Recommended Practice for Design, Manufacture, and Installation of Prestressed Concrete Piling

PCI Prestressed Concrete Piling Committee

JP Binard, PE, chair
Sriram Aaleti, PhD
Simon Harton, PE
Louis Klusmeyer, PE, SE
Andrew Levens, PE
Richard Potts, PE
Mehedi Rashid, PE, SE
Sami Rizkalla, PhD
Joe Rose, PE
Stephen Seguirant, PE
Sri Sritharan, PhD
Catrina Walter, PE
Christopher White, PE
Danny Wong

Consulting Members

John Dick
Roy Eriksson, PE
Mohamad Hussein, PE
John Kelly, PE
Jan Landreth-Smith
Farshad Mazloom
Mac McCalla, PE
Saad Eldin Moustafa, PhD
John Ryan, PhD, PE

Staff Liaison

William N. Nickas, PE

Technical Editor

Timothy Mays, PhD, PE

Section 1—Introduction

Prestressed concrete piles are vital elements in the foundations of buildings, bridges and marine structures throughout the world. They usually vary in size from 12 in. (305 mm) square piles used in building foundations to 66 in. (1680 mm) diameter cylindrical piles used in marine structures and bridges. Many areas of North America have poor soil conditions requiring pile foundations for even relatively light structures. In such areas, prestressed concrete piling has come to be the usual method of construction, with prestressed concrete having been proved the logical choice of materials where permanence, durability, and economy must be considered. Heavy marine structures often rely on prestressed concrete piles driven through deep water or through deep layers of unsuitable material for their support. Prestressed concrete piles can be designed to safely support these heavy axial loads, as well as lateral loads caused by wind, waves, and earthquakes. In marine environments, these piles can resist corrosion caused by salt water and by thousands of cycles of wetting and drying.

1.1 Scope of report

This report contains recommendations and guidelines for the design and detailing of prestressed concrete piles based on current knowledge and standards. It updates the previous version of the report published in the March–April 1993 issue of the *PCI Journal*.[1] Although typical pile cross sections discussed in this report are those most commonly found throughout the United States and Canada, the intent of this report is not to limit the geometric configuration or material properties of prestressed concrete piling. For the first time in a PCI publication on prestressed pile design, performance-based design is presented. Performance-based design of prestressed concrete piles allows the designer to detail prestressed concrete piles more economically than prescriptive methods typically allow, with nontraditional materials or material properties or novel construction techniques, as long as the expected performance meets the requirements of this report and those contained in the governing code for the structure. Local prestressed concrete manufacturers should be consulted for readily available cross sections. Design tables and details presented are intended to aid the qualified designer. Actual design details, including the selection of materials, pile sizes, and pile shapes, should conform to local practices and code requirements.

1.1.1 Materials Section 2 discusses cements, aggregates, water, admixtures, and reinforcement. Recommendations are made regarding these constituents and their effects on the quality and strength of concrete.

1.1.2 Design Section 3 begins with a discussion of various factors that should be considered in the design of prestressed concrete piles and pile foundations. Prescriptive and performance-based design provisions are presented in detail with reference to how they should be used in conjunction with governing codes and standards. Although some design aids are provided, reference is made to PCI's free software program (PCI PD-01) for developing axial-moment interaction diagrams for the ultimate capacity of commonly used pile sizes of varying concrete strengths and effective prestress levels.

Good design practice requires clear communication between the project engineer of record, the project geotechnical engineer, and, when applicable, the specialty engineer responsible for the design of the prestressed concrete piling.

1.1.3 Manufacture and transportation Section 4 covers special requirements involved in the manufacture, handling, transportation, and tolerances for prestressed concrete piles.

1.1.4 Installation The purpose of section 5 is to set forth general principles for proper prestressed concrete pile installation. Discussion of handling- and driving-induced compression, tension, bending, and torsion provides a basis for recommendations to prevent damage to prestressed concrete piles. Techniques related to the cutting or occasional extension of piles are suggested.

1.1.5 Current research and future applications for prestressed concrete piling Section 6 concludes this report by highlighting some of the current issues and future research needs involving the design, manufacture, and installation of prestressed concrete piles. A state of practice overview of mitigating and monitoring the effects of pile-driving vibrations is included. Current pile-related research showing great promise is mentioned and future research needs are also examined.

1.2 Standards and references

1.2.1 ASTM standards

ASTM A29, *Standard Specification for General Requirements for Steel Bars, Carbon and Alloy, Hot-Wrought*

ASTM A416, *Standard Specification for Low-Relaxation, Seven-Wire Steel Strand for Prestressed Concrete*

ASTM A421, *Standard Specification for Stress-Relieved Steel Wire for Prestressed Concrete*

ASTM A572, *Standard Specification for High-Strength Low-Alloy Columbium-Vanadium Structural Steel*

ASTM A615, *Standard Specification for Deformed and Plain Carbon-Steel Bars for Concrete Reinforcement*

ASTM A706, *Standard Specification for Deformed and Plain Low-Alloy Steel Bars for Concrete Reinforcement*

ASTM A722, *Standard Specification for High-Strength Steel Bars for Prestressed Concrete*

ASTM A882, *Standard Specification for Filled Epoxy-Coated Seven-Wire Prestressing Steel Strand*

ASTM A884, *Standard Specification for Epoxy-Coated Steel Wire and Welded Wire Reinforcement*

ASTM A1060, *Standard Specification for Zinc-Coated (Galvanized) Steel Welded Wire Reinforcement, Plain and Deformed, for Concrete*

ASTM A1064, *Standard Specification for Carbon-Steel Wire and Welded Wire Reinforcement, Plain and Deformed, for Concrete*

ASTM C31, *Standard Practice for Making and Curing Concrete Test Specimens in the Field*

ASTM C33, *Standard Specification for Concrete Aggregates*

ASTM C39, *Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens*

ASTM C143, *Standard Test Method for Slump of Hydraulic Cement Concrete*

ASTM C150, *Standard Specification for Portland Cement*

ASTM C172, *Standard Practice for Sampling Freshly Mixed Concrete*

ASTM C260, *Standard Specification for Air-Entraining Admixtures for Concrete*

ASTM C330, *Standard Specification for Lightweight Aggregates for Structural Concrete*

ASTM C494, *Standard Specification for Chemical Admixtures for Concrete*

ASTM C595, *Standard Specification for Blended Hydraulic Cements*

ASTM C618, *Standard Specification for Coal Fly Ash and Raw or Calcined Natural Pozzolan for Use in Concrete*

ASTM C989, *Standard Specification for Slag Cement for Use* *in Concrete and Mortars*

ASTM C1107, *Standard Specification for Packaged Dry, Hydraulic-Cement Grout (Nonshrink)*

ASTM C1157, *Standard Performance Specification for Hydraulic Cement*

ASTM C1240, *Standard Specification for Silica Fume Used in Cementitious Mixtures*

ASTM C1260, *Standard Test Method for Potential Alkali Reactivity of Aggregates (Mortar-Bar Method)*

ASTM C1602, *Standard Specification for Mixing Water Used in the Production of Hydraulic Cement Concrete*

ASTM C1611, *Standard Test Method for Slump Flow of Self-Consolidating Concrete*

ASTM C1778, *Standard Guide for Reducing the Risk of Deleterious Alkali-Aggregate Reaction in Concrete*

ASTM D1143, *Standard Test Methods for Deep Foundations Under Static Axial Compressive Load*

ASTM D3689, *Standard Test Methods for Deep Foundations Under Static Axial Tensile Load*

ASTM D3966, *Standard Test Methods for Deep Foundations Under Lateral Load*

ASTM D4945, *Standard Test Method for High-Strain Dynamic Testing of Deep Foundations*

ASTM D7383, *Standard Test Methods for Axial Compressive Force Pulse (Rapid) Testing of Deep Foundations*

1.2.2 American Concrete Institute (ACI) standards and guides

ACI 201.2R, *Guide to Durable Concrete*

ACI 318, *Building Code Requirements for Structural Concrete and Commentary*

ACI 543R, *Guide to Design, Manufacture, and Installation of Concrete Piles*

1.2.3 PCI standards and references

PCI MNL-116, *Manual for Quality Control for Plants and Production of Structural Precast Concrete Products*

PCI MNL-120, *PCI Design Handbook: Precast and Prestressed Concrete*

PCI MNL-133, *PCI Bridge Design Manual*

PCI MNL-137, *Manual for the Evaluation and Repair of Precast, Prestressed Concrete Bridge Products*

PCI PD-01, *Calculation of Interaction Diagrams for Precast, Prestressed Piles*

PCI STD-112, *Standard Prestressed Concrete Piles*

PCI STD-113, *Prestressed Concrete Sheetpiles*

1.2.4 American Welding Society (AWS) standards

AWS D1.1, *Structural Welding Code – Steel*

AWS D1.4, *Structural Welding Code – Reinforcing Steel*

1.2.5 Post-Tensioning Institute (PTI) References

PTI M55.1, *Specification for Grouting of Post-Tensioned Structures*

Section 2—Materials

Material properties and requirements for prestressed concrete piles are almost identical to those for all precast, prestressed concrete structural members. Piles, however, are subject to long-term contact with soil and moisture as well as submergence in water and other aggressive environments. Therefore, the in-service conditions require that materials used in piles be carefully selected to reduce long-term durability issues such as corrosion, alkali-silica reaction, and delayed ettringite formation. Special consideration should be given to using appropriate materials that ensure satisfactory short- and long-term performance of the piling. On any project, in addition to satisfying the recommendations provided in this report, designers must also consider local or state specifications governing the use and availability of certain products or materials used to construct prestressed concrete piles. This section provides a detailed discussion regarding material constituents of concrete. Before casting prestressed concrete piles, the mixture proportions for the concrete should be submitted to the engineer or owner for review and approval.

2.1 Cementitious materials

Portland cement conforming to ASTM C150 Types I, II, III, or V is commonly used in prestressed concrete piling. Blended and performance-based cements conforming to ASTM C595 and ASTM C1157 may also be used in prestressed concrete piling. Selection of the appropriate cement type and other constituents for a particular project should be based on the exposure conditions to which the piles will be subjected and local experience in these conditions, in addition to strength, strength gain, and workability requirements. Type III (or high early strength) cement is often used for piles to facilitate rapid reuse of the formwork.

In areas of moderate exposure to sulfate-containing soils (soils containing 0.10% to 0.20% of water-soluble sulfate by weight) or sulfate-containing waters (seawater or waters containing 150 to 1500 ppm dissolved sulfate), the tricalcium aluminate (C_3A) content of the cement should be limited to 8% (ACI 318-14).[2] Note that seawater is explicitly listed under Exposure Class S1 with the acknowledgment that water-soluble-sulfate concentration may exceed 1500 ppm dissolved sulfate. For areas of severe sulfate exposure, ACI 318 recommends a 5% limit on C_3A for concentrations over 0.20% for soils or 1500 ppm for waters (note that Exposure Class S2 has upper bound concentrations of 2.00% of water-soluble sulfate and 10,000 ppm for waters). However, for higher-strength concretes (6000 psi [41.4 MPa] and greater) employed in the manufacture of prestressed concrete piles, the 8% limit on C_3A is considered adequate for sulfate concentrations over 0.20%. See ACI 543R-12[3] for more information. When using blended cements, MS- and HS-designated cements should be used in moderate sulfate (MS) exposures and high sulfate (HS) exposures, respectively. It is suggested that local prestressed concrete manufacturers be consulted for readily available cements, as special cement requirements will affect cost and performance.

Cements meeting the sulfate requirements presented previously are classified respectively as Type II (providing moderate sulfate resistance) and Type V (providing high sulfate resistance). A number of Type III cements also meet these requirements. Blended hydraulic cements meeting ASTM C595 or hydraulic cements complying with the performance requirements of ASTM C1157 are classified as either MS- or HS-resisting based on results from an ASTM C1012 test. Alternative combinations of cementitious materials may be used if tests for sulfate resistance demonstrate satisfactory performance in ASTM C1012 testing. Type V cement may not be readily available in all parts of the country, in which case ACI 543R-12[3] and some state highway specifications recommend substitution of Type II cement combined with Class F fly ash. For the higher-strength concretes used with prestressed concrete piles, the use of Type V cement may not be absolutely necessary, even for severe sulfate exposure conditions, unless required by the governing code for the structure.

For prestressed concrete piles exposed to seawater, tests show that concrete made with cement containing 5% to 8% C_3A exhibits less cracking because of steel reinforcement corrosion than concretes made with cement with less than 5% C_3A.[4] Although cements with these lower C_3A contents (Type V) provide increased sulfate resistance, they tend to increase the risk of steel corrosion (chloride exposure). It is therefore recommended that Type V cement not be used for piles in seawater or other high-chloride environments.

Fly ash (ASTM C618), slag cement (ASTM C989), silica fume (ASTM C1240), or pozzolanic materials (ASTM C618) may be combined with cement to enhance certain characteristics of concrete. Although these supplementary cementitious materials are often combined with cement to produce higher compressive strengths, their use in piles is primarily to increase the long-term durability of concrete. When combined in the proper proportions, the result is generally a denser, less porous concrete with greater resistance to chloride and sulfate penetration. Many local and state specifications have requirements for their use under certain exposure conditions or limit their proportions relative to the total cementitious material content. Blended cements, when used, should conform to ASTM C595.

2.2 Aggregates

Fine and coarse aggregates should conform to ASTM C33. Although not commonly used, lightweight aggregates should conform to ASTM C330. Aggregates failing to meet these specifications, but which have been shown by special tests or actual service to produce concrete of adequate strength and durability, may be used with approval of the governing authority. Aggregates susceptible to alkali-aggregate reactivity (AAR) are not recommended for use in prestressed concrete piles; if they are used, special precautions must be observed. ASTM C1778 can be used to address potential AAR concerns. Testing for alkali reactivity of aggregates should be performed in accordance with guidance given in ASTM C1778-16. Special attention should be paid to the type, distribution, and maximum size of aggregate to promote proper mixing and consolidation, particularly for concrete containing pozzolanic materials and those used with closely spaced transverse reinforcement (for example, for seismic applications).

2.3 Water

Water used for washing aggregates, mixing concrete, or curing concrete must be clean and free of oil, salt, acid, alkali, and deleterious substances. Water of potable quality is recommended but not required. Mixing water for concrete should conform to ASTM C1602 and should not contain a chloride ion concentration more than 500 ppm or sulfates in the form of SO_4 in excess of 1300 ppm.

2.4 Admixtures

All admixtures used in concrete piling must be compatible with each other and with the cement being used. Certain combinations of admixtures may further improve handling, placement, finishing, and performance characteristics of the concrete. A brief discussion of admixtures used for prestressed concrete piles follows.

Air-entrained concrete may be used in piles that will be subjected to cycles of freezing and thawing or wetting and drying. Air-entraining admixtures conforming to ASTM C260 should be used where concrete piles are exposed to these conditions. The amount of air entrainment and its effectiveness depend on the admixture used, the size and type of the coarse aggregate, the moisture content of the aggregate, and other variables. Too much air will lower the strength of the concrete and too little will reduce its effectiveness. It is recommended that the air content of the concrete be less than 7% by volume, depending on the size of the coarse aggregate and the severity of exposure. It is important to note that air-entraining admixtures are typically not considered effective for prestressed high-strength concrete piling applications, due to the high density and low porosity associated with high-strength concrete. Using air entrainment in high-performance concretes, and other mixtures using pozzolanic or nonpozzolanic materials for durability considerations, normally will not increase freezing and thawing resistance significantly. In some cases, however, air-entraining admixtures may be used to improve concrete workability, especially for piles with very closely spaced transverse reinforcement. Overall, site conditions should be evaluated against the quality of the concrete before specifying air entrainment.

When used, water-reducing admixtures (including those for self-consolidating concrete), retarding admixtures, accelerating admixtures, water-reducing and retarding admixtures, and water-reducing and accelerating admixtures should conform to ASTM C494. Calcium chloride or admixtures containing calcium chloride should not be used. Water-reducing admixtures are sometimes required when significant percentages of silica fume or pozzolanic materials are used in the concrete, and the improved workability due to their use is especially beneficial for piles with closely spaced transverse reinforcement.

Corrosion inhibitors, such as calcium nitrite, may be specified when concrete piles will be exposed to seawater, or in other applications where the potential for chloride attack is high. The use of calcium nitrite may affect the workability and setting characteristics of the concrete, so it is important to adjust the mixture proportions to account for these effects. The use of high-performance, high-strength concrete, with decreased permeability, may make it unnecessary to use corrosion inhibitors.

When approved by the engineer, shrinkage-reducing admixtures can be used to reduce prestress losses due to concrete shrinkage.

2.5 Concrete strength

For prestressed concrete piles, the minimum concrete strength at the time of prestress transfer f'_{ci} should be 3500 psi (24.1 MPa). Concrete in precast, prestressed concrete piles and build-ups should have a minimum compressive strength f'_c of 5000 psi (34.5 MPa) at 28 days. Pile design, handling configurations, and some agencies may require higher minimum concrete strengths at transfer and at 28 days. Economy in handling and driving along with higher load capacity can be achieved with concrete strengths up to 10,000 psi (68.9 MPa). Designers should check with local pile manufacturers to determine attainable strengths.

For the sake of durability, concrete piles should have a minimum cementitious material content of 564 lb/yd^3 (335 kg/m^3) of concrete. The water–cementitious material ratio (by weight) should correspond to the least amount of water that will produce a plastic mixture and provide the desired workability for the most effective placement of the concrete. Maximum water–cementitious material ratios are typically based on exposure. In aggressive environments, such as for marine applications or for sites with high chloride or high sulfate exposure, a minimum cementitious material content of 658 lb/yd^3 (390 kg/m^3) is recommended. See ACI 201.2R-08[4] for durability recommendations where pozzolans and fly ash (ASTM C618) are used.

2.6 Steel reinforcement

Prestressed concrete piles are typically produced using seven-wire strand conforming to ASTM A416 for longitudinal prestressing, wire spiral conforming to ASTM A1064 for transverse reinforcement, and deformed reinforcement conforming to ASTM A615 or ASTM A706, typically used for connection detailing. Strand sizes range from ⅜ in. (9.5 mm) to 0.6 in. (15.2 mm) diameter, with ½ in. (12.7 mm) diameter being a commonly used size in piles. Although not commonly used, epoxy-coated prestressing strand conforming to ASTM A882 and epoxy-coated wire conforming to ASTM A884 for spiral reinforcement can be specified. Wire for spiral reinforcement typically has higher yield strength than typical nonprestressed reinforcement, with yield strengths up to 100 ksi (690 MPa). Galvanized strand and deformed reinforcing bars as well as stainless-steel strand and reinforcing bars are also design options but are rarely specified for use in prestressed concrete piles. Some agencies do require the use of galvanized wire (ASTM A1060). The use of both galvanized and nongalvanized steel in a pile may not be a good idea. Although pile-driving conditions necessitate symmetrically placed spiral reinforcement and smaller spiral pitch at the ends of a pile, nonsymmetrical spiral reinforcement may be desired any time large shear forces or seismic confinement requirements exist along a designated length of the pile. For these reasons, special consideration should be given in these instances to mark the pile ends (top and bottom) so that the pile is installed in its correct orientation.

2.7 Structural steel

For exceptionally hard pile-driving conditions, particularly for bearing on rock or penetration through hard strata, special treatment of the pile tip may be advantageous. This may take the form of added spiral or other reinforcement at the tip, steel shoes, or points. In some cases, a structural steel H-pile is used to provide a permanent, load-bearing structural tip, commonly called a *stinger*, on the end of a prestressed concrete pile. This material must withstand driving, so it should meet the requirements for steel H-pile sections (ASTM A572 Grade 50). The thickness of any steel in a pile tip should not be less than ⅜ in. (9.53 mm).

The suitability of the steel for welding should be predetermined because some producers prefer to cast the piles with just an embedded plate or only a short portion of the tip extending outside the pile. This preserves the casting capacity of the bed and simplifies the precast-concrete-pile portion of the fabrication. The remainder of the specified length of the tip is then welded onto the plate or the protruding portion after the pile is removed from the bed.

Headed steel studs (conforming to ASTM A29) on each side of the web in the embedded portion of the steel stinger or deformed bar anchors, with or without a stinger plate, may be required in some cases for the anchorage of the embedded portion of the steel tip into the pile. The use and installation of headed steel studs should conform to the requirement of AWS D1.1.

2.8 Grout

The need for grout in prestressed concrete piles is not common, and is generally limited to the grouting of dowels in pile heads or the bonding and protection of prestressing strands used to join sections of cylinder piles. The former application involves piles that are field-spliced with dowels in oversized holes or to provide pile-to-pile cap connections using grouted dowels. The latter case is common with large-diameter hollow cylinder piles available in several regions of the country.

Cement grout, where used in prestressed concrete piles, should conform to ASTM C1107 and comprise materials that conform to the requirements stipulated herein for cement, sand, admixtures, and water. Approved expanding admixtures or expansive cements may be used. Some expanding admixtures contain calcium chloride and should not be used. Neat cement grout is frequently used to grout dowels in pile heads. PTI M55.1-12, *Specification for Grouting of Post-Tensioned Structures*, should be consulted for information regarding grouting of prestressing tendons when designing segmented prestressed concrete piles. Epoxy grout may also be used in some applications.

2.9 Anchorages

Anchorage fittings for post-tensioning assemblies should conform to the requirements of ACI 318-14, section 25.8.[2]

Section 3—Design

This section covers important design considerations and presents design aids for prestressed concrete piling. Load-capacity determination procedures, structural design and detailing procedures, and special considerations for compression, tension, flexure, shear, and combined actions are discussed. Detailed provisions for prescriptive- and performance-based design are included, and reference is made to PCI's free software program for developing axial-moment interaction diagrams for the ultimate capacity of commonly used pile sizes of varying concrete strengths and effective prestress levels.

3.1 Load capacity of individual piles

The failure of an individual pile or a pile group in compression can occur when the applied load on the pile exceeds both the ultimate frictional resistance of the soil along the sides of the pile and the resistance of the bearing strata underneath the pile tip. The sum of these two resistance mechanisms is referred to as the ultimate bearing capacity of the pile. In most cases, the ultimate bearing capacity of the pile is calculated to be less than the structural strength of the pile as determined using the methods presented in this section. The failure of an individual pile or a pile group in tension can occur when the applied load on the pile exceeds just the ultimate frictional resistance of the soil along the sides of the pile.

3.1.1 Load capacity of individual piles: preliminary design considerations During the preliminary design phase of any project, the project geotechnical engineer or engineer of record should make recommendations concerning pile type, pile size, drivability, test pile locations, and the need for static or dynamic testing.

Before installation, the project geotechnical engineer or engineer of record should review pile shop drawings in conjunction with the pile driver's proposed installation method, including hammer size, to ensure that the hammer used during installation and the proposed installation method are consistent with pile's properties, which include size, strength, and weight, and with the soil conditions of the site.

During construction, the project geotechnical engineer or engineer of record should be involved in pile testing operations, the interpretation of test results, and the evaluation of production pile-driving observations. In conjunction with the engineer of record, the geotechnical engineer should be prepared to modify the design, where necessary, based on the resulting test data and construction observations. Changes to pile lengths and project driving operations in the field should be anticipated.

3.1.2 Load capacity of individual piles: compression Consistent with standard practice for lightly loaded building pile applications, this document permits the use of pile installations without the added expense of load testing when the allowable compressive axial load for the pile does not exceed 40 tons (356 kN). In such cases, approved driving formulas are sufficient for specifying a pile's allowable load. It should be noted that pile-driving formulas have never really been considered accurate and they are based on several assumptions that may be unconservative.[5] Furthermore, pile dimensions, allowable pile loads, and pile-driving-hammer rated energies have increased substantially over recent years, making the use of pile-driving formulas even less applicable.

For allowable compressive axial loads greater than 40 tons (356 kN), wave equation analysis may be used in conjunction with load tests (for some small percentage of piles, as discussed in section 3.1.5) to the degree deemed necessary by the project geotechnical engineer or engineer of record to estimate driving conditions and to determine the appropriate allowable compressive load for the pile, respectively. Unlike pile-driving formulas, wave equation analysis includes a realistic model of important parameters such as pile, soil, and hammer characteristics.

Allowable compressive axial loads are most commonly determined from load tests. Static (ASTM D1143), rapid (ASTM D7383), and dynamic (ASTM D4945) procedures are permitted. Dynamic testing is a real-time blow-by-blow evaluation of the pile's capacity, hammer energy transfer, internal stresses, and pile integrity. The allowable axial load is determined using the data obtained at the end of the pile installation or data from a restrike applied sometime after installation (thereby accounting for capacity gain with time or setup). The signal-matching software CAPWAP or similar program should be used to analyze the dynamic testing data to dynamic soil parameters, driving stresses over the length of the pile, the resistance distribution, and the total pile-bearing capacity.

When compressive load testing is used, at least one prestressed concrete pile must be load tested in each area of uniform subsoil conditions. Unless higher values are permitted by the project geotechnical engineer or engineer of record, 50% of the ultimate compressive axial load capacity from the test serves as the upper bound allowable compressive load for the test pile. The actual specified allowable compressive load must be based on full consideration of total and differential settlement effects on the supported structure as expected under design loading.

Prestressed concrete piles are tested during the driving process. As such, where the observed driving resistance (blows per foot or blows per inch) at the end of installation or beginning of restrike is greater than or equal to the test pile-driving resistance, each pile may be assigned the same allowable compressive axial load as the test pile, as long as the same hammer is used and the installation depth is nearly constant.

3.1.3 Load capacity of individual piles: tension The allowable uplift capacity of prestressed concrete piles may be determined by load testing (ASTM D3689) or by approved analysis methods that use a factor of safety greater than or equal to three (unless a lower factor of safety is permitted with the analysis method by the project geotechnical engineer or engineer of record). The upper bound for the allowable uplift load is the ultimate compressive load capacity divided by a safety factor of three (that is, the upper bound uplift allowable load is approximately two-thirds of the upper bound compressive allowable load, which is based on a factor of safety of two). A special exception exists for wind and seismic uplift loading, in which case the minimum factor of safety is reduced to 2 when based on analysis methods, and to 1.5 when based on load testing.

3.1.4 Load capacity of individual piles: lateral The allowable lateral load on a prestressed concrete pile or group of prestressed concrete piles can be determined using analysis methods or lateral load tests (ASTM D3966). When prescriptive or force-based design procedures are used, the allowable load must not be taken as greater than 50% of the load causing a deflection of 1 in. (25 mm) at the top of the foundation element or the ground surface, whichever is lower, unless the engineer of record determines that larger deflections are acceptable. When performance-based design procedures are used, no lateral deflection limit is specified, but overall structural stability and local pile stability must be maintained with full consideration of pile buckling, second-order effects, and the presence of liquefied soil layers when considering seismic design load combinations of the governing code.

3.1.5 Test piles

Test piles are permanent piles driven in advance of production piles on some projects. The driving of test piles not only proves the drivability of the pile, but test piles are also used to determine the length required for production piles, ensure the proposed driving equipment can install the piling without damage, and provide proof that a certain pile, driven to a specific depth in accordance with specific driving criteria, can safely support the load specified in the project plans.

The total number of piles that should be installed as test piles before constructing production piles should be established by the project geotechnical engineer or engineer of record. This number (or percentage) and required locations are both project- and site-specific. For uniform site conditions where piles are also designed using the prescriptive design approach, as described later in this section, the recommended number of test piles may be minimal. Additional test piles may be required for nonuniform site conditions or where performance-based design procedures, as defined later in this section, are used.

3.2 Before service loads and stresses

3.2.1 Effective prestress The effective prestress for a pile is defined as the compressive stress in the pile after all losses have occurred. Losses may be assumed equal to 30,000 psi (210 MPa) per strand, unless a more accurate estimate of losses is required. **Table 3.1** presents minimum effective prestress values as a function of pile length. These values are based on experience that has shown that when the minimum tabulated values are not obtained, adequate crack control during handling and installation activities may not be realized.

3.2.2 Handling stresses Flexural stresses should be investigated for all conditions of handling (that is, lifting from casting beds, storage, transportation, and pitching from horizontal to vertical). Stress analysis should be based on the weight of the pile plus a 50% allowance (or greater depending on individual conditions) for impact, with tensile stresses limited to $6\sqrt{f'_{ci}}$ (note that in this equation, f'_{ci} is defined as the concrete strength at the time of handling operations). A pile should be handled only at clearly marked support points. Regardless of the method used to calculate handling stresses, or the allowable stress limits, the desired result is the installation of an undamaged pile. Chips and minor spalls of the concrete, which may be created during handling, pitching, and driving, and which do not impair performance of the pile, should be allowed. Concrete chips that

Table 3.1. Minimum effective prestress for piles

Pile length L	Minimum effective prestress
L < 30 ft (9.14 m)	400 psi (2.76 MPa)
30 ft (9.14 m) ≤ L < 50 ft (15.2 m)	550 psi (3.79 MPa)
L ≥ 50 ft (15.2 m)	700 psi (4.83 MPa)

expose steel reinforcement or otherwise impair performance and affect pile service life must be repaired.

Piles are generally stripped from their casting forms using embedded lifting eyes consisting of unstressed seven-wire strands bent to shape in the plant. The number of lifting eyes depends on the length and size of the pile. Lifting eyes are spaced to provide the best use of the pile cross section considering the maximum positive and negative moments. In many cases, the maximum moment is equal to the cantilever moment at the lifting point nearest the end. However, sometimes lift points are selected based on available handling gear or equipment, particularly when shipping on rocker bunks, or tripping the piles up for driving. Either way, and for all cases, as long as the maximum moment (positive or negative) plus impact plus the effective prestress force does not result in actual stresses that exceed the allowable tension or compression stresses as provided in **Table 3.2**, the design for handling is satisfactory. Pile lengths for a given lifting scheme should be limited so that the allowable stresses in Table 3.2 are not exceeded for the given number of lifting locations.

3.2.3 Driving stresses The most severe stress conditions a pile will endure typically occur during driving. Driving stresses are a complex function of pile and soil properties, and are influenced by driving resistance, hammer weight and stroke, cushioning materials, and other factors. Such stresses are alternately compression and tension and, under certain conditions, could exceed the ultimate strength of the pile cross section in either tension or compression. For piles longer than about 50 ft (15.2 m), tensile stresses can occur during soft or irregular driving. For shorter piles, tensile driving stresses that could damage the pile usually do not occur. Table 3.1 lists minimum effective prestress levels to resist driving stresses; Table 3.2 presents driving stress limits that should be compared with given driving loads. Refer to section 5 for a more complete discussion of driving stresses.

3.3 Service load stresses

Prestressed concrete piles are required to meet certain allowable stress criteria for serviceability. Service loads in prestressed concrete piles are either permanent (dead load and possibly some portion of the live load), repetitive (live load), or transient (environmental, construction load). Because more stringent limits apply to repetitive loads than to transient loads, it is important to further clarify the difference between the two. In the context of this report, repetitive loading is the application of some form of significant live load that could qualify as either a cyclic or fatigue-type load under the governing code and that is expected to occur at least 10,000 times during the design life of the structure. Possible repetitive live-load applications include, but are not limited to, vehicular loads, machine vibration live loads, high-occupancy area live loads, and live-load applications in industrial facilities. Transient loading may be of significant magnitude, but the occurrence of this type of load and the duration of the application are both limited. Possible transient load applications include, but are not limited to, seismic load, wind load, snow load, and vessel mooring/

Table 3.2. Handling and driving stress limits for prestressed concrete piles—concrete

Description	Stress limit
Handling stresses:	
Compute stresses using a 50% increase in pile weight for impact. The actual stress in the pile includes the effective prestress f_{pc} after losses, as applicable. (Note: f'_{ci} should be concrete strength at time of handling operations if before 28 days.)	
Tension	$6\sqrt{f'_{ci}}$
Compression	$0.60f'_{ci}$
Driving stresses:	
The actual stress in the pile does not include the effective prestress f_{pc} after losses. (Note: actual stresses are based only on given driving loads.)	
Compression	$0.85f'_c - f_{pc}$
Tension—normal environments*	$3\sqrt{f'_c} + f_{pc}$
Tension—severely corrosive environments	f_{pc}

Note: f'_c = compressive strength of concrete.

* The allowable tensile stress is limited to f_{pc} for segmentally constructed post-tensioned piles.

berthing/impact loads associated with a variety of environmental conditions.

Service load stresses occur after driving and are the result of several types of loadings or combinations of loadings. In the context of this report, piles can be designed using allowable stress design (ASD) procedures under the following generalized cases:

- axially loaded compression piles (section 3.3.1)
- axially loaded tension piles (section 3.3.2)
- combined axial and bending piles (section 3.3.3)

As will be discussed in section 3.7, load combinations involving seismic, wind, and lateral earth pressure that induce flexural behavior of piles require design in accordance with the strength design (load and resistance factor design [LRFD]) provisions presented in section 3.6. The combined axial and bending piles case presented previously refers to bending moments caused by construction loading, temporary loading, accidental eccentricities, or any other similar condition.

It should be noted that the ASD procedure discussed previously is in addition to any serviceability requirements of the governing code for the overall structure. Traditionally, building codes limit overall system serviceability performance (for example, deflections, vibrations, crack control) above the foundation system and the foundation system is usually considered as a fixed base for these purposes. On the contrary, sometimes machinery and similar elements are directly supported by piling systems and serviceability provisions are paramount. Codes related to the bridge and pier/wharf industries also have serviceability limits, such as those related to lateral drift of the structure which must be considered as part of the overall structural design process.

3.3.1 Axially loaded compression piles The most common type of pile is the one designed to resist predominantly axial compression loads. Compression piles can be bearing piles, friction piles, or combined bearing and friction piles. Bearing piles are used to transfer structure dead and live loads through an unsuitable medium to a firm bearing material below. Friction piles transfer the axial load delivered to the pile via skin friction between the soil and pile side surface or "skin." Compression piles must be capable of safely supporting design loads without buckling. Standard practice (and modern codes for all types of structures) allows for any soil type, except fluid soils and liquefied soils, to be considered as providing sufficient lateral restraint to prevent pile buckling. The reader can refer to ACI 543-12[3] and the American Association of State Highway and Transportation Officials' 2014 *AASHTO LRFD Bridge Design Specifications*[6] for more information on methods for computing the effective unbraced length of prestressed concrete piles. **Table 3.3** presents the allowable concrete stresses for these piles.

Note that the focus of this section is on axially loaded compression piles. For different structure types and applicable codes, standard practice for lateral analyses in liquefied soils can vary. In some cases, for example, standard practice for the lateral analyses of piles in liquefied soils is to assign some resistance (or range of resistances) to the liquefied layer. The same is true for stability analyses when designing for lateral loads. A very low strength/stiffness is typically applied to a liquefied soil, but neglecting all strength/stiffness may be overly conservative for some limit states and possibly unconservative for others. In all cases, the governing code and project geotechnical engineer should provide stiffness assumptions for liquefied soil layers.

3.3.2 Axially loaded tension piles Occasionally, piles must be designed to resist uplift loads. The magnitude of the uplift, or tensile stress, should be analyzed in relation to the effective prestress in the pile, and whether the loads are sustained or transient. Piles loaded in tension are not subject to buckling. Proper head connection details should be designed to transfer axial loads from the structure past the point of full prestressing force transfer. In piles subject to pure axial tension, the level of prestress should be high enough to ensure that concrete tensile stress does not occur under permanent or repetitive loads. Table 3.3 presents the allowable concrete stresses for these piles.

3.3.3 Combined axial and bending piles A pile under the effects of combined loading is one that is subjected to a bending moment and an axial load. In this section, bending moments caused by construction loading, temporary loading, accidental eccentricities, or any other similar condition permit the use of ASD procedures. These stresses may be permanent or temporary and may act separately or simultaneously. The axial load may be tensile or compressive and may be reversed a number of times during the life of the pile. As will be discussed in section 3.7, load combinations involving seismic, wind, and lateral earth pressure that induce flexural behavior of piles require design in accordance with the strength design provisions presented in section 3.6. Axial-moment interaction diagrams offer a simple approach to the design of such piles. Piles subjected to a tensile load and bending moment are not susceptible to buckling and allowable axial and flexural stresses should conform to those prescribed in Table 3.3.

3.4 Allowable service load stresses in steel strands

Table 3.4 shows the allowable recommended steel strand service stresses.

3.5 Allowable load for compression-only pile

Compression-only piles are commonly used to transfer dead and live loads from the structure to a firm bearing material below. Equation (3.1) provides the allowable axial service capacity for concentrically loaded prestressed concrete piles with full lateral support provided by the surrounding soil.

Table 3.3. Service stress limits for prestressed concrete piles—concrete

Description	Stress limit
Service condition stresses (axial, axial + bending; includes effective prestress)	
Compression—all loads	$0.60f_c'$
Compression—dead lead	$0.45f_c'$
Compression (fatigue only)—(½ dead load + ½ effective prestress + live load)*	$0.40f_c'$
Service condition stresses (axial + bending; includes effective prestress)	
Tension—normal environments	$6\sqrt{f_c'}$
Tension—severely corrosive environments	$3\sqrt{f_c'}$
Service condition stresses (uniform axial tension; includes effective prestress)	
Tension—permanent and repetitive	0
Tension—transient	$6\sqrt{f_c'}$

Note: f_c' = compressive strength of concrete.

* This special case loading includes only 50% of the calculated effective prestress when calculating the actual stress in the pile.

$$P_a = A_g(0.33f_c' - 0.27f_{pc}) \tag{3.1}$$

where

P_a = allowable service level axial load

A_g = gross cross-sectional area of pile

f_c' = 28-day compressive strength of concrete

f_{pc} = effective prestress in the pile after losses

Equation (3.1) is intended exclusively 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, such as those resulting from construction loading, temporary loading, accidental eccentricities, or any other similar condition. For other load combinations, where piles are subjected to large bending moments, bending moments caused by seismic or wind loading, or for piles with long, unsupported lengths, the piles must be designed in accordance with the strength design provisions presented in section 3.6.

Figure 3.1 and **Table 3.5** provide section properties and allowable concentric service loads based on a 700 psi (4.83 MPa) effective prestress and concrete strengths of 5000 to 10,000 psi (35 to 68.9 MPa).

The effect of negative skin friction or downdrag must also be considered when evaluating the service load behavior of a pile.

3.6 Strength design of prestressed concrete piles

Strength design methods are required for all cases where piles are subjected to lateral loading resulting in larger moments than those caused by accidental eccentricity or associated with temporary loads or construction and installation meth-

Table 3.4. Service stress limits for prestressed concrete piles—steel strands

Description	Stress limit
Prestressing steel:	
Due to temporary jacking force, but not greater than the maximum value recommended by the manufacturer of the steel	$0.8f_{pu}$
Pretensioning tendons immediately after transfer, or post-tensioning tendons immediately after anchoring	$0.74f_{pu}$
Tension due to transient loads	n.a. (concrete governs; see Table 3.3)
Effective prestress	$0.6f_{pu}$ or $0.8f_{py}$, whichever is smaller
Unstressed prestressing steel	30,000 psi (207 MPa)

Note: f_{pu} = ultimate strength of prestressing reinforcement; f_{py} = yield strength of prestressing reinforcement; n.a. = not applicable.

ods. Note that strength design methods are also required when lateral support is not provided over the full height of the pile.

Depending on the type of structure being designed and the governing code, strength design can be referred to as load factor design (LFD) or load and resistance factor design (LRFD). Strength design remains the standard method used to design reinforced concrete members.

Strength design methods compare factored (amplified) design loads with factored (reduced) member capacities (strengths). Load and resistance factors are provided by the governing code for the subject structure and are not presented here. The combined effect of axial loads and moments is important for piles. The effect of slenderness, which may amplify the applied moments, may sometimes be significant because piles may be long, slender members. Shear strength must also be considered, but it rarely governs the design of piles.

PCI's free software program for developing axial-moment interaction diagrams can produce customized interaction diagrams that include slenderness effects. Specific pile characteristics, such as pile type and size, concrete strength, pretensioning strand size and pattern, and slenderness ratio, can be entered. The program can also be used to compute lifting points for precast concrete piles.

3.7 Special design provisions for prestressed concrete piles

3.7.1 Transverse reinforcement Transverse reinforcement must be provided along the entire length of all prestressed concrete piles to confine the concrete within the core of the pile and to control any longitudinal cracks that may form during handling, driving, or under design load conditions. When inelastic behavior of piles is a design consideration, increased levels of transverse reinforcing may be required at certain locations along the length of the pile to ensure stable inelastic hinging mechanisms can occur.

The concrete confining effect of transverse reinforcement is well established in the literature (see Mander et al.[7] for discussion). Both circular and square mild transverse reinforcement work together with longitudinal strands to restrain the lateral expansion of the concrete under combined axial compressive stresses that result from the effects of flexural and axial demands. As a result, the confined core area is stable under cyclic loading and is stronger (resists larger compressive stresses) and less brittle (resists much larger compressive strains before failure) than the unconfined concrete outside of the core area.

Proper application of the transverse reinforcement provisions contained in this section requires a clear understanding of the key terms related to confined concrete. **Figure 3.2** shows important terms for both circular and square transverse reinforcement as typical for precast concrete piles. Note that the dimensions D' and h_c are defined as center-to-center dimensions for both circular and square transverse reinforcement for consistency. The 2015 *International Building Code*[8] uses out-to-out dimensions for circular transverse reinforcement (that is, spiral) and center-to-center dimensions for square ties. In Fig. 3.2, A_{sp} is the cross-sectional area of the circular transverse reinforcement, while A_{sh} is the total cross-sectional area of square transverse reinforcement in each direction. For example, in Fig. 3.2, A_{sh} is simply two times the cross-sectional area of the transverse reinforcing bar being used for confinement.

For circular transverse reinforcing bars, the volumetric ratio ρ_s of the transverse reinforcement is the volume of transverse reinforcement within the concrete core volume divided by the gross volume of the concrete core being considered. For example, Eq. (3.2) gives the volumetric ratio for the circular transverse reinforcement of spacing s shown in Fig. 3.2(a):

$$\rho_s = \frac{A_{sp}\left(\pi D'\right)}{\left(\pi\left(D'\right)^2/4\right)s} = \frac{4A_{sp}}{D's} \tag{3.2}$$

Similarly, Eq. (3.3) gives the volumetric ratio ρ_s for the square transverse reinforcement of spacing s shown in Fig. 3.2(b):

$$\rho_s = \frac{A_{sh}\left(2h_c\right)}{h_c^2 s} = \frac{2A_{sh}}{h_c s} \tag{3.3}$$

Recalling that A_{sh} is the total area of transverse reinforcement *in each direction*, the volumetric ratio for the two cases results in the same value when the same size transverse bar is used

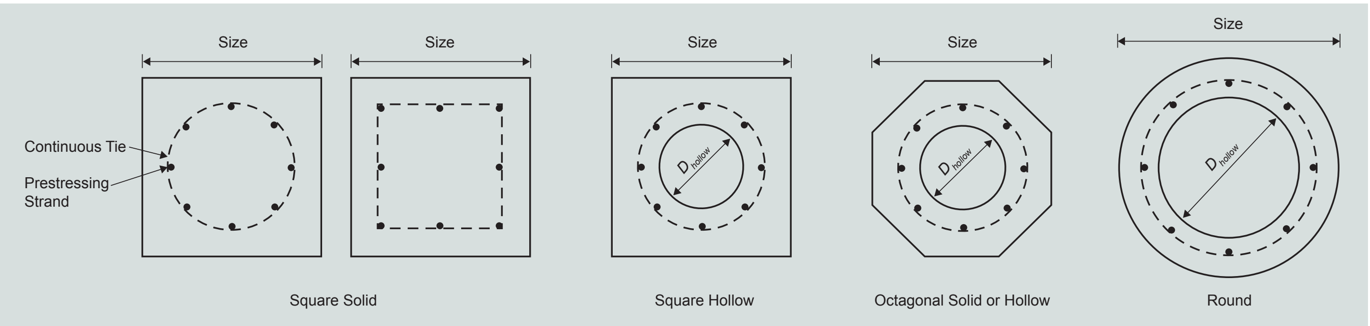

Figure 3.1. Section properties of typical prestressed concrete piles.

Table 3.5a. Allowable service loads of typical prestressed concrete piles (IP units)

Size, in.	D_{hollow}, in.	Section properties*						Allowable concentric service load, ton†					
		Area, in.²	Weight, lb/ft	Moment of Inertia, in.⁴	Section modulus, in.³	Radus of gyration, in.	Perimeter, ft	f'_c					
								5000 psi	6000 psi	7000 psi	8000 psi	9000 psi	10,000 psi
Square piles													
10	Solid	100	104	833	167	2.89	3.33	73	89	106	122	139	156
12	Solid	144	150	1728	288	3.46	4.00	105	129	152	176	200	224
14	Solid	196	204	3201	457	4.04	4.67	143	175	208	240	273	305
16	Solid	256	267	5461	683	4.62	5.33	187	229	271	314	356	398
18	Solid	324	338	8748	972	5.20	6.00	236	290	344	397	451	504
20	Solid	400	417	13,333	1333	5.77	6.67	292	358	424	490	556	622
20	11	305	318	12,615	1262	6.43	6.67	222	273	323	373	424	474
24	Solid	576	600	27,648	2304	6.93	8.00	420	515	610	705	801	896
24	12	463	482	26,630	2219	7.58	8.00	338	414	491	567	644	720
24	14	422	439	25,762	2147	7.81	8.00	308	377	447	517	587	656
24	15	399	415	25,163	2097	7.94	8.00	291	357	423	488	555	621
30	18	646	672	62,347	4157	9.82	10.00	471	578	685	791	898	1005
36	18	1042	1085	134,815	7490	11.38	12.00	761	933	1105	1276	1449	1621
Octagonal piles													
10	Solid	83	85	555	111	2.59	2.76	60	74	88	101	115	129
12	Solid	119	125	1134	189	3.09	3.31	86	106	126	145	165	185
14	Solid	162	169	2105	301	3.60	3.87	118	145	172	198	225	252
16	Solid	212	220	3592	449	4.12	4.42	154	189	224	259	295	330
18	Solid	268	280	5705	639	4.61	4.97	195	240	284	328	373	417
20	Solid	331	345	8770	877	5.15	5.52	241	296	351	405	460	515
20	11	236	245	8050	805	5.84	5.52	172	211	250	289	328	367
22	Solid	401	420	12,837	1167	5.66	6.08	292	359	425	491	558	624
22	13	268	280	11,440	1040	6.53	6.08	195	240	283	328	373	417
24	Solid	477	495	18,180	1515	6.17	6.63	348	427	506	584	663	742
24	15	300	315	15,696	1308	7.23	6.63	219	268	318	368	417	467
Round piles													
36	26	487	507	60,007	3334	11.10	9.43	355	436	516	596	677	758
42	32	581	605	101,273	4823	13.20	11.00	424	520	616	712	808	904
48	38	675	703	158,222	6592	15.31	12.57	493	604	715	827	939	1050
54	44	770	802	233,373	8643	17.41	14.14	562	689	816	943	1071	1198
66	54	1131	1178	514,027	15,577	21.32	17.28	826	1013	1199	1386	1573	1759

* Form dimensions may vary with producers, with corresponding slight variations in section properties.

† Allowable loads based on f_{pc} = 700 psi. Check local producer for available concrete strengths.

Table 3.5b. Allowable service loads of typical prestressed concrete piles (metric units)

Size, mm	D_{hollow}, mm	Section properties*						Allowable concentric service load, kN†					
		Area, mm²	Weight, kg/m	Moment of inertia, mm⁴	Section modulus, mm³	Radius of gyration, mm	Perimeter, m	f'_c					
								34.5 MPa	41.4 MPa	48.3 MPa	55.2 MPa	62.1 MPa	68.9 MPa
Square piles													
254	Solid	64,520	155.02	3.47E+08	2.74E+06	73.4	1.02	649	792	943	1085	1237	1384
305	Solid	92,909	223.22	7.19E+08	4.72E+06	87.9	1.22	934	1148	1352	1566	1781	1993
356	Solid	126,459	303.83	1.33E+09	7.49E+06	102.6	1.42	1272	1557	1850	2135	2425	2712
406	Solid	165,171	396.84	2.27E+09	1.12E+07	117.3	1.63	1664	2037	2411	2793	3167	3543
457	Solid	209,045	502.26	3.64E+09	1.59E+07	132.1	1.83	2100	2580	3060	3532	4008	4484
508	Solid	258,080	620.07	5.55E+09	2.18E+07	146.6	2.03	2598	3185	3772	4359	4948	5535
508	279	196,786	472.73	5.25E+09	2.07E+07	163.3	2.03	1975	2429	2874	3318	3773	4221
610	Solid	371,635	892.90	1.15E+10	3.78E+07	176.0	2.44	3737	4582	5427	6272	7125	7971
610	305	298,728	717.56	1.11E+10	3.64E+07	192.5	2.44	3007	3683	4368	5044	5728	6407
610	356	272,274	654.24	1.07E+10	3.52E+07	198.4	2.44	2740	3354	3977	4599	5220	5840
610	381	257,435	618.93	1.05E+10	3.44E+07	201.7	2.44	2589	3176	3763	4341	4936	5522
762	457	416,799	1000.63	2.60E+10	6.81E+07	249.4	3.05	4190	5142	6094	7037	7991	8940
914	457	672,298	1614.50	5.61E+10	1.23E+08	289.1	3.66	6770	8300	9831	11,352	12,890	14,420
Octagonal piles													
254	Solid	53,552	126.49	2.31E+08	1.82E+06	65.8	0.84	534	658	783	899	1027	1149
305	Solid	76,779	186.02	4.72E+08	3.10E+06	78.5	1.01	765	943	1121	1290	1472	1647
356	Solid	104,522	251.50	8.76E+08	4.93E+06	91.4	1.18	1050	1290	1530	1761	2004	2242
406	Solid	136,782	327.40	1.50E+09	7.36E+06	104.6	1.35	1370	1681	1993	2304	2623	2934
457	Solid	172,914	416.69	2.37E+09	1.05E+07	117.1	1.52	1735	2135	2527	2918	3315	3709
508	Solid	213,561	513.42	3.65E+09	1.44E+07	130.8	1.68	2144	2633	3123	3603	4095	4581
508	279	152,267	364.60	3.35E+09	1.32E+07	148.3	1.68	1530	1877	2224	2571	2919	3266
559	Solid	258,725	625.03	5.34E+09	1.91E+07	143.8	1.85	2598	3194	3781	4368	4961	5549
559	330	172,914	416.69	4.76E+09	1.70E+07	165.9	1.85	1735	2135	2518	2918	3315	3709
610	Solid	307,760	736.64	7.57E+09	2.48E+07	156.7	2.02	3096	3799	4502	5196	5901	6601
610	381	193,560	468.77	6.53E+09	2.14E+07	183.6	2.02	1948	2384	2829	3274	3711	4152
Round piles													
914	660	314,212	754.50	2.50E+10	5.46E+07	281.9	2.88	3158	3879	4591	5302	6024	6739
1067	813	374,861	900.34	4.22E+10	7.90E+07	335.3	3.36	3772	4626	5480	6334	7187	8040
1219	965	435,510	1046.18	6.59E+10	1.08E+08	388.9	3.83	4386	5373	6361	7357	8350	9341
1372	1118	496,804	1193.51	9.71E+10	1.42E+08	442.2	4.31	5000	6130	7259	8389	9525	10,656
1676	1372	729,721	1753.06	2.14E+11	2.55E+08	541.5	5.27	7348	9012	10,667	12,330	13,991	15,651

* Form dimensions may vary with producers, with corresponding slight variations in section properties.

† Allowable loads based on f_{pc} = 4.826 MPa. Check local producer for available concrete strengths.

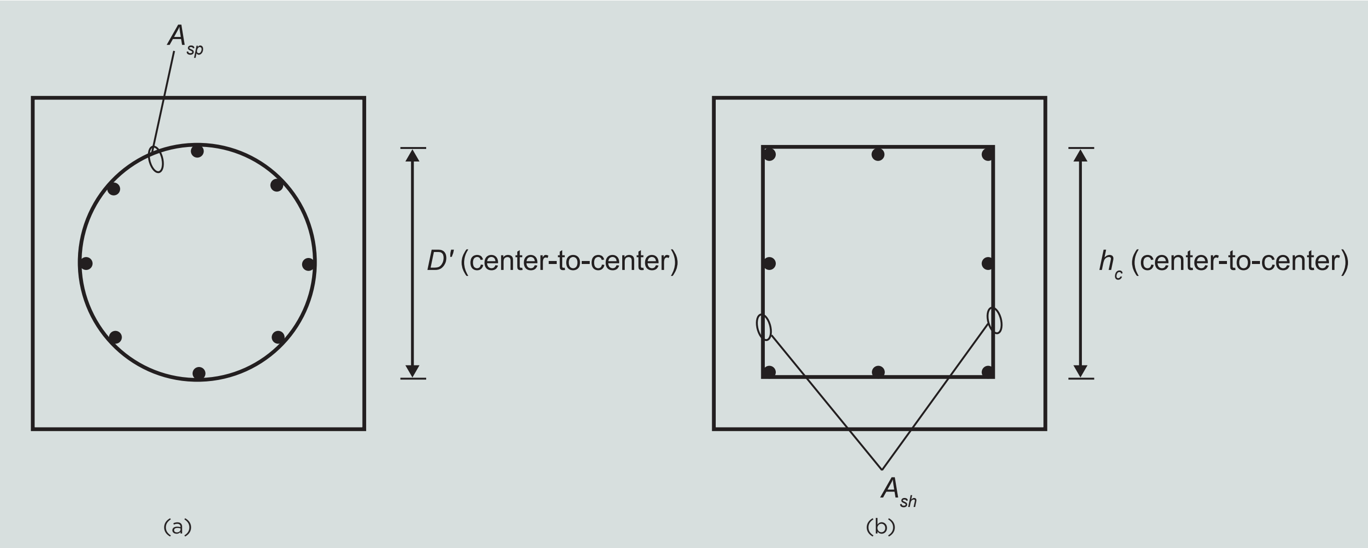

Figure 3.2. Key transverse reinforcement terms for (a) circular and (b) square configurations. Note: A_{sp} = cross-sectional area of the circular transverse reinforcement; A_{sh} = total cross-sectional area of square transverse reinforcement in each direction; D' = center-to-center dimension for circular transverse reinforcement; h_c = center-to-center dimension for square transverse reinforcement.

at the same spacing and D' is set equal to h_c. For this special case (that is, for the same s and $D' = h_c$), the volumetric ratio of transverse reinforcement *in each direction* for the square transverse reinforcement case should be exactly half of that required for the circular transverse reinforcement case. However, Mander et al.[7] and others (for example, Priestly et al.[9]) discuss in detail that square transverse reinforcement is not nearly as good at confining concrete as circular transverse reinforcement. They suggest that in terms of confinement, circular transverse reinforcement is nearly 95% effective compared with 75% for the square transverse reinforcement case. As such, a 127% (95/75) increase in transverse reinforcement is required whenever square patterns are being used. Although not specifically a transverse reinforcement provision explicitly stated as applicable to piles, note that for square configurations, most codes require cross-ties be provided when the center-to-center spacing of longitudinal steel (strands) is greater than 6 in. (152 mm) as measured from a laterally supported (corner) bar.

As previously mentioned, typical seismic design detailing includes increased transverse reinforcement at certain locations along the length of the pile. The added transverse reinforcement increases the available ductility of the pile, which enables the pile to withstand significant displacements and inelastic curvatures. In the case of cover spalling due to high seismic demands and inelastic pile curvature, transverse reinforcement not only ensures that the confined concrete can resist compressive strains significantly greater than 0.003, but it also prevents buckling of compressed longitudinal reinforcing bars and tendons at large deformations and provides shear strength that supplements the shear strength of the concrete alone.

Many piles are not in moderate to high seismic areas and specifying W3.4 wire spiral reinforcement (W4.0 for larger piling) surrounding the strands is common. The spiral reinforcement is fabricated in a circular or square pattern, depending on the arrangement of the strands. Circular and square ties are also permitted, but are less commonly used. The spiral reinforcement pitch (longitudinal spacing between turns) is normally 6 in. (152 mm) for most of the pile length. The turns of the spiral reinforcement are more closely spaced at the ends of the pile to absorb energy and resist splitting forces during driving. Again, these typical details are used where piles are not anticipated to be subjected to significant seismic actions and therefore have no special detailing or design requirements.

There are two options for designing transverse reinforcement: prescriptive design and performance-based design. Both approaches are currently used in national codes and standards (for example, the 2015 *International Building Code*,[8] the 2014 *AASHTO LRFD Bridge Design Specifications*,[6] and the 2016 Marine Oil Terminal Engineering and Maintenance Standards[10]). Building codes have traditionally focused on prescriptive design, which attempts to ensure a degree of available ductility in piles that is directly related to the seismicity of the site. This approach, focused on essentially elastic pile curvatures, usually results in an overly conservative spiral reinforcement quantity that conflicts with the foundation demands and pile lateral displacement limits mandated in these same codes. Bridge codes and pier/wharf codes and standards have traditionally used prescriptive design approaches for areas of low to moderate seismicity, whereas designs in areas of high seismicity have been based on performance-based design. Both options require the same minimum transverse reinforcement as presented in section 3.7.1.1, irrespective of the site's seismic hazard level.

3.7.1.1 Minimum transverse reinforcement requirements for areas of low, moderate, and high seismicity

For all structures, and for both hollow and solid piles, pile prestressing strands must be enclosed in a spiral transverse reinforcement (or equivalent transverse reinforcement ties) equal to or exceeding the following minimum requirements:

For nominal pile sizes not greater than 24 in. (610 mm):

- Spiral wire must be a minimum of W3.4 (A_{sp} = 0.034 in.^{2} [21.9 mm^2]).
- Spiral pitch of 1 in. (25.4 mm) for five turns (approximately 6 in. [152 mm]) at both ends of pile, then a pitch of 3 in. (76.2 mm) for 16 turns at both ends, followed by a pitch of 6 in. for the remainder of the pile.

For nominal pile sizes greater than 24 in. (610 mm):

- Spiral wire must be a minimum of W4.0 (A_{sp} = 0.040 in.^{2} [25.8 mm^2]).
- Spiral pitch of 1½ in. (38.1 mm) for four turns (approximately 6 in. [152 mm]) at both ends of pile, then a pitch of 2 in. (50.8 mm) for 16 turns at both ends, followed by a pitch of 4 in. (102 mm) for the remainder of the pile.

Figures 3.3 and **3.4** show the minimum spiral reinforcement details. It should be noted that the minimum transverse reinforcement presented above and required for all prestressed concrete piling does exceed the minimum transverse reinforcement as specified for buildings in the 2015 *International Building Code*.[8] The 2015 *International Building Code* had relaxed PCI's previously published minimum transverse reinforcement requirements and provided minimum transverse spacings that were independent of the pile dimension.[1] The 2014 *AASHTO LRFD Bridge Design Specifications*[6] maintained the previous PCI minimum transverse reinforcement requirements. Although the exact reason for the 2015 *International Building Code*'s reduction is not known, providing one minimum amount for all pile sizes is reasonable for building piles because piles larger than 14 in. (360 mm) are not commonly used in building structures. For bridge structures, piles of all sizes greater than or equal to 18 in. (460 mm) are very common.

Although the minimum spiral reinforcement quantities presented above apply to hollow piles, hollow piles are not recommended for areas of moderate to high seismicity or in any application where pile ductility is a design consideration. Research is needed to more fully assess the behavior of hollow piles subject to seismic loads and to develop applicable design equations or construction strategies. The equations presented in this report for transverse reinforcement were developed specifically for solid pile sections. Extrapolation of their use for hollow pile sections is not recommended. Alternatively, hollow piles can be filled with concrete and detailed to perform well during seismic events. However, required detailing (depth of fill, augering requirements before fill, cleaning requirements, connection details, and the like) is code-specific and outside the scope of this report. Where hollow piles are filled with concrete for seismic performance, it is recommended that the performance-based design procedures presented in this report be used.

The top of hollow cylinder piles must be given special attention depending on the driving conditions encountered. Hard driving can cause large bursting forces at the driving end. Empirically, it has been found that providing about 1% spiral reinforcement in the top 12 in. (305 mm) of the pile results in the prevention of splitting of the pile head for diameters up to 54 in. (1370 mm).[1] However, because the selection of appropriate spiral reinforcement for cylinder piles is largely empirical and highly dependent on driving conditions, the ability to drive cylinder piles without cracking should be confirmed before pile production by conducting a pile-driving test using the planned driving equipment.

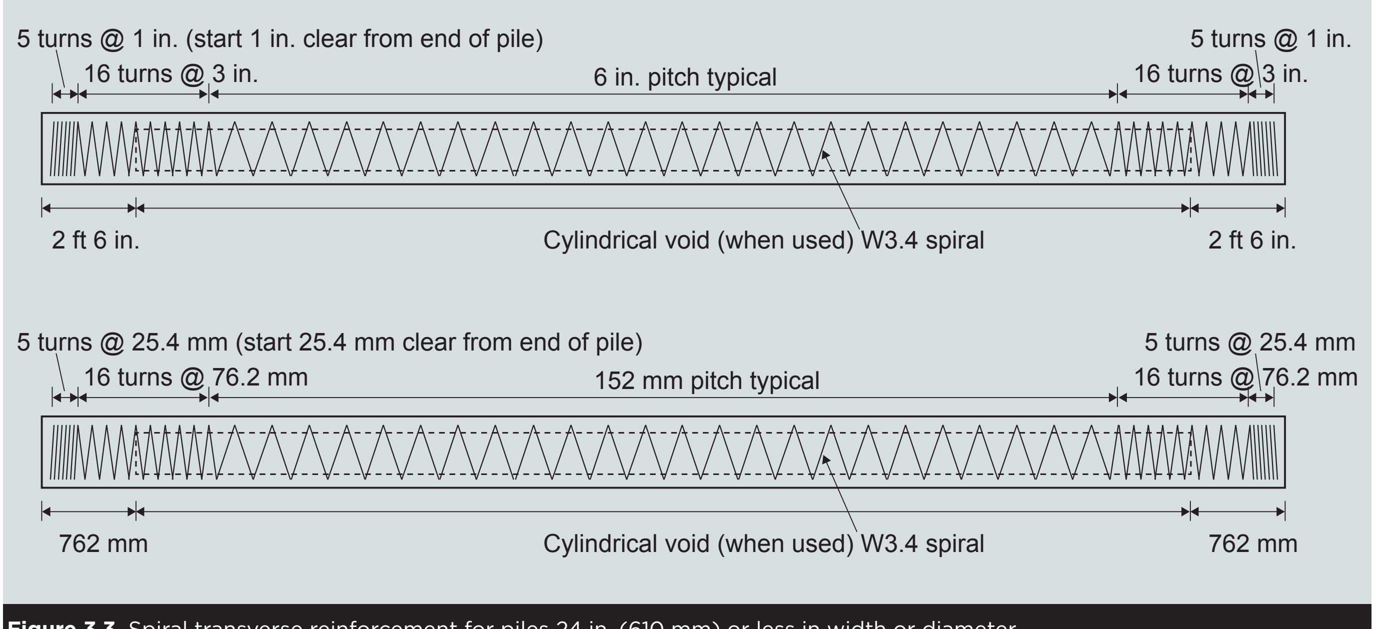

Figure 3.3. Spiral transverse reinforcement for piles 24 in. (610 mm) or less in width or diameter.

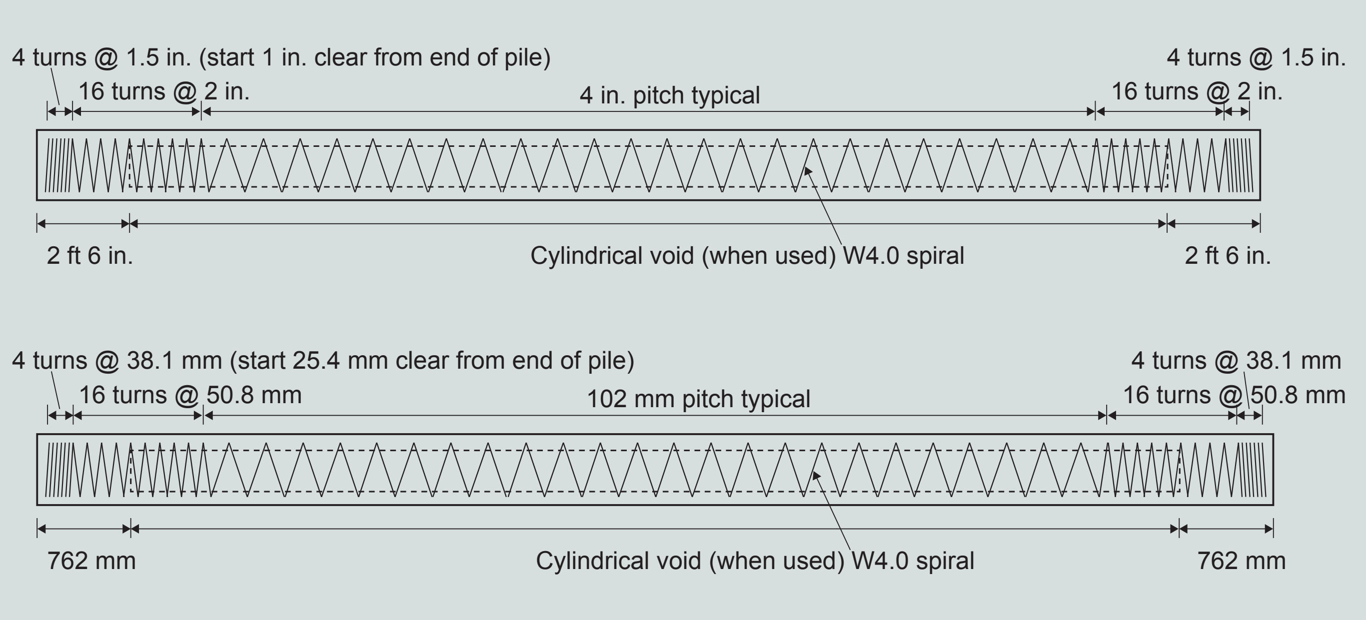

Figure 3.4. Spiral transverse reinforcement for piles greater than 24 in. (610 mm) in width or diameter.

Sections 3.7.1.2 and 3.7.1.3 present transverse reinforcement design and detailing provisions for prescriptive design and performance-based design, respectively. Section 3.7.1.2 first introduces the prescriptive design philosophy and then presents required design and detailing provisions for the following cases:

- areas of low seismicity—all applications
- areas of moderate seismicity—pile not considered part of the lateral force-resisting system
- areas of high seismicity—pile not considered part of the lateral force-resisting system
- areas of moderate seismicity—pile considered part of the lateral force-resisting system
- areas of high seismicity—pile considered part of the lateral force-resisting system

The wording "pile considered part of the lateral force-resisting system" has a very specific definition. The case is meant primarily for the bridge and pier/wharf industries where piles that extend significantly above grade are used intentionally as the primary lateral force-resisting system for the structure. Alternatively, this is primarily for cases where an *R*-value is used for pile bent-type behavior in prescriptive design or, when performance-based design is used, pile hinging is a purposely selected energy dissipation mechanism. For traditional building code applications using prescriptive design, the applicable case is usually "pile not considered part of the lateral force-resisting system."

Note that the definitions of low, moderate, and high seismicity are code-specific and not defined here. Earthquake return periods, risk factors, assumed soil conditions, and other assumptions result in design spectral accelerations that can be very different for one site depending on what code is being used to design the structure. Section 3.7.1.3 first introduces the performance-based design philosophy and then provides performance characteristics, experimentally and analytically proven modeling assumptions, and recommended detailing practices that can be used in conjunction with code-specific performance-based design procedures.

3.7.1.2 Prescriptive design of transverse reinforcement

For structures in areas of moderate to high seismicity, it is both practical and conservative, although sometimes possibly overly conservative, to detail transverse reinforcement using prescriptive equations that provide minimum volumetric ratios and pitch for the spiral reinforcement along specified lengths of the pile. Prestressed concrete piles can be considered part of or not part of the lateral force-resisting system. In most building applications, prestressed concrete piles are not considered part of the lateral force-resisting system (that is, pile demands are based on loads determined for the seismically damaged structure above the foundation and may or may not include system overstrength as discussed in this section). In building applications, such as those designed to meet the 2015 *International Building Code*,[8] pile ductility demand is not permitted by code, yet pile ductility capacity is considered paramount. For bridge (for example, flat slab bridges with pile bents) and pier and wharf structures, prestressed concrete piles are often considered part of the lateral force-resisting system. In these cases, pile ductility demand and capacity are typically used as part of the overall design philosophy. Note that the very descriptive subsection headings used in this section are intended to designate the level of seismicity and whether or not use of the section applies to piles used as part of the lateral force-resisting system.

The reader should note that the provisions presented in this section are intended to produce levels of ductility that are similar to ordinary reinforced concrete moment frames in areas of low seismicity, intermediate reinforced concrete moment frames in areas of moderate seismicity, and special reinforced concrete moment frames in areas of high seismicity, where all frame types are as defined by ACI 318-14 (sections 18.3, 18.4, and 18.7, respectively).[2] However, given the various approaches used for seismic design as presented in different governing codes and standards, this report does not attempt to provide or recommend response modification or *R*-values for the different performance levels, but defers to the code committees responsible for selecting these values consistent with each code's independent analysis approach.

3.7.1.2.1 Minimum transverse reinforcement requirements for areas of low seismicity: all applications

For structures in areas of low seismicity where the pile is or is not considered part of the lateral force-resisting system, pile prestressing strands must be enclosed in a transverse reinforcement that meets or exceeds the requirements of section 3.7.1.1.

3.7.1.2.2 Minimum transverse reinforcement requirements for areas of moderate seismicity: pile not considered part of the lateral force-resisting system

For structures in areas of moderate seismicity where the pile is not considered part of the lateral force-resisting system, pile prestressing strands must be enclosed in a transverse reinforcement that meets or exceeds the following minimum requirements.

The pile's assumed ductile region is defined here as:

- For fully embedded piles, the upper 20 ft (6.10 m) of the pile or the section from the top of the pile to the location of maximum moment below grade plus three pile dimensions, whichever is greater.
- For piles that extend above grade, the top of pile ductile region is the upper two pile dimensions measured from the underside of the pile cap and the bottom of pile ductile region is the larger of (a) the upper 20 ft (6.10 m) of the pile below grade and (b) the section of the pile from grade to the location of maximum moment below grade plus three pile dimensions (the effect of scour should be considered).

In this region, the volumetric ratio ρ_s shall not be less than the following.

For piles using a circular strand configuration with spiral reinforcement:

$$\rho_s = 0.04\left(\frac{f_c'}{f_{yh}}\right)\left(2.8+\frac{1.25P}{0.53f_c'A_g}\right) \quad (3.4)$$

where

ρ_s = spiral reinforcement index or volumetric ratio (vol. spiral/vol. core)

f_c' = specified compressive strength of concrete

f_{yh} = yield strength of spiral reinforcement ≤ 100,000 psi (700 MPa)

P = factored axial load on pile, as determined using all applicable load combinations that include the effect of seismic load E as contained in governing building, bridge or other structural codes; note that effective prestress is not used to determine P

A_g = pile cross-sectional area

For piles using a square strand configuration with square transverse reinforcement:

$$A_{sh} = 0.03sh_c\left(\frac{f_c'}{f_{yh}}\right)\left(2.8+\frac{1.25P}{0.53f_c'A_g}\right) \quad (3.5)$$

where

A_{sh} = total cross-sectional area of transverse reinforcement provided separately in each direction, including cross-ties, where applicable

s = spacing of transverse reinforcement measured along the length of the pile

h_c = cross-sectional dimension of pile core measured center-to-center of transverse reinforcement

f_c' = specified compressive strength of concrete

f_{yh} = yield strength of transverse reinforcement ≤ 100,000 psi (700 MPa)

P = factored axial load on pile, as determined using all applicable load combinations that include the effect of seismic load E as contained in governing building, bridge or other structural codes; note that effective prestress is not used to determine P

A_g = pile gross cross-sectional area

Exception: The minimum transverse reinforcement required by the equations above shall not apply in cases where the applicable overstrength factor Ω_0, as defined by the governing code, has been used to increase the seismic demand on the pile beyond that required for the structure's lateral force-resisting system. In such cases, minimum transverse reinforcement shall be as specified in section 3.7.1.1, which is applicable to all piles.

For the remaining length of the pile outside the ductile region, the volumetric ratio ρ_s, or the total cross-sectional area of transverse reinforcement A_{sh}, shall not be less than half that required in the ductile region.

For areas of moderate seismicity, there are no maximum transverse reinforcement spacing or pitch requirements beyond those already presented in section 3.7.1.1 and applicable to all piles.

Recent research reviewed the relationship between curvature ductility demand on prestressed concrete piles and overall system ductility demand in the context of all soil profiles identified in ASCE/SEI 7-10.[11,12] The research concluded that the spiral equation presented above would result in curvature ductility capacities exceeding 12, which was established by the authors of the research as a minimum limit needed for areas of moderate seismicity. More information regarding the rationale for selecting of an appropriate minimum ductility capacity limit is included in the next section. The allowance for reduced required transverse reinforcement in the bottom of the pile is a result of the expectation that inelastic pile curvatures will be concentrated in the pile's ductile region.

The previously mentioned research considered only circular transverse reinforcement, yet square transverse reinforcement is sometimes used.[11] For square transverse reinforcement, the equation for minimum transverse reinforcement area A_{sh} in each direction is a simple conversion from effective confinement pressure in the concrete core area associated with circular spiral reinforcement to an equivalent effective confinement pressure associated with square reinforcement. The conversion is in accordance with Mander et al.[7] and is discussed in detail in Priestly et al.[9] A similar conversion is used in the *AASHTO LRFD Bridge Design Specifications*.[6] It should be noted that for piles, the transverse reinforcement ratios in orthogonal directions ρ_x and ρ_y are typically equal and in all cases their sum is theoretically equivalent to ρ_s. As such, ρ_x and ρ_y must individually provide a steel area equivalent to $0.5\rho_s$. In addition, and as discussed in section 3.7.1, it should be noted that spiral reinforcement is approximately 95% effective, whereas square transverse reinforcement effectiveness is generally taken conservatively as 75%. As such, the required steel area for square transverse reinforcement should be increased by a factor of 1.27. In this document, for equivalent confinement, the area of transverse reinforcement for square transverse reinforcement, in each direction, is taken as approximately 65% (0.5 × 1.27) of the required spiral reinforcement.

The exception statement is consistent with other overstrength philosophy statements presented in most codes and load/material standards (for example, the 2015 *International Building Code*[8] and the 2014 *AASHTO LRFD Bridge Design Specifications*[6]). It recognizes that the volumetric ratio of transverse reinforcement required should not be increased beyond that required for driving and handling stresses, when the pile considered is not part of the lateral force-resisting system and has been designed for load combinations including overstrength.

In summary, because these piles have already been designed to respond elastically to design earthquake forces that are further amplified by the effect of overstrength, the increased axial forces, shear forces, and bending moments in the piling provide a large factor of safety against nonlinear pile curvatures over the entire length of the pile.

3.7.1.2.3 Minimum transverse reinforcement requirements for areas of high seismicity: pile not considered part of the lateral force-resisting system

For structures in areas of high seismicity where the pile is not considered part of the lateral force-resisting system, pile prestressing strands must be enclosed in a transverse reinforcement equal to or exceeding the following minimum requirements.

The pile's assumed ductile region is defined here as:

- For fully embedded piles, the upper 35 ft (11 m) of the pile or the section from the top of the pile to the location along the pile length of maximum moment below grade plus three pile dimensions, whichever is greater.

- For piles that extend above grade, the top of pile ductile region is the upper two pile dimensions measured from the underside of the pile cap and the bottom of pile ductile region is the larger of (a) the upper 35 ft (11 m) of the pile below grade and (b) the section of the pile from grade to the location of maximum moment below grade plus three pile dimensions (the effect of scour should be considered).

In this region, the volumetric ratio ρ_s shall not be less than the following.

For piles using a circular strand configuration with spiral reinforcement:

$$\rho_s = 0.06\left(\frac{f_c'}{f_{yh}}\right)\left(2.8 + \frac{1.25P}{0.53 f_c' A_g}\right) \tag{3.6}$$

where

ρ_s = spiral reinforcement index or volumetric ratio (vol. spiral/vol. core)

f_c' = specified compressive strength of concrete

f_{yh} = yield strength of spiral reinforcement ≤ 100,000 psi (700 MPa)

P = factored axial load on pile, as determined using all applicable load combinations that include the effect of seismic load E as contained in governing building, bridge or other structural codes; note that effective prestress is not used to determine P

A_g = pile cross-sectional area

For piles using a square strand configuration with a square transverse reinforcement pattern:

$$A_{sh} = 0.04sh_c\left(\frac{f_c'}{f_{yh}}\right)\left(2.8+\frac{1.25P}{0.53f_c'A_g}\right) \quad (3.7)$$

where

A_{sh} = total cross-sectional area of transverse reinforcement provided separately in each direction, including cross-ties, where applicable

s = spacing of transverse reinforcement measured along the length of the pile

h_c = cross-sectional dimension of pile core measured center-to-center of square transverse reinforcement

f_c' = specified compressive strength of concrete

f_{yh} = yield strength of square transverse reinforcement ≤ 100,000 psi (700 MPa)

P = factored axial load on pile, as determined using all applicable load combinations that include the effect of seismic load E as contained in governing building, bridge or other structural codes; note that effective prestress is not used to determine P

A_g = pile gross cross-sectional area

Exception: The minimum transverse reinforcement required by the equations above shall not apply in cases where the applicable overstrength factor Ω_0 as defined by the governing code has been used to increase the seismic demand on the pile beyond that required for the structure's lateral force-resisting system. In such cases, minimum transverse reinforcement shall be as specified in section 3.7.1.1, which is applicable to all piles.

For the remaining length of the pile outside the pile's ductile region, the volumetric ratio ρ_s, or the total cross-sectional area of transverse reinforcement A_{sh}, shall not be less than half that required in the ductile region.

Table 3.6 presents the maximum lateral tie spacing and spiral pitch as required for areas of high seismicity.

Recent research considered the relationship between curvature ductility demand on prestressed concrete piles and overall system ductility demand in the context of all soil profiles identified in ASCE/SEI 7-10, and concluded that Eq. (3.7) would result in curvature ductility capacities exceeding 18, which was established by the authors of the research as a minimum limit needed for areas of high seismicity.[11,12] Establishing target curvature ductility capacities for areas of high seismicity are discussed throughout the literature. For example, the highest codified ductility demand for buildings is in the New Zealand Standard NZS 3101,[13] where designs are based on a curvature ductility capacity of 20. Similarly, ATC-32[14] set the curvature ductility capacity target for vertical compression members at 13 with the expectation that 50% more capacity is available (that is, maximum available curvature ductility is 19.5). It is important to note that the ATC-32 assumption of 13 was incorporated into design provisions used for bridge columns in areas of high seismicity and for applications where significant ductility was required in the column. The allowance for reduced required transverse reinforcement in the bottom of the pile is a result of the expectation that inelastic pile curvatures should be concentrated in the pile's ductile region.

The exception statement is consistent with other overstrength philosophy statements presented in most codes and load or material standards and is discussed in detail in section 3.7.1.2.2 for areas of moderate seismicity where the same exception statement applies.

3.7.1.2.4 Minimum transverse reinforcement requirements for areas of moderate seismicity: pile considered part of the lateral force-resisting system

For structures in areas of moderate seismicity where the pile is considered part of the lateral force-resisting system, transverse reinforcement requirements as specified for piles in areas of moderate seismicity that are not part of the lateral force-resisting system are applicable, subject to the following modifications.

Because pile ductility is expected during the design earthquake, the pile ductile region as previously defined for areas of moderate seismicity remains unchanged. However, this pre-

Table 3.6. Maximum lateral tie spacing and spiral pitch requirements for prestressed concrete piles in areas of high seismicity (pile not considered part of the lateral force-resisting system)

Pile location	Maximum lateral tie spacing and spiral pitch
Ductile region	Minimum of: ⅕ times the least pile dimension six strand diameters 6 in. (152 mm) (for piles up to 24 in. [610 mm]) 4 in. (102 mm) (for piles larger than 24 in.)
Below the ductile region	See section 3.7.1.1

scriptive requirement for enhanced confinement from the top of the pile may reach excessive lengths, and in such cases, it is recommended that performance-based design be considered.

The exception statement that allows for a reduction in transverse reinforcement due to consideration of overstrength is not permitted when designing piles that are part of the lateral force-resisting system. Overstrength amplifications are not traditionally applied to the lateral force-resisting element designed for ductility.

Table 3.7 presents the maximum lateral tie spacing and spiral pitch as required for areas of moderate seismicity for piles that are part of the lateral force-resisting system.

3.7.1.2.5 Minimum transverse reinforcement requirements for areas of high seismicity: pile considered part of the lateral force-resisting system

For structures in areas of high seismicity where the pile is considered part of the lateral force-resisting system, transverse reinforcement requirements as specified for piles in areas of high seismicity that are not part of the lateral force-resisting system are applicable, subject to the following modifications.

Because pile ductility is expected during the design earthquake, the pile ductile region as previously defined for areas of high seismicity remains unchanged. However, this prescriptive requirement for enhanced confinement from the top of the pile may reach excessive lengths, and in such cases, it is recommended that performance-based design be considered.

The exception statement that allows for a reduction in transverse reinforcement due to consideration of overstrength is not permitted when designing piles that are part of the lateral force-resisting system. Overstrength amplifications are not traditionally applied to the lateral force-resisting element designed for ductility.

3.7.1.3 Performance-based design of transverse reinforcement

For many structures in areas of moderate to high seismicity, it is likely beneficial to detail the transverse reinforcement along the length of the pile using a performance-based design methodology as described in this section. Performance-based design procedures are typically referred to as either nonlinear static analysis, which is also called pushover analysis, or nonlinear time history analysis. In this document, the nonlinear modeling requirement is the inclusion of axial-load-dependent nonlinear moment rotation characteristics of the pile as defined by moment curvature curves developed for each pile making up the foundation system. In addition to nonlinear pile hinging considerations, performance-based design procedures require explicit modeling of the pile foundation system to include the effects of soil-structure interaction and appropriate modeling of pile head fixity. Performance-based design can be used irrespective of whether or not the prestressed pile is considered part of the lateral force-resisting system. Performance-based design requires the curvature of the pile to be fully defined along the length of the pile under all applicable load combinations. This approach aims to ensure that the transverse reinforcement is adequate to resist pile curvatures expected under design earthquake motions. The remaining portion of this section is not intended to provide or recommend complete performance-based design procedures, which are code-specific, but to provide performance characteristics, experimentally and analytically proven modeling assumptions, and recommended detailing practices specific to prestressed concrete piles and their connections (see sections 3.7.1.3.1 through 3.7.1.3.3).

In performance-based design, axial-moment interaction failure of a pile occurs during the nonlinear analysis whenever a strain limit in a pile's ductile region is exceeded. Performance-based design does not permit failures to occur during the analysis for displacement demands resulting from code-prescribed design accelerations for the site. Practically speaking, failures occur when the lateral demand-induced curvature along the length of a pile causes a steel strand to reach a tensile strain limit established by the governing code, or the concrete reaches its confined compressive strain limit within the core of the cross section.

It should be noted that the performance-based design process typically requires that the foundation system be modeled as part of a global model of the entire structure. This is in contrast to standard practice using prescriptive codes, which allow for the assumption of a fixed base with foundation loads sometimes provided to a separate pile designer working for the pile fabricator. Any decoupling of the foundation system model from the structural model above the foundation should require engineer of record approval.

Table 3.7. Maximum lateral tie spacing and spiral pitch requirements for prestressed concrete piles in areas of moderate seismicity (pile considered part of the lateral force-resisting system)

Pile location	Maximum lateral tie spacing and spiral pitch
Ductile region	Minimum of: ⅕ times the least pile dimension six strand diameters 6 in. (152 mm) (for piles up to 24 in. [610 mm]) 4 in. (102 mm) (for piles larger than 24 in.)
Below the ductile region	See section 3.7.1.1

3.7.1.3.1 Plastic hinge length

Plastic hinge length is the length over which the plastic curvature is assumed to be constant for estimating the plastic rotation of the pile at a discrete location. In performance-based design, the plastic rotation is used to calculate the plastic displacement of the pile from the point of maximum moment to the point of contraflexure where it occurs along the length of the pile. The plastic hinge length is directly proportional to the plastic rotation capacity of the pile. Research (see the 2016 Marine Oil Terminal Engineering and Maintenance Standards[10] or ASCE/COPRI 61-14[15] for discussion) on prestressed concrete pile connections to pile caps and in ground hinges of prestressed concrete piles suggests that the minimum plastic hinge length for a precast concrete pile should be taken as given in Eq. (3.8) and (3.9).

Fixed-head or prestressed concrete piles framing into a footing or bent cap:

$$L_p = D^* \tag{3.8}$$

where

L_P = plastic hinge length

D^* = diameter or cross-sectional dimension in direction of bending

Below-grade plastic hinges:

$$L_p = 2D^* \tag{3.9}$$

For cases where the pile is not considered part of the lateral force-resisting system, or for special cases where the pile is considered part of the lateral force-resisting system but the engineer chooses to design an essentially elastic response of the pile system, the length of the plastic hinge below grade can be increased. Essentially elastic pile curvatures are defined in this report as a curvature ductility demands less than 4 (that is, $\mu_\phi \le 4$). The lower ductility demand may be due to lower seismic demands or code provisions prohibiting pile damage below grade, as is standard practice for important bridges in areas of high seismicity. The below-grade plastic hinge length may be increased to at least 150% of the value defined above ($L_p = 3D^*$). The conservative increase is based on research for piles hinging below grade, which has shown that essentially elastic pile responses have considerably larger plastic hinge lengths of at least three pile dimensions in areas of moderate to high seismicity.[16]

3.7.1.3.2 Minimum transverse reinforcement requirements for areas of moderate to high seismicity

When using performance-based design, the general intent is to require an amount of transverse reinforcement consistent with the required ductility demand along the length of the pile. The pile's ductile regions are not prescriptively defined but are determined as part of the design process. Piles with pinned head conditions are not detailed with a ductile region at the pile head but are detailed with a ductile region at the maximum moment location below grade. Fixed-head piles are detailed with ductile regions both at the pile head and at the maximum moment region below grade.

For structures in areas of moderate to high seismicity, irrespective of whether or not the pile is considered part of the lateral force-resisting system, pile prestressing strands must be enclosed in a transverse reinforcement that meets or exceeds the following minimum requirements.

In the pile's assumed ductile regions, the volumetric ratio ρ_s shall be based on the actual maximum curvature ductility demand μ_0 for all applicable load combinations, including seismic, and determined as follows.

For piles using a circular strand configuration with spiral:

$$\rho_s = 0.06\left(\frac{f_c'}{f_{yh}}\right)\left(\frac{\mu_0}{18}\right)\left(2.8+\frac{1.25P}{0.53f_c'A_g}\right) \tag{3.10}$$

where

ρ_s = spiral reinforcement index or volumetric ratio (vol. spiral/vol. core)

f_c' = specified compressive strength of concrete

f_{yh} = yield strength of spiral reinforcement ≤ 100,000 psi (689 MPa)

μ_0 = curvature ductility demand as required by performance-based analysis

P = factored axial load on pile, as determined using all applicable load combinations that include the effect of seismic load E as contained in governing building, bridge or other structural codes; note that effective prestress is not used to determine P

A_g = pile cross-sectional area

For piles using a square strand configuration with a square transverse reinforcement pattern:

$$A_{sh} = 0.04sh_c\left(\frac{f_c'}{f_{yh}}\right)\left(\frac{\mu_0}{18}\right)\left(2.8+\frac{1.25P}{0.53f_c'A_g}\right) \tag{3.11}$$

where

A_{sh} = total cross-sectional area of transverse reinforcement provided separately in each direction, including cross-ties, where applicable

s = spacing of transverse reinforcement measured along the length of the pile

h_c = cross-sectional dimension of pile core measured center-to-center of transverse reinforcement

f_c' = specified compressive strength of concrete

f_{yh} = yield strength of transverse reinforcement ≤ 100,000 psi (689 MPa)

μ_0 = curvature ductility demand as required by performance-based analysis

P = factored axial load on pile, as determined using all applicable load combinations that include the effect of seismic load E as contained in governing building, bridge or other structures code; note that effective prestress is not used to determine P

A_g = pile gross cross-sectional area

Exception: The limits for ρ_s and A_{sh} as provided above are based on parametric study equations used to develop the prescriptive limits presented in the previous section. As such, values for ρ_s and A_{sh} may be obtained independently of the equations above by simply showing that the curvature demand is less than the curvature capacity as required by the code governing performance-based design for the structure.

In performance-based design, the location of damage is known but the actual length of damage at this location is not certain. As such, the length of the ductile regions requiring enhanced transverse reinforcing as determined at specific locations along the length of the pile is defined by the larger of (a) 2.0 times the cross-sectional dimension of the pile in the direction of bending, (b) anywhere the moment exceeds 75% of the nominal moment capacity as defined by the governing code for the structure, and (c) L_p.

Segments of the pile outside the ductile region should have spiral or tie reinforcement quantities that are no less than half that required for the ductile region, and these areas must also meet the general provisions for transverse reinforcement required for all piles (section 3.7.1.1) irrespective of seismicity level.

3.7.1.3.3 Additional performance-based design considerations for prestressed concrete piles

The following additional performance-based design considerations for prestressed concrete piles are based primarily on work completed by Sritharan et al.[11]

Considering all load combinations that include seismic, axial load ratios $P_u/(f_c'A_g)$ should be limited as follows:

- 0.4 for octagonal piles less than 24 in. (610 mm) in diameter

- 0.45 for octagonal piles 24 in. (610 mm) in diameter and larger

- 0.2 for square piles less than or equal to 14 in. (360 mm) per side

- 0.4 for all other square piles

Exception: The axial load limits above are conservative and are intended to avoid cases where high axial loads might lead to cover spall-related curvatures less than those associated with tensile cracking on the opposite face. The previously mentioned axial load limits can be neglected if the designer considers strand buckling directly, ensures that the moment curvature curve after spalling is stable, and checks that the confined concrete core itself (neglecting the concrete cover) can resist all applicable factored load combinations that include seismic. The maximum prescriptive transverse reinforcement spacing requirements presented in Table 3.6 for areas of high seismicity may be assumed to satisfy strand buckling requirements.

3.7.2 Splices and build-ups Pile splices are defined as any method of joining prestressed concrete pile sections in the field during driving so that driving may continue. Pile splices may be used for a variety of reasons. The supplied length of a pile may not be sufficient to obtain the specified bearing capacity and further driving may be required. On some projects, this issue may be caused by nonuniform substrata conditions that make it difficult to determine required pile lengths. More commonly, the estimated length of pile cannot be economically or feasibly transported to the site (or handled in the driver) and therefore a process whereby shorter piles spliced together to make one long pile is required. A pile may have one or more splices. Each splice must be capable of resisting all subsequent stresses and deformations that may be induced through driving or under service loads. Because there is no net prestressed-induced compression at the ends of the two pile segments being spliced together, any net load induced tension transferred across the splice must be detailed to occur through properly developed reinforcing in the pile. The type of splice selected must depend primarily on service loads and conditions. Not all splices are capable of resisting moment or uplift.

Some codes and standards may require all pile splices to resist some portion of the moment or tensile capacity of the pile and where prescriptive design methods are used, this approach is reasonable. However, when performance-based design is used, designers can ensure that the pile splice is located in a portion of the pile that will be subject, as a result of all applicable load combinations, only to compression and in such cases the splice should be designed for the actual actions induced by all load combinations or as required by the nonlinear static analysis results.

Commonly used splices can be categorized generally as the following:

- dowel with epoxy

- welded

- bolted or pinned
- mechanical locking
- connector ring
- wedge
- sleeve
- post-tensioned

Illustration of these pile splice types can be found in chapter 20 of the *PCI Bridge Design Manual* (MNL-133). There is a variation in the behavior of various splices under field conditions. Failure of some could occur directly in the joint, while in other cases, failure could occur at the dowels anchoring the splice to the piles. In some splice systems, failure would occur completely outside of the spliced region. The ability of a splice to develop the strength of the pile, or reasonable percentage of that strength, depends on close tolerances and proper procedures in making the splice. Careless workmanship or improper field procedures can result in significant deviations from the strength and behavior levels desired.

When piles are purposely over-driven a significant distance below the required cutoff elevation to obtain required bearing capacity, a pile splice can be used. However, sometimes piles are accidentally or purposely over-driven only a short distance below the prescribed cutoff elevation and lowering the pile cap is not considered a feasible option. Build-up is the term applied to any method of extending a driven pile to the required cutoff elevation. Build-up can be accomplished through the use of a precast concrete section, but is generally made of cast-in-place concrete. The build-up must be capable of developing the service and strength level stresses. Concrete quality must be compatible with the prestressed concrete pile. Typical build-up details are shown in chapter 20 of PCI MNL-133.

When piles are driven to bearing capacity before reaching the predicted tip elevation, they should be cut to the specified elevation and orientation. Piles should be cut in a manner to minimize the likelihood of damage to the concrete at the pile head. Where projecting reinforcement is specified, reinforcement should not be damaged during pile-cutting operations.

3.7.3 Segmented cylinder piles Large diameter cylinder piles are often manufactured with centrifugal casting in segments 8 to 16 ft (2.44 to 4.88 m) in length. Longitudinal holes are formed during casting to receive post-tensioning strands or wires. Post-tensioning follows assembly of the segments and proper application of the joint sealant material. Such sealing material (generally polyester resin) should be of sufficient thickness to fill all voids between surfaces. The pile sections should be brought into contact and held together under compression while the sealing material sets. After completing the prestressing, all tendons should be fully grouted and stress on tendons maintained until the grout develops the required strength.

3.7.4 Connections between pile and pile cap The design of the pile-to-pile cap connection depends on the load magnitude and how the load is directed. The load can be axial (tension or compression), or a bending moment, or a combination of axial load and moment. If the connection requires moment resistance, it must be recognized that the prestress in a pile, as in other pretensioned members, varies from zero at the end of the pile to full effective prestress approximately 50 to 60 strand diameters from the end. A pin-head condition allows rotation of the pile head with respect to the pile cap. A fixed-head condition minimizes the rotation of the pile head at the connection to the cap and therefore extremely high moments may be transferred between the pile head and the cap.

A pinned pile-to-pile cap connection can be used where moment transfer between the pile and pile cap is not required or considered in the structural model. If the pile is expected to remain in compression, it can be embedded into the pile cap without a mechanical connection (note that some codes and agencies require mechanical connection details for all pile-to-pile cap connections). The embedment distance should be sufficient to ensure a positive connection, while still permitting some rotation to eliminate excessive moment development. Specific requirements for pinned connections, such as embedment lengths, roughening the pile surface over the embedded length, exposing strands, extending mild steel reinforcement, or providing spiral reinforcement around the embedded length, are code-specific and are not discussed here.

A fixed pile-to-pile cap connection is required whenever moment must be transferred between the pile cap and the top of the pile. Common fixed methods of connection include the following:

- Pile head extension: The pile head is extended into the cap generally one pile dimension minimum. Embedded surfaces of the pile must be clean and preferably roughened before casting concrete.
- Strand extension: Prestressing strands are extended into the pile cap. Prestressing steel in this case cannot be assigned allowable stresses greater than reinforcing steel (maximum allowable stress of about 30,000 psi [210 MPa]). Embedded lengths depend on design requirements but should be 18 in. (460 mm) minimum.
- Mild steel dowels: Dowels can be cast in the head of piles either projecting or fully embedded for exposure after driving. When piles are cast with projecting dowels, a special driving helmet must be provided. The same applies to piles cast with projecting strand.
- The preferable method of connection is to cast piles with formed holes in the pile head. Dowels are grouted into these holes following pile driving. Holes can also be drilled into the pile head after driving to accommodate grouted dowels. The area of the holes in the pile head should not exceed 6% of the gross area of the pile. In

addition, embedded reinforcement or dowel holes should have their terminus points within the pile staggered.

- It should be recognized that either embedded mild steel or holes cast to receive dowels can change the effective prestress level in the pile near the pile head.

- Other connections: For cylindrical piles, a cage of reinforcement can be concreted into the pile core following driving. A form must be provided either of disposable materials, such as wood, or a precast concrete plug, which is grouted into the core. Structural steel members can also be used as connectors in cylindrical piles.

- Various extensions: Combinations of pile head extension, strand extension and/or dowels can be used.

Typical pile-to-pile cap connection details and design philosophies are shown and discussed in chapter 20 of PCI MNL-133.

3.7.5 Cover The minimum recommended cover for spiral and tie reinforcement is as follows:

- normal exposure: 2 in. (50.8 mm)

- marine or similar corrosive environment: 2½ in. (63.5 mm)

Note that some agencies, jurisdictions, codes, or standards may require more or less cover, depending on local experience.

Section 4—Manufacture and transportation of prestressed concrete piles

This section covers important considerations specifically related to manufacture and transportation of prestressed concrete piles. More detailed information on these topics can be found in chapter 20 of PCI MNL-133.

4.1 Manufacturing plants

Because casting of prestressed concrete piles is repetitive and the work is performed by experienced, trained personnel in a controlled environment, the quality of plant-manufactured prestressed concrete piles is ensured and consistently maintained. Specifications should require that manufacturers be regularly engaged in the production of prestressed concrete piles and be able to demonstrate, through past performance, their ability to achieve the required quality. Proven capability should be shown through participation in the PCI Plant Certification Program.

Plant operations often include a central concrete batch plant and delivery vehicles that transfer concrete from the batch plant to the casting beds. Casting beds for piles are usually constructed in long lines and casting is done on a continuous, daily cycle.

4.2 Handling and storage

Damage to piles can occur during the handling, storage, and transporting stages. Handling should be done using designed lifting points. On many long slender piles, three, four, or five pick-up points are required. This is done using equalizing slings and strong-backs. If proper care and caution are not used, severe damage can occur. Piles in storage should be properly supported to avoid permanent sweep introduced during curing. Points at which piles are to be lifted or supported should be clearly apparent. When other picking methods are used (inserts, slings, and vacuum pads), suitable markings to indicate correct support points should be provided. Piles stacked in storage should have intermediate dunnage supports in vertical alignment.

4.3 Transporting

Prestressed concrete piles are normally delivered from the manufacturing plant to the construction site via barge, truck, or rail. Piles up to approximately 50 ft (15 m) long can be carried on flat bed trailers. Piles over this length are generally carried on expandable flatbed trailers or telescoping pole trailers.

For piles requiring more than two support points, special supports should be articulated to avoid excessive bending stress in the pile. One method is to build A-frames, one at the tractor end and one at the dolly end. On top of the A-frames, a long steel support will provide the pile two or more supports at each end and will bring the load down to single points at the front and back of the hauling unit. Job access conditions should be reviewed before delivery and all obstructions, ruts, holes, or dangerous conditions corrected.

4.4 Tolerances

Piles are typically manufactured in steel forms on a long-line casting bed using steel headers or plates that form the ends of each pile and cross-sectional dimensions are defined by the form. Strands are held in the proper position at pile ends as they pass through holes in the headers. In long piles, where the tensioned strands may sag between headers, chairs or other means are used to properly support the strands.

Piles should be fabricated in accordance with the generally accepted dimensional tolerances found in PCI MNL-116. These tolerances are also presented in chapter 20 of PCI-MNL 133. Closer tolerances are required when using mechanical splices, as recommended by the splice manufacturer.

Section 5—Installation of prestressed concrete piles

5.1 Structural integrity

A pile that is properly designed is only capable of serving its intended purpose if it is properly installed and undamaged during the installation process. When piles are damaged during the installation process, they must be extracted or repaired to the satisfaction of the engineer of record, or in accordance with the governing code or standard applicable to the structure.

A wide variety of methods have been used for the installation of prestressed concrete piles. These methods differ according to the factors listed below, but all have one common objective: any prestressed concrete pile should be installed in a manner that ensures the structural integrity of the pile itself, so that it is able to resist known site conditions and imposed design loads. The selection and appropriate use of equipment that considers a matching of the driving system to the pile-soil system is a key part of any successful installation process.

5.2 Factors affecting installation

Installation methods may vary with the following factors (PCI MNL-133):

- size, type, and length of pile
- required driving capacity of the pile
- inclination of the piles (vertical or battered)
- type of soil/rock into which the pile is to be installed
- surrounding element (soil or water)
- uniformity of the soil strata from bent to bent
- effective prestress in the pile
- pile group arrangement
- site location and accessibility (such as presence of surrounding structures, potential to use larger equipment supported by floating barges as opposed to land-based equipment)
- number of piles on the project and the construction schedule (economy of scale)

5.3 Methods of installation

The most common method of installing prestressed concrete piles is by driving with an impact hammer. Commonly used hammer types are air powered, diesel powered, and hydraulic powered.

Each of these types may be either single-acting or double-acting. In single-acting hammers, the ram is powered up only and allowed to gravity fall. In double-acting hammers, the ram is powered up and powered down.

For on-land installation applications, rigs are usually mounted on caterpillar treads or rubber-tires with outriggers. Templates are not as commonly used for land applications because alignment is facilitated by ground stability. Water installation is more difficult and barge or temporary work platform installations are performed. Templates that hold the piles in position are often required for water installations.

5.3.1 Hammer selection Matching of an appropriate hammer to the pile-soil system is of critical importance. The hammer selected must be able to drive the pile to the required capacity or depth without damaging the pile. In comparing hammers of equal energy, those with a heavier ram and lower impact velocity, such as air hammers, are less likely to cause damaging stresses in the pile. Knowledge of the particular soils, experience of local geotechnical consultants and experience of pile-driving contractors are often the primary sources of information in the selection of hammers.

Wave equation analysis may be used to aid in hammer selection. The wave equation is used to predict pile capacity, driving resistance, and stresses in the pile.

In the field, dynamic testing of selected piles during driving may be helpful, or necessary in some cases, in evaluating the effectiveness of the selected pile installation system.

Driving stresses caused by hammer impact must be maintained below levels that could result in pile damage. Pile cushion material and thickness can be varied to alter driving stresses, particularly tensile stresses, which are critical in longer piles. Similarly, the hammer may be operated with a lower drop height or fuel setting to reduce driving stresses.

5.3.2 Dynamic pile testing Dynamic pile testing consists of monitoring production or designated test piles during driving by the use of electronic equipment. The objective is to provide the engineer with information for evaluating pile hammer performance, cushion adequacy, driving stresses, and pile load capacity. These data are particularly useful for capacity estimates on restruck piles. Restruck piles are piles that are restruck by the pile hammer sometime after installation (typically one to seven days). The idle period allows capacity gain with time to occur (setup, freeze).

5.3.3 Driving heads (helmets, cap blocks) and pile cushions Piles driven by impact require an adequate driving head to distribute the hammer blow to the head of the pile. This driving head should be axially aligned with the hammer and pile. It should not fit tightly on the pile head, as this might cause transfer of moment or torsion and result in damage to the pile head. The driving head itself is intended to transmit the hammer energy to the pile without dissipating a significant amount of the energy. The driving head also holds or retains the cushion block to reduce the shock of the blow and distribute the driving force evenly over the pile head. Pile cushions are typically constructed of wood, although other materials have been used. A new cushion is typically used for each pile installation.

5.3.4 Jetting (jet-spudding, prejetting) Prejetting is the technique of inserting into the soil, before pile installation, a weighted jet for the purpose of breaking up hard soil layers. The jet is then withdrawn and the pile installed in the same location. This prejetting may also leave the soils in a temporarily suspended or liquefied condition, which will permit easier penetration of the pile. Jetting will temporarily reduce the skin friction in sands and sandy material.

When jet pipes are installed in the piles and used in conjunction with driving, precautions should be taken in choosing the material used for the embedded jet pipe to ensure the compatibility of the pipe with the surrounding concrete. Differences in the modulus of elasticity between polyvinylchloride (PVC) and concrete, for example, may cause damage to joints in the pipe, with corresponding damage to piles. Jetting while driving may not be recommended by the geotechnical engineer, as pile capacity may be compromised.

5.3.5 Predrilling Predrilling a starter hole may be appropriate to locate or penetrate subsurface obstructions that may interfere with pile installation. Predrilling may also be used to break up hard strata or simply to loosen upper soils to facilitate pile installation. Drill size and depth of predrilling is often recommended by the experienced pile-driving contractor, subject to the approval of the engineer of record.

5.3.6 Spudding Spudding is the technique of inserting into the soil, before pile installation, a shaft or mandrel for the purpose of forcing a hole through hard soil layers, trash, overlying fill, and other conditions. Spudding a starter hole may be helpful in penetrating especially difficult material located near the surface that may tend to deflect the pile when driven.

5.3.7 Followers Sometimes a rigid structural member, also called a follower, is installed between the drive head and the pile head to drive the pile to a given depth below grade. Followers are not recommended because their use modifies the dynamic energy transferred to the pile during driving. In cases where a follower is required and its use is approved by the engineer of record, a wave equation analysis may be necessary to better assess the pile's behavior.

5.3.8 Excavating Driving, jetting, predrilling, or spudding are not always feasible to penetrate obstructions or difficult material. It may be necessary to excavate and remove material before pile installation.

5.3.9 Batter piles The installation of batter piles is more difficult than the installation of plumb piles and must be carried out by a skilled pile-driving contractor who has knowledge of local subsurface conditions and experience with the equipment being used and the type of piles being driven. Maintaining the proper alignment during installation is challenging. Long slender batter piles bend downward due to gravity during driving. These deflections create additional bending stresses in the pile when hit by a hammer, which must be taken into account.

5.4 Prevention of damage

Piles may be damaged during handling, transportation, or storage, but this type of damage is similar to that for other precast concrete elements and can usually be corrected or repaired using conventional repair methods (PCI MNL-137). In some cases, prestressed concrete piles have cracked or spalled during driving. The damage or failure of such concrete piles occurring during driving can be classified into four types:

- spalling of concrete at the head of the pile due to high compressive stress
- spalling of concrete at the tip of the pile due to hard driving resistance at the tip
- transverse cracking or breaking of the pile due to torsion or reflected tensile stress sometimes accompanied by spalling at the crack
- bursting of hollow prestressed concrete piles

5.4.1 Compression damage at head Spalling of concrete at the head of a pile is due to very high or nonuniform compressive stresses or compression stress concentrations caused by the following:

- Insufficient cushioning material between the driving head and the concrete pile will result in very high compressive stresses on impact of the pile driver ram.
- When the top of the pile is not square or perpendicular to the longitudinal axis of the pile, the ram impact force will be concentrated on one edge. A warped or worn helmet can create the same situation.
- Improper fit (generally too tight) of the driving head on top of the pile.
- If the prestressing steel is not cut flush with the end of the pile, the ram impact force may be transmitted to the concrete through the projecting prestressing steel resulting in high stress concentrations in the concrete adjacent to the steel.
- Lack of adequate spiral reinforcing steel at the pile head or pile tip may lead to spalling or longitudinal splitting. In prestressed concrete piles, anchorage of the strands is developed in these areas and transverse tensile stresses are present.
- If the top edges and comers of the concrete pile are not adequately chamfered, they are likely to spall on impact of the ram.

5.4.2 Compression damage at pile tip Spalling of concrete at the tip of a pile can be caused by extremely hard driving resistance at the tip. Such resistance may be encountered when founding the pile tip on bed rock. Compressive stresses when driving on bare rock can theoretically be twice the magnitude of those produced at the head of the pile by the hammer impact.

Under such conditions, over-driving of the pile and, particularly, high ram velocity should be avoided. In the more normal cases, with overburden of soil overlying the rock, tip stresses will generally be of the same order of magnitude as, but slightly lower than, the head stresses.

5.4.3 Transverse cracking Transverse cracking of a pile due to reflected tensile stress can lead to failure of the pile during driving. This type of cracking is indicated by the presence of parallel cracks, often initiated at points of discontinuity (such as lifting loops, inserts, honeycombs, and the like), perpendicular to the longitudinal axis of the pile. Repeated driving once cracking has initiated produces puffs of concrete dust and can result in spalling and widening of the cracks, possibly followed by localized fatigue of the concrete and brittle fracture of the prestressing tendons.

This transverse cracking, although rare, most often occurs when driving in very soft soil. It can, however, also occur when driving resistance is extremely hard, as when the tip of the pile bears on solid rock.

When the pile driver ram strikes the head of a pile, compressive stress is produced at the head of the pile. This compressive stress travels as a wave down the pile at a velocity of approximately 12,000 to 15,000 ft/sec (3660 to 4570 m/sec). The intensity of the stress wave depends on the ram, the impact velocity, the cushion at the head of the pile, the structural characteristics of the pile, and the soil resistance.

Because the stress wave in a given pile travels at a constant velocity, the length of the stress wave period will depend, among other things, on the length of time the 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 with equal energy, thus providing a longer stress wave period. If a ram strikes a thick or soft cushion, it will also stay in contact for a longer time than if it strikes a thin, hard cushion. The longer contact time generally results in a decrease in driving stress.

The compressive stress wave traveling down the pile may be reflected from the point of the pile as either a tensile or compressive stress, depending on the soil resistance at the tip. In order for the pile to penetrate the soil, the compressive stress wave must pass into the soil. If little or no soil resistance is present at the pile tip, the compressive stress wave will be reflected as a tensile stress wave.

The net tensile stress in the pile at any point is the algebraic sum of the compressive stress traveling down the pile and tensile stress traveling up the pile. Whether or not a critical tensile stress will result depends on the magnitude of the initial compressive stress and the length of the stress wave relative to the pile length. A long stress wave is desirable to prevent damaging the pile.

If the soil resistance at the tip of the pile is very hard or firm, the initial compressive stress wave traveling down the pile will be reflected back up the pile as a compressive stress wave. Tensile stresses in the pile will not occur under these conditions until this compressive stress wave reaches the top-free end of the pile and is reflected back down the pile as a tensile stress wave.

It is possible for critical tensile stress to occur near the pile head in this case. Internal damping characteristics of the pile and the surrounding soil may reduce the magnitude of the reflected tensile stress wave. However, cracking has occurred when driving onto rock with very light hammers.

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

- When insufficient cushioning material is used between the pile driver's steel helmet, or cap, and the concrete pile, a stress wave of high amplitude and of short length is produced, both characteristics being undesirable because of potential pile damage.

- Use of adequate softwood cushions is frequently the most effective way of reducing driving stresses, with reductions in the order of 50% being obtained with new uncrushed cushions. As the cushion is compressed by hard driving, the intensity of the stress wave increases; therefore, a new cushion for each pile is recommended.

- When a pile is struck by a ram at a very high velocity, a stress wave of high amplitude is produced. The stress developed in the pile is proportional to the ram velocity.

- When little or no soil resistance at the tip of long piles (50 ft [15 m] or more in length) is present during driving, critical tensile stresses may occur in the pile. Tensile driving stresses greater than the ultimate concrete tensile stress plus the effective prestress can result in development of transverse cracks. This may occur when driving through a hard layer into a softer layer below, 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 of the pile length, but they may also occur at midlength or lower in the pile length.

- When very hard driving resistance is encountered at the tip of piles (50 ft [15 m] or more in length), critical tensile stresses may occur in the upper half of the pile when a tensile stress is reflected from the pile head.

5.4.4 Bursting of hollow prestressed concrete piles Longitudinal splits due to internal bursting pressure may occur with open-ended hollow prestressed concrete piles.

When driving in extremely soft, semifluid soils, the fluid pressure builds up and a hydraulic ram effect known as "water hammer" occurs.

When driving open-ended precast concrete piles in sands, a plug can form and exert an internal bursting/splitting force in the pile shell wall. This can be broken up with a jet during driving, but the most practicable remedy appears to be providing adequate lateral steel in the form of spiral or tie reinforcement. Use of a solid or steel armored tip will eliminate the splitting problems mentioned, but may not be compatible with other installation requirements.

5.5 Repair of damaged piles

Damaged piles can often be repaired when damage has been sustained during handling or driving.

5.5.1 Spalling of concrete at the head of the pile
When the head of a pile is damaged during handling, the broken concrete should be removed, the surface cleaned, and a patch applied and allowed to thoroughly cure before driving. Patching material may consist of prepackaged grout with rapid-setting characteristics or epoxy patching compounds.

When spalling occurs at the head of a pile during driving and the pile is damaged to the extent that driving must be discontinued, it is often possible to cut off the pile and resume driving. The pile must be cut at a point below any damage and cutting must be done in such a manner to provide a square, flat end at the top of the pile.

Precautions should be taken in providing cushioning that is thicker than the normal cushion block when driving is resumed on a pile that has been cut. Piles can often be driven to their final elevation in this manner, even though closely spaced spiral reinforcement at the head of the pile has been cut off.

5.5.2 Cracks in piling Cracks can be repaired, if necessary, by injecting epoxy under pressure into the cracks. Generally recognized guidelines suggest that cracks wider than 0.007 in. (0.18 mm) can be successfully injected. Smaller cracks often need no repair.

5.6 Good driving practices

Some guidelines for good driving practices for prestressed concrete piles can be summarized as follows:

- Use the proper hammer.
- Use adequate cushioning material between the driving head and the concrete pile.
- To reduce driving stresses, use a heavy ram with a low impact velocity (short stroke) to obtain the desired driving energy rather than a light ram with a high impact velocity (large stroke). Driving stresses are proportional to the ram impact velocity.
- Reduce the ram velocity or stroke during early driving when light soil resistance is encountered. Anticipate soft driving, reducing the ram velocity or stroke to avoid critical tensile stresses. This is very effective when driving long piles through very soft soil.
- If predrilling or jetting is permitted in placing the piles, ensure that the pile tip is well seated with moderate soil resistance at the tip before full driving energy is used.
- When jetting, avoid jetting near or below the tip of the pile to produce low resistance at the tip. In many sands, it is preferable and desirable to drive with larger hammers or greater resistances, rather than to jet and drive simultaneously.
- Ensure that the driving head fits loosely around the pile top so that the pile may rotate easily within the driving head.
- Ensure that bearing piles are straight and not cambered. High flexural stresses may result during driving of an initially bent pile.
- Ensure that the top of the pile is square, or perpendicular, to the longitudinal axis of the pile, and that no strands or reinforcing bars protrude from the head. Chamfer top edges and comers of the pile head.
- Use adequate spiral reinforcement throughout the pile, particularly near the head and tip.
- The prestress level should be adequate to prevent cracking during transport and handling and, in addition, the values should be adequate to resist reflected tensile stresses.

The prestress level found to be effective in resisting these effects has been established empirically at about 700 to 1200 psi (4.83 to 8.27 MPa) after losses. Very short piles have been installed with lower prestress levels (350 to 400 psi [2.41 to 2.76 MPa]). Where moment resistance in service is a requirement, effective prestress levels up to $0.2f_c'$ and even higher have been used without difficulty.

5.6.1 Pile cushioning A wood cushioning material of 3 or 4 in. (76.2 or 102 mm) may be adequate for short piles (50 ft [15.2 m] or less) with moderate tip resistances. A wood cushioning material of 6, 8, or as much as 20 in. (152, 203, or 508 mm) may be required when driving longer piles in very soft soil.

When the wood cushioning becomes highly compressed, or chars or bums, it should be replaced. A new cushion should be provided for each pile. If driving is extremely hard, the cushion may have to be replaced during driving of a single pile. Use of an adequate cushion is usually an economical means of controlling driving stresses.

In the past, concern has been expressed that cushioning might reduce the effectiveness of the driving energy transmitted to the pile. Actual experience with concrete piles and recent dynamic wave theory both indicate that normal cushioning, by lengthening the time that the ram is in contact with the head of the pile, may in some cases actually increase the penetrating power of the pile.

Further, as the pile nears final tip elevation, the cushion is usually substantially compressed. Within practical limits, adequate cushioning does not reduce driving penetration. Thus, the computed pile capacities from dynamic formulas are usually not significantly altered.

5.7 Handling and transportation

Prestressed concrete piles should be picked up, handled, and transported so as to avoid tensile cracking and any impact damage. Piles cracked due to mishandling cannot be relied on for resisting driving tensile stresses that may develop.

Superficial surface cracks, minor chips, and spalls may occur during handling and installation and are often unavoidable. As long as these minor imperfections do not affect the structural integrity or the drivability of the pile, they should not be cause for rejection. Damage that may impair performance must be repaired.

5.8 Positioning and alignment

Correct position can best be assured by accurate setting of the pile. Removal of surface obstructions will aid in attaining accurate positioning. When accuracy of position is critical, a template or a predrilled starter hole, or both, may be employed to advantage. The position is largely established when the pile is set. Attempts to correct position after driving has commenced usually results in excessive bending and damage to the pile, and should not be permitted.

As a general statement, proper control of alignment should be ensured before driving starts. It is almost impossible to correct vertical or lateral alignment after driving has commenced without inducing bending stresses.

Caution must be taken to see that the pile is started truly vertically or on the proper batter, as the case may be. Once the driving starts, the hammer blow should be delivered essentially axially, and excessive sway prevented at the pile head. The use of fixed leads, which are often specified, is primarily a means to ensure these two conditions.

Attempting to correct misalignment by chocking at the base of the leads may, except at the start of driving, introduce excessive bending and damage the piles.

Long piles should be given necessary support in the leads. Batter piles should be supported to reduce gravity bending to acceptable limits; use of rollers in the leads is one method. Long slender vertical piles may require guides at intervals to prevent buckling under the hammer blow.

When driving a long way below the leads, especially with batter piles, telescopic support leads or other appropriate means should be provided to prevent excessive bending and buckling.

If the pile is installed in water, the pile should be protected against excessive bending from waves, currents, dead weight (in case of batter pile), and accidental impact. Staying and girting should be employed until the pile is finally tied into the structure it is supporting. Pile heads should be stayed so as to eliminate bending. This is particularly relevant to batter piles where the head should be lifted to overcome the dead weight of the pile. Frequently, when driving in deep water, a batter pile should be stayed before it is released from the hammer.

The heads of piles, even in water, cannot be pulled into position without inducing bending. Because of the long lever arm available in many water installations, piles have been severely damaged even when the pulling force is relatively small. Strict pulling limits should be set by the designer.

Section 6—Current research and future applications for prestressed concrete piles

The last version of this report was written in 1993.[1] Since that time, and as discussed elsewhere in this report, significant research on precast concrete pile ductility has been performed, a better understanding of soil structure interaction principles has been promulgated by various experts, pile-driving monitoring equipment technology has improved, and the ability of modern software to incorporate complex modeling conditions has drastically improved. Based on these continuing improvements, and with other lessons learned by the industry, it is highly recommended that performance-based design procedures be used for prestressed concrete piling for all structures. Because by definition it locates and controls where damage will occur and ensures that adequate ductile details are provided in the pile, performance-based design will often result in a more reliable, life-safety-compliant foundation system, while also reducing pile-related construction costs considerably. The remainder of this section provides a few brief summaries of the current state of such research and suggests what impact this research could have on the prestressed concrete pile industry.

6.1 Ultra-high-performance concrete (UHPC) piles

In 2008, with an objective of producing a minimum service life of 75 years and reduced maintenance costs over time, Vande Voort et al.[17] studied the feasibility of using H-shaped prestressed UHPC piles as a substitute for traditional steel piles commonly used in bridge foundations (**Fig. 6.1**). UHPC has high compressive strength, on the order of five times the normal concrete strength used in traditional concrete pile applications (for example, a 26 ksi [180 MPa] mixture was used in the subject research). Although the tensile strength is greatly increased as well, it is the increased compressive strength that makes UHPC an ideal material for prestressed concrete piles constructed using this material. Other benefits regarding the use of UHPC is that little to no shrinkage occurs after the steam heat treatment has been applied and the material has a very high modulus of elasticity, approaching 8000 ksi (55 MPa).

By taking advantage of the self-levelling nature of UHPC, and the presence of steel fibers, UHPC piles can be constructed using a reduced cross section and no mild steel reinforcement. The high material strengths can be used to optimize the cross-sectional properties such that the UHPC piles have weights and stiffnesses similar to those of standard steel piles. The optimized H-section improves drivability of UHPC piles, while the high material strengths effectively prevent damage during driving.[17]

According to researchers, UHPC piles are very durable. Most importantly, the capillary porosity is very low, the material is extremely resistant to chloride permeability, and there is almost no deterioration due to freezing and thawing. It is anticipated that UHPC piles may reduce maintenance costs compared with traditional steel and prestressed concrete pile applications.

Continued research on UHPC piles has included the development of connection details for pile splices and details for anchoring piles into pile caps and bridge abutments.[18] The splices have been tested in the laboratory and in the field and the authors have noted that the results are excellent. The current connection detail to the pile cap is a simple pile embedment connection that has been shown to develop the pile flexural capacity without having any cracking in the connection region.[19] Following a successful detailed geotechnical investigation including driving of test piles, the Iowa Department of Transportation installed a 60 ft (18 m) long UHPC H-pile, to replace a steel H-pile in an integral abutment bridge.[20] Because UHPC piles require no special installation methods or additional labor costs, the only limiting factor is the cost of the UHPC itself, which is significantly more expensive than the traditional concrete used for prestressed concrete piles. However, it is expected that as UHPC becomes more readily available, and thus more affordable, and as researchers help develop recommendations on the use of these piles in accordance with governing codes and standards, UHPC pile use will replace applications where steel H-piles and pipe piles would once have been selected.

6.2 High-performance fiber-reinforced concrete (HPFRC) piles

HPFRC is constructed by adding hooked and twisted steel fibers to a traditional concrete mixture. The concrete compressive strength remains as expected, but the special fiber reinforcement actually helps create a concrete that exhibits a tensile strain hardening phenomenon when the concrete goes into tension, and when in compression the concrete actually acts more like well-confined concrete. Noting that the prestressed concrete pile industry relies on transverse reinforcement to create confined concrete cores needed for seismic resistance and for resisting driving stresses, the use of HPFRC for piles appears to be extremely promising.

Although no known research currently exists on this topic, it is anticipated that the use of HPFRC for prestressed concrete piles may result in a reduction in required transverse reinforcement needed for shear resistance, seismic ductility, and possibly even the resistance of driving stresses. In 2014, the results of a very similar application were presented by Aviram et al.[21] In this study, the authors show clearly that the use of HPFRC in reinforced concrete columns (1.5% to 2% steel fibers by volume) may result in an over 50% reduction in transverse reinforcement required. In fact, experimental testing showed that when compared with the results of a seismically detailed column meeting the Caltrans Seismic Design Criteria, a similar HPFRC column with the same longitudinal steel reinforcement but only half the spiral reinforcement quantity had significantly more ductility, significantly more energy dissipation capacity, and significantly less damage at the same drift ratios, while actually resisting a larger shear demand during the testing.

Although it is not yet proven, it is expected that the use of HPFRC piles may allow for a reduction in required transverse reinforcement for seismic resistance while still meeting the performance-based design provisions previously presented in this report. It is unclear whether the presence of the steel fibers will allow a reduction in transverse reinforcement to a level less than the minimum prescriptive requirement for driving. Another limiting factor is the additional cost of the steel fibers, which may or may not offset the cost savings that result from reduced transverse reinforcement. Even if the costs are simply competitive, the use of HPFRC may result in significantly less steel congestion in areas of high seismicity.

Figure 6.1. Ultra-high-performance concrete pile end after driving.

6.3 Fiber-reinforced-polymer reinforcement

While widespread use of composite materials to replace strands and deformed bars in prestressed concrete piles is not likely in the near future, their use as external transverse reinforcement for prestressed concrete piles may gain more rapid acceptance. Fiberglass and carbon fibers embedded in polymeric resins are already being used to wrap columns and other concrete members in repair, strengthening, and seismic applications. This same principle is being employed by at least one piling manufacturer to encase prestressed concrete piling in a composite shell. Although composite shells may eventually be used in place of steel spiral reinforcement, presently their primary purpose is for corrosion protection in marine applications and other aggressive environments.

Because the use of composite materials is a developing technology, it is essential that designers verify the acceptability and availability of their use in piles before incorporating them into a project. Pile manufacturers should be consulted concerning recent advances and the availability of products in a certain region. They are generally aware of projects and locations where a particular piling product has been used. The designer should also verify that the intended product meets the specifications governing the project, or if necessary, that special provisions are written to ensure the owner's requirements are met.

6.4 Pile-driving vibrations

The installation of driven piles produces ground vibrations that, if not monitored and controlled, can be considered anywhere from a general annoyance to the actual source of minor or even major structural damage to an adjacent structure. The magnitude or level of vibrations caused by pile driving is usually measured in the field as peak particle velocity (PPV). In the context of this report, the PPV is the maximum speed of a particle on the ground surface at a given location as it oscillates about a point of equilibrium that is moved by a passing wave generated by the pile-driving process. It is well known that the PPV is directly related to the physical properties of the pile, input energy associated with the pile installation method, and the geotechnical properties of the soil into which the pile is driven. For more information and theory related to the mechanics of ground motion caused by pile driving, the reader is referred to Woods (**Table 6.1**).[22] Although there is no consensus on precisely what limiting PPV values prevent vibration-induced damage and human complaints, much research over the past 40 years has attempted to quantify these values for certain adjacent structural system types and governing geotechnical conditions.

According to Woods,[22] there are three primary concerns regarding pile-driving-induced vibrations: human response, direct damage to structures, and vibration-induced settlement. Human response is the most difficult to quantify because sensitively levels for humans vary from person to person and are affected by external factors such as ambient or background vibrations and noise. Small vibrations that are perceptible to humans but are not damaging to structures can generate complaints from adjacent residents and building occupants. As shown in **Fig. 6.2**, Bay[23] presents a summary of human perceptibility levels that indicates that very disturbing levels of pile-driving vibrations typically occur at PPV values greater than 1 in./sec (25.4 mm/sec). Note that even PPV values as low as 0.01 in./sec (0.254 mm/sec) may still be perceptible.

The most commonly referenced PPV limits for preventing damage to adjacent structures appears to be the United States Bureau of Mines (USBM) recommendations presented by Siskind et al.[24] **Figure 6.3** shows these limits graphically, and although developed for blast-generated vibrations, they have been applied to pile-driving vibrations throughout the literature. The limit is intended to prevent cosmetic damage (such as cracking of plaster or wallboard joints) and the structural damage threshold would be much higher. The flat portion of the PPV limits shown as 0.5 in./sec (13 mm/sec) for plaster and 0.75 in./sec (19 mm/sec) for the drywall are shown for a frequency range associated with resonant frequencies for residential structures. Some vibration criteria have been developed for historic structures, such as Konon and Schuring.[25] The German Institute for Standardization criteria were developed for different types of structures such as residential structures and office buildings.[26] Figure 6.3 presents the Konon and Schuring criteria and the German Institute for Standardization criteria are also presented for comparison. In line with theory, all the aforementioned criteria provide PPV limits that are a function of vibration frequency, but conservative absolute limits are suggested as well.[22] It should also be noted that state-specific (such as Mingjiang et al.[27]) and city-specific (Hajduk et al.[28]) pile-driving vibration criteria have been proposed in the literature with evidence of successful use.

Vibration-induced settlement is also a major concern because it can lead to structural damage on adjacent property. Although pile-driving-induced settlement of cohesive soils is not common, the presence of loose sandy soils is a major concern during pile driving. When such soils are present, reductions in PPV limits presented above should be considered.

Because no national standard exists in regard to controlling pile-driving-induced vibrations, it is recommended that each project have a project-specific pile-driving vibration monitoring specification and plan that considers application of the following best practices as established in the literature (for example, Woods[22] and Mingjiang et al.[27]) and well known to those involved in the pile-driving industry.

To immediately begin to address the human perception factor, the pile-driving contractor should contact residents and adjacent business establishments that are within a project-specific area to inform them that piles will be driven nearby and that precautions are being taken to ensure no damage will occur on adjacent properties. Next, a preconstruction survey should be performed to determine the susceptibility of adjacent structures to permanent ground movements, cosmetic cracking, structural damage, and temporary impacts on sensitive equipment. The preconstruction survey results are used to select limiting PPV values and to confirm the adequacy of site-specific construction survey area and vibration monitoring area.

After the project criteria have been selected, a preconstruction building survey should be performed so that the existing conditions of neighboring buildings can be documented as necessary to show that new damage has not occurred as a result of pile-driving operations. Ideally, both interior and exterior surveys should be performed, but access is not always possible. Engineering notes, photographs, and video records are all recommended. During pile installation, all three components (longitudinal, transverse, and vertical) of particle velocity should be measured on the ground and the largest value used as the controlling value for comparison with the aforementioned PPV limits. Because the PPV will decrease with distance, the measurements should be performed at the locations of structures of interest or at comparable distances from current pile-driving activities.

Table 6.1. Frequency independent peak particle velocity limits recommended by Woods

Structure and condition	Limiting particle velocity, in./sec
Historic and some old structures	0.1 to 0.5*
Residential structures	0.5
New residential structures	1.0
Industrial building	2.0
Bridges	2.0

Note: 1 in./sec = 25 mm/sec.

* The 0.5 value given in Woods (1997) may be unconservative; see, for example, Mingjiang et al. (2012).

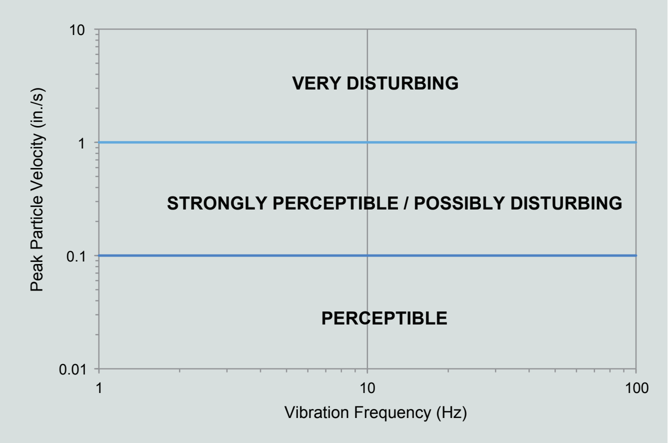

Figure 6.2. Perceptibility of vibration levels for humans. Source: Adapted from Bay (2003). **Note:** 1 in./sec = 25 mm/sec.

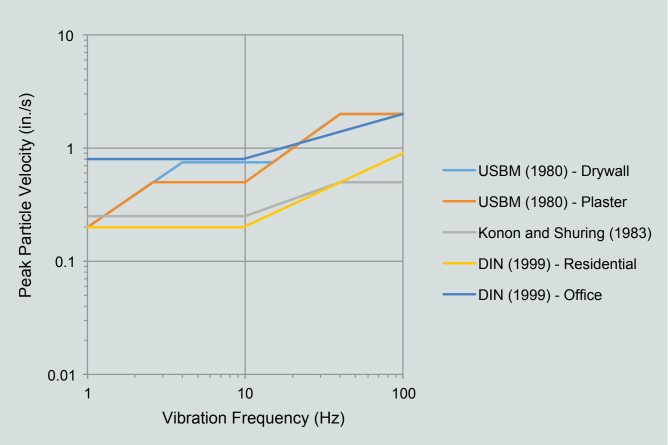

Figure 6.3. Commonly cited vibration criteria currently in use. Source: Adapted from Woods (1997). **Note:** 1 in./sec = 25 mm/sec.

Section 7—References

1. PCI Committee on Prestressed Concrete Piling. 1993. "Recommended Practice for Design, Manufacture and Installation of Prestressed Concrete Piling." *PCI Journal* 38 (2): 64–83.

2. ACI (American Concrete Institute) Committee 318. 2014. *Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary (318R-14)*. Farmington Hills, MI: ACI.

3. ACI Committee 543. 2012. *Guide to Design, Manufacture, and Installation of Concrete Piles (543R-12)*. Farmington Hills, MI: ACI.

4. ACI Committee 201. 2008. *Guide to Durable Concrete (201.2R-08)*. Farmington Hills, MI: ACI.

5. Allin, R., G. Likins, and J. Honeycutt. 2015. Pile Driving Formulas Revisited. In *Proceedings of the International Foundations Congress and Equipment Expo 2015*, March 17–21, 2015, San Antonio, Texas. Reston, VA: ASCE (American Society of Civil Engineers).

6. AASHTO (American Association of State Highway and Transportation Officials). 2014. *AASHTO LRFD Bridge Design Specifications*. 7th ed. Washington, DC: AASHTO.

7. Mander, J. B., M. J. N. Priestley, and R. Park. 1988. "Theoretical Stress-Strain Behavior of Confined Concrete." *Journal of the Structural Division* 114 (8): 1804–1825.

8. ICC (International Code Council). 2015. *International Building Code*. Falls Church, VA: ICC.

9. Priestley, M. J. N., F. Seible, and M. Calvi. 1996. *Seismic Design and Retrofit of Bridges*. New York: John Wiley & Sons.

10. MOTEMS (Marine Oil Terminal Engineering and Maintenance Standards). 2016. *Chapter 31F, Marine Oil Terminals*. Title 24, California Code of Regulations, Part 2, California Building Code. Long Beach, CA: California State Lands Commission.

11. Sritharan, S., A. Cox, J. Huang, M. Suleiman, and K. Arulmoli. 2016. “Minimum Confinement Reinforcement for Prestressed Concrete Piles and a Rational Seismic Design Framework.” *PCI Journal* 61 (1): 51–69.

12. ASCE. 2010. *Minimum Design Loads for Buildings and Other Structures*. ASCE/SEI 7-10. Reston, VA: ASCE.

13. Standards New Zealand. 2006. *Concrete Structures Standard*. NZS 3101.1&2. Wellington, New Zealand: Standards New Zealand.

14. ATC (Applied Technology Council). 1996. *Improved Seismic Design Criteria for California Bridges: Provisional Recommendations*. ATC-32. Redwood City, CA: ATC.

15. ASCE. 2014. *Seismic Design of Piers and Wharfs*. ASCE/COPRI 61-14. Reston, VA: ASCE.

16. Goel, R. K. 2015. “Evaluation of In-Ground Plastic-Hinge Length and Depth for Piles in Marine Oil Terminals.” *Earthquake Spectra* 31 (4): 2397–2417.

17. Vande Voort, T., M. T. Suleiman, and S. Sritharan. 2008. “Design and performance verification of ultra-high performance concrete piles for deep foundations.” Final Report, Iowa DOT IHRB Project TR-558 and CTRE Project 06-264. Center for Transportation Research and Education, Iowa State University, Ames, IA.

18. Aaleti, S., and S. Sritharan. 2016. Experimental and Analytical Investigation of UHPC Pile-to-Abutment Connections. In *First International Interactive Symposium on UHPC: Proceedings, July 18–20, Des Moines, Iowa*. http://dx.doi.org/10.21838/uhpc.2016.117.

19. Aaleti, S., J. Garder, and S. Sritharan. 2012. Experimental Evaluation of UHPC Piles with a Splice and Pile-to-Abutment Connection Performance. In *The PCI Convention and National Bridge Conference: Proceedings, September 29–October 2, 2012*, Nashville, Tennessee. Chicago, IL: PCI.

20. Garder, J. 2012. “Use of UHPC Piles in Integral Abutment Bridges.” MS thesis, Department of Civil, Construction & Environmental Engineering, Iowa State University, Ames, IA.

21. Aviram, A., B. Stojadinovic, and G. J. Parra-Montesinos. 2014. “High-Performance Fiber-Reinforced Concrete Bridge Columns under Bidirectional Cyclic Loading.” *ACI Structural Journal* 111 (2): 303–312.

22. Woods, R. D. 1997. *Dynamic Effects of Pile Installations on Adjacent Structures*. National Academy Press: Washington, D.C.

23. Bay, J. A. 2003. A Summary of the Research on Pile Driving Vibrations. In *7th Pile Driving Contractors Association Annual Winter Roundtable Conference Proceedings, February 21–22, 2003, Atlanta, Georgia*. Boulder, CO: Pile Driving Contractors Association.

24. Siskind, D. E., M. S. Stagg, J. W. Kopp, and C. H. Dowding. 1980. “Structure Response and Damage Produced by Ground Vibration from Surface Mine Blasting.” Report of Investigations 8507, United States Bureau of Mines, Washington, D.C.

25. Konon, W. and J. R. Schuring. 1983. “Vibration Criteria for Historic and Sensitive Buildings.” Preprint 83-501. ASCE.

26. DIN (German Institute for Standardization). 1999. *Vibrations in Buildings*. DIN 4150.

27. Mingjiang T. and M. Zhang. 2012. “Update LADOTD Policy on Pile Driving Vibration Management.” In *Proceedings of the 5th International Conference on Case Histories in Geotechnical Engineering*, New York, NY.

28. Hajduk, E. L., D. L. Ledford, and W. B. Wright. 2004. “Construction Vibration Monitoring in the Charleston, South Carolina Area,” In *Proceedings of the 5th International Conference on Case Histories in Geotechnical Engineering*, New York, NY.