Concrete

309R Guide for Consolidation of Concrete

315 ACI Detailing Manual (SP-66)

336.1 Reference Specifications for the Construction of Drilled Piers

336.1R Commentary—Reference Specifications for the Construction of Drilled Piers

336.3R Design and Construction of Drilled Piers

347R Guide to Formwork for Concrete

503R Use of Epoxy Compounds with Concrete

517.2R Accelerated Curing of Concrete at Atmospheric Pressure—State of the Art

*ASTM*

A 36 Standard Specification for Carbon Structural Steel
A 82 Standard Specification for Steel Wire, Plain, for Concrete Reinforcement
A 185 Standard Specifications for Welded Steel Wire Fabric, Plain, for Concrete Reinforcement
A 242 Standard Specification for High-Strength Low-Alloy Structural Steel
A 252 Standard Specification for Welded and Seamless Steel Pipe Piles
A 283 Standard Specification for Low and Intermediate Tensile Strength Carbon Steel Plates
A 366 Standard Specification for Commercial Steel (CS) Sheet, Carbon, (0.15 Maximum Percent), Cold-Rolled
A 416 Standard Specification for Steel Strand, Uncoated Seven-Wire for Prestressed Concrete
A 421 Standard Specification for Uncoated Stress-Relieved Steel Wire for Prestressed Concrete
A 496 Standard Specification for Steel Wire, Deformed, for Concrete Reinforcement
A 497 Standard Specification for Steel Wire Fabric, Deformed, for Concrete Reinforcement
A 569 Standard Specification for Steel, Carbon (0.15 Maximum, Percent), Hot Rolled Sheet and Strip Commercial Quality
A 570 Standard Specification for Steel, Sheet and Strip, Carbon, Hot-Rolled, Structural Quality
A 572 Standard Specification for High-Strength Low-Alloy Columbium-Vanadium Structural Steels
A 615 Standard Specification for Deformed and Plain Billet-Steel Bars for Concrete Reinforcement
A 616 Standard Specification for Rail-Steel Deformed and Plain Bars for Concrete Reinforcement
A 617 Standard Specification for Axle-Steel Deformed and Plain Bars for Concrete Reinforcement
A 706 Standard Specification Low-Alloy Steel Deformed and Plain Bars for Concrete Reinforcement
A 722 Standard Specification for Uncoated High-Strength Steel Bar for Prestressing Concrete
A 775 Standard Specification for Epoxy-Coated Reinforcing Steel Bars
A 882 Standard Specification for Epoxy-Coated Seven-Wire Prestressing Steel Strand
A 884 Standard Specification for Epoxy-Coated Steel Wire and Welded Wire Fabric for Reinforcement
A 955 Standard Specification for Deformed and Plain Stainless Steel Bars for Concrete Reinforcement
C 31 Standard Practice for Making and Curing Concrete Test Specimens in the Field
C 33 Standard Specification for Concrete Aggregates
C 39 Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens
C 109 Standard Test Method for Compressive Strength of Hydraulic Cement Mortars (Using 2-in. or 50-mm Cube Specimens)
C 143 Standard Test Method for Slump of Hydraulic-Cement Concrete
C 150 Specification for Portland Cement
C 172 Standard Practice for Sampling Freshly Mixed Concrete
C 173 Standard Test Method for Air Content of Freshly Mixed Concrete by the Volumetric Method
C 230 Standard Specification for Flow Table for Use in Tests of Hydraulic Cement
C 231 Standard Test Method for Air Content of Freshly Mixed Concrete by the Pressure Method
C 260 Standard Specification for Air-Entraining Admixtures for Concrete
C 309 Standard Specification for Liquid Membrane-Forming Compounds for Curing Concrete
C 360 Standard Test Method for Ball Penetration in Freshly Mixed Hydraulic Cement Concrete
C 403 Standard Test Method for Time of Setting of Concrete Mixtures by Penetration Resistance
C 494 Standard Specification for Chemical Admixtures for Concrete
C 595 Standard Specification for Blended Hydraulic Cements
C 618 Standard Specification for Fly Ash and Raw or Calcined Natural Pozzolan for Use as a Mineral Admixture in Concrete
C 939 Standard Test Method for Flow of Grout for Preplaced-Aggregate Concrete (Flow Cone Method)
C 1017 Standard Specification for Chemical Admixtures for Use in Producing Flowing Concrete
C 1240 Standard Specification for Silica Fume for Use as a Mineral Admixture in Hydraulic-Cement Concrete, Mortar, and Grout
D 1143 Standard Test Method for Piles Under Static Axial Compressive Load
D 3689 Standard Test Method for Individual Piles under Static Axial Tensile Load
D 3966 Standard Test Method for Piles under Lateral Loads
D 4945 Standard Test Method for High Strain Dynamic Testing of Piles

*American Welding Society*

D1.1 Structural Welding Code—Steel
D1.4 Structural Welding Code—Reinforcing Steel

*Precast/Prestressed Concrete Institute*

MNL 116 Manual for Quality Control for Plants and Production of Precast and Prestressed Concrete Products

*U.S. Army Corps of Engineers*

CRD-C611 Standard Test Method for Flow of Grout for Preplaced-Aggregate Concrete (Flow-Cone Method)

The above publications are available from the following organizations:

American Concrete Institute
P.O. Box 9094
Farmington Hills, Mich. 48333-9094

ASTM
100 Barr Harbor Dr.
West Conshohocken, Pa. 19428-2959

American Welding Society
550 N.W. LeJeune Rd.
P.O. Box 351040
Miami, Fla. 33135

Precast/Prestressed Concrete Institute
175 West Jackson Blvd.
Chicago, Ill. 60604

U.S. Army Corps of Engineers
Waterways Experimental Station
3909 Hall Ferry Rd.
Vicksburg, Miss. 39180-6199

## 6.2—Cited references

ACI Committee 340, 1990, *ACI Design Handbook, V. 2, Columns*, SP-17A(90), American Concrete Institute, Farmington Hills, Mich., 221 pp.

ACI Committee 318, 1995, "Building Code Requirements for Structural Concrete (ACI 318-95) and Commentary (ACI 318R-95)," American Concrete Institute, Farmington Hills, Mich., 369 pp.

ASCE Deep Foundations Committee, 1984, *Practical Guidelines for the Selection, Design, and Installation of Piles*, ASCE, New York, 105 pp.

Banerjee, S.; Stanton, J. F.; and Hawkins, N. M., 1987, "Seismic Performance of Precast Prestressed Concrete Piles," *Journal of Structural Engineering,* ASCE, V. 113, No. 2, Feb., pp. 381-396.

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