PCI COMMITTEE REPORT

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

Prepared by

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Covers design of prestressed concrete piling, materials and manufacture, handling and transportation, and an extensive discussion of installation including best driving practices. Under design, allowable stresses are given, typical interaction charts show their application to piles subject to bending, and sample design problems suggest design approaches. Load testing is also covered. Selected references and reference standards are included.

CONTENTS

CHAPTER 1 — INTRODUCTION

Prestressed concrete piling are vital elements in the foundations of buildings, bridges and marine structures throughout the world. They vary in size from 10 in. (254 mm) square piles used in building foundations to the 66 in. (1676 mm) diameter cylinder 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 have come to be the usual method of construction, having been proven the logical choice of materials where permanence, durability and economy must be considered.

Heavy marine structures often rely on prestressed concrete piling driven through deep water or through deep layers of unsuitable material for their support. Prestressed concrete piling can be designed to safely support these heavy 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 based on current knowledge and standards. It updates the original report published in the March-April 1977 PCI JOURNAL.

Although the typical pile sections included are those most commonly found throughout the United States and Canada, the intent of this report is not to limit the shape of prestressed concrete piling. Local prestressed concrete manufacturers should be consulted for readily available cross sections.

Design examples, tables and details presented are intended as helpful aids to the qualified designer. Actual design details, including the selection of materials, pile sizes and shapes, should conform to local practices and code requirements.

1.1.1 Design

Chapter 2 discusses various factors that should be considered in the design of prestressed concrete piles and pile foundations. Data are presented to assist the design engineer in evaluating and providing for factors that affect the load-carrying capacities of prestressed concrete piles. Design aids are provided including interaction diagrams for the ultimate capacity of commonly used pile sizes in concrete strengths of 5000 to 8000 psi (34.5 to 55.2 MPa) and effective prestress levels of 700 and 1200 psi (4.8 and 8.3 MPa).

1.1.2 Materials

Chapter 3 discusses cements, aggregates, water, admixtures and reinforcement. Recommendations are made regarding these constituents and their effect on quality and strength of concrete.

1.1.3 Manufacture and Transportation

Special requirements involved in the manufacture, handling, transporting and tolerances for prestressed piling are covered in Chapter 4.

1.1.4 Installation

The purpose of Chapter 5 is to set forth general principles for proper prestressed piling installation to maintain structural integrity and design purpose of the piles.

Discussion of handling and driving induced compression, tension, bending and torsion provides a basis for recommendations to prevent damage to prestressed piles.

Piles occasionally must be cut off or extended, and suggestions are made on methods and techniques.

1.2 ASTM STANDARDS

ASTM A82 — Standard Specification for Steel Wire, Plain, for Concrete Reinforcement

ASTM A416 — Standard Specification for Steel Strand, Uncoated Seven-Wire for Prestressed Concrete

ASTM A421 — Standard Specification for Uncoated Stress-Relieved Steel Wire for Prestressed Concrete

ASTM A615 — Standard Specification for Deformed and Plain Billet-Steel Bars for Concrete Reinforcement

ASTM A722 — Standard Specification for Uncoated High-Strength Steel Bar for Prestressing Concrete

ASTM A882 — Standard Specification for Epoxy-Coated Seven-Wire Prestressing Steel Strand

ASTM A884 — Standard Specification for Epoxy-Coated Steel Wire and Welded Wire Fabric for Reinforcement

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 C360 — Standard Test Method for Ball Penetration in Fresh Portland Cement Concrete

ASTM C494 — Standard Specification for Chemical Admixtures for Concrete

ASTM D1143 — Standard Method of Testing Piles Under Static Axial Compressive Load

ASTM D3689 — Standard Method for Testing Individual Piles Under Static Axial Tensile Load

1.3 ACI STANDARDS

ACI 318-89 — Building Code Requirements for Reinforced Concrete, American Concrete Institute

ACI 543R-74(80) — Recommendations for Design, Manufacture and Installation of Concrete Piles

1.4 PCI STANDARDS AND REFERENCES

PCI MNL-116-85 — Manual for Quality Control for Plants and Production of Precast Prestressed Concrete Products

PCI MNL-120-92 — PCI Design Handbook — Precast and Prestressed Concrete

PCI STD-112-84 — Standard Prestressed Concrete Piles

PCI STD-113-86 — Prestressed Concrete Sheetpiles

CHAPTER 2 — DESIGN

2.1 GENERAL DESIGN CONSIDERATIONS

This chapter covers the important design considerations and presents design aids for prestressed concrete piling. Included are interaction curves presented to assist the designer in selecting piles subjected to combined axial load and bending moment.

2.1.1 Load Capacity of Individual Piles

The failure of an individual pile or of a pile group can occur when the applied load on the pile exceeds both the ultimate shearing resistance of the soil along the sides of the pile and the resistance of the bearing strata underneath the pile tip.

The ultimate bearing capacity of a pile is generally governed by the pile-soil system which is usually limited by the strength of the soil itself and not by the structural strength of the pile. This is especially true for piles completely embedded in the soil. Commonly used methods to evaluate the capacity of the pile include:

(a) Static load testing

(b) Static resistance analysis based on soils data and soils mechanics principles

(c) Dynamic analysis

All evaluation methods for pile capacity may be used in the selection of the appropriate pile design and installation. Frequently, the methods are combined to obtain the final design. The pile foundation for any specific project must, as with any other structural element, be designed by a qualified professional engineer to meet the particular conditions and applicable codes.

2.1.1.1 Static Load Testing

On larger, more important projects, it is economically feasible and technically advisable to perform load tests. Load tests accomplish three important purposes:

1. Provide accurate information regarding the load-carrying capacity of the pile-soil system.

2. Disclose the feasibility of installing the pile under specified means with the specified penetration to achieve test capacity.

3. Verify predictions as to required pile length.

Satisfactory performance of the test pile under loads equal to 1.5 to 2.0 times the proposed design capacity should be required. A safety factor of two is recommended for the pile-soil capacity. Piles subject to uplift may require greater factors of safety against uplift. Wherever possible, load testing to failure is strongly recommended, since this discloses the real safety factor inherent in the design, may lead to a more economical redesign, and provides valuable data both to the designer and to the profession as a whole.

If carried to failure, load tests furnish the most reliable measure of the load-carrying capacity of the pile-soil system. A load test reflects the properties and interaction of the pile-soil system and detects simultaneously any structural weakness of the pile. It provides an excellent basis for evaluating the physical or empirical constants contained in theoretical bearing capacity formulas.

The results of a pile load test are strictly applicable only for the pile or pile group tested. For this reason, it is important that a sufficient number of borings be made and that driving resistances be obtained to disclose any dissimilarities between soil conditions at the test pile location and at other locations on the job site. It is also essential that job inspection and control ensure that all piles are installed consistent with the test pile.

Several criteria are in use for evaluating the results of the load test. In general, the pile must satisfactorily carry the service load with an adequate factor of safety. ASTM D1143 and D3689 cover standard test methods for load capacity and uplift capacity, respectively.

When limiting values of settlement are used as criteria for evaluating pile load tests, it should be kept in mind that a pile shortens elastically under applied loads, and that similar strains also occur in the completed structure. If piling stresses, material, and length are comparable, and materials penetrated are similar, deformations and displacements will likely result in uniform settlement of the whole structure without objectionable differential settlement. High capacity load tests may include unload and reload cycles to determine the elastic deformation of the pile. Where practical, strain rods or gauges may be incorporated in the pile to disclose the relative movements of the pile tip and head under test load.

2.1.1.2 Static Resistance Analysis

Soil properties derived from making borings and then performing laboratory and field tests of the samples collected can, when interpreted by experienced and qualified professionals, be used to estimate pile capacity. Capacity is estimated as the sum of the skin friction along the embedded length of the pile and the end bearing capacity at the pile tip. The designer must recognize that layered soils may provide quite different skin friction values. Piles driven through poor material into good material will derive skin friction only in the portion of the pile that is embedded in the good material.

2.1.1.3 Dynamic Analysis

Pile driving criteria based on resistance to penetration are of significant value and often indispensable to ensure that each pile is driven to uniform capacity. Such criteria minimize differential settlement due to variable sub-surface conditions by forcing each pile to obtain a resistance comparable to other piles, including the test pile. As long as the operation of the hammer is consistent, variations in depth, density and quality of the bearing strata can all be taken into account.

Most driving resistance formulas are semi-empirical and factors of safety vary. Attempting to assign empirical values to many variables in a formula may lead to erroneous results. The use of a pile driving formula in conjunction with load tests will best determine the applicability of the formula to specific pile-soil systems and driving conditions. In some areas, driving formulas have been successfully used when applied with experience and good judgment and with proper recognition of their limitations. Such formulas are generally more applicable to non-cohesive soils.

The driving formulas in general use today are based on Newtonian impact principles and, in their simplest form, tend to equate work done in moving the pile through a distance against a soil resistance.[1,2,3] Some formulas try to take into account the many energy losses within the hammer-cap-block-pile-soil system.

Various methods and approaches have been attempted to properly evaluate the large number of variables that could affect the results of a driving formula. In general, there is no indication that the more elaborate formulas are more reliable or that any one dynamic formula is equally applicable to all types of piles under all soil conditions.

A wave equation "dynamic" formula has been developed based on a one-dimensional wave equation.[4,5,6] This application of longitudinal wave transmission to the pile driving process results in a mathematically accurate expression describing the mechanics of stress wave travel along a pile after it has been hit by the hammer ram. The wave equation may be used, with the aid of computer programs, to analyze stresses in piles during driving and to make predictions of the capacity of the piles.

2.2 LOADS AND STRESSES

2.2.1 Handling Stresses

Flexural stresses should be investigated for all conditions of handling, i.e., lifting from casting beds, storage, transportation, and pitching from horizontal to vertical. The stress analysis should be based on the weight of the pile plus a 50 percent allowance (or greater depending on individual conditions) for impact, with tensile stresses limited to $6\sqrt{f_c'}$.

The pile should be designed to resist the stresses imposed upon it during handling as well as the service load stresses. Fig. 2.1 shows a 10 in. (254 mm) square pile 113 ft (34.4 m) long being loaded on a special trailer for transporting to the job site.

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, installation of an undamaged pile is the desired end result.

Chips and minor spalls, which may be created during handling, pitching and driving and which do not impair performance of the pile, should be allowed. Chips which expose steel reinforcment or otherwise impair performance must be repaired.

Fig. 2.1. Loading a 10 in. (254 mm), 113 ft (34.4 m) long prestressed concrete pile.

2.2.2 Driving Stresses

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 which could damage the pile usually do not occur. Minimum effective prestress levels to resist driving stresses are listed in Table 2.2. Various authorities throughout the world use prestress levels ranging from 600 to 1200 psi (4.14 to 8.27 MPa). Refer to Chapter 5 for a more complete discussion of driving stresses.

2.2.3 Service Load Stresses

Service load stresses are the result of several types of loading or combinations of loading.

2.2.3.1 Axially Loaded Compression Piles

The most common type of piles are those that resist axial compression loads. Such piles are used to transfer structure dead and live loads through an unsuitable medium to a firm bearing material below. Compression piles must be capable of safely supporting design loads without buckling. Many static load tests have been performed on long slender piling that were driven through very soft material and then into firm soil. These tests show that even very soft soils will provide lateral restraint and thereby prevent buckling.

2.2.3.2 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 the axial loads from the structure to the fully prestressed section of the pile. 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.

2.2.3.3 Moment Resisting Piles

Examples of moment resisting piles are sheet piles or fender piles.

1. Sheet pile details are shown in PCI Standard 113-86. Allowable bending stresses should conform to those prescribed in Table 2.2.

2. Fender piles are piles placed along the faces of piers and bulkheads to absorb impact from floating vessels and thereby protect the pier from damage. The fender pile must, therefore, be able to absorb energy by withstanding service berthing impacts with little or no damage.

The U.S. Navy[7] has sponsored research for the testing of prestressed concrete fender pile concepts. The tests demonstrated the cyclic behavior of prestressed concrete fender piles in flexure. The behavior of spiral reinforcement in square and circular patterns was observed.

Prestressed concrete fender piles can provide an efficient means of protecting shore structures from damage due to floating vessels, and successful projects have been completed by the Navy on both the east and west coasts of the United States.

2.2.3.4 Combined Loadings

A pile under the effects of combined loading is one that is subjected to a bending moment and an axial load. 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.

Piles subjected to a compressive load and a bending moment may be categorized as either having full lateral support or limited lateral support, depending on the physical pile dimensions and restraint conditions. Long unsupported piles should be investigated for buckling. Load-moment interaction diagrams described in Section 2.4.2 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 2.2.

2.2.3.5 Lateral Loadings

Lateral forces may be the result of wind, earthquake, waves, ice or earth pressure. Many structures have been successfully designed using batter piles to resist lateral loads. Fig. 2.2 shows batter piles driven to support a pier.

When batter piles are used to resist seismic loads, it is important to consider the intense reaction forces, both vertically and laterally, that batter piles can impose on pile caps and footings. Margason[8] describes the interaction of batter piles and pile caps during an earthquake.

When vertical piles are used to resist lateral loads, the piles are subjected to bending moments that must be considered in the design of the piles. There are several methods available to predict moments, deflections and inflection points along the moment curve. Some methods are more sophisticated than others and require the use of a computer. These methods are based upon making use of the soil modulus and the beam-on-elastic foundation theory as applied by Reese and Matlock.[9,10]

One method is presented in NAVFAC DM-7.2[11] and analysis by this method can be made by hand calculations. Other methods involving the use of commercially available computer programs involve the development of P-Δ curves which allow a soil profile to be developed for the computer.

A simple analysis for lateral load design is included in Chapter 5 of the *Manual for the Design of Bridge Foundations.*[12] Design curves are included for calculating bending moments and deflections of laterally loaded piles in different types of soil.

Table 2.2. Concrete and steel allowable stresses.

CONCRETE STRESSES

a. Uniform axial compression for foundation piles fully embedded in soils providing lateral support ..... $0.33\,f_c' - 0.27 f_{pc}$

where

f_{pc} = effective prestress after all losses (see Note 1)

b. Uniform axial tension
- Permanent and repetitive 0
- Transient ..... $6\sqrt{f_c'}$

c. Compression due to bending ..... $0.45\,f_c'$
- AASHTO governed design[14] ..... $0.40\,f_c'$

d. Tension due to bending
- Permanent and repetitive ..... $4\sqrt{f_c'}$ or 0 in corrosive environment
- Transient ..... $6\sqrt{f_c'}$

e. For combined loading, concrete stresses should be checked in interaction:

$(f_a/F_a + f_b/F_b) \leq 1$

where

F_a = allowable direct stress

F_b = allowable compressive stress in bending (see Note 2)

f. Effective prestress (recommended values)
- For piles longer than 40 ft (12.2 m) ..... 700 to 1200 psi (4.8 to 8.3 MPa) (see Note 3)
- For piles shorter than 40 ft (12.2 m) ..... 400 to 700 psi (2.7 to 4.8 MPa)

STEEL STRESSES

a. Temporary stresses
- Due to temporary jacking force, but not greater than the maximum value recommended by the manufacturer of the steel ..... $0.80\,f_{pu}$ (max)
- Pretensioning tendons immediately after transfer, or post-tensioning tendons immediately after anchoring ..... $0.74\,f_{pu}$ (max)
- Tension due to transient loads ..... (concrete governs)

b. Effective prestress ..... $0.6\,f_{pu}$ or $0.80\,f_{py}$, whichever is smaller

c. Unstressed prestressing steel ..... $0.50\,f_{py}$ (max. 30,000 psi) (207 MPa)

Note 1: The designer should be aware that this formula for allowable stress has been derived by applying a safety factor of 2.2 to the ultimate pile strength. If design criteria are not compatible with this figure, adjustments are required.

Note 2: Conservative results are achieved using this method of analysis. When the pile is designed for combined axial load and bending, the working stress design should also be checked using ultimate strength design methods to ensure that the required minimum factor of safety is achieved in accordance with ACI 318-89.

Note 3: It is frequently advantageous for piles in the 40 to 140 ft (12 to 43 m) range to have an effective prestress of 700 to 1200 psi (4.8 to 8.3 MPa) to resist bending stresses during handling and tensile driving stresses. Chapter 5 provides additional information in regard to driving stresses.

2.3 ALLOWABLE SERVICE LOAD STRESSES

Stresses in prestressed concrete piles are either permanent (dead and live loads), repetitive (live loads), transient (wind, earthquake, etc.) or temporary (handling, driving). Since handling and driving stresses occur before the pile is put in service, the only reason for limiting such stresses is to ensure an initially undamaged pile in the ground.

The allowable recommended stresses are shown in Table 2.2.

2.4 DESIGN PROCEDURES

The following design procedures are based on pile strength. The ability of the soil to support these loads should be established by load tests or evaluated by soils engineers.

Two design procedures are suggested for consideration by the design engineer. The first is based on the commonly accepted building code formula for concentrically loaded short columns:

$$N = (0.33\,f_c' - 0.27\,f_{pc})A_g$$

where

N = allowable concentric load
f'_c = 28-day cylinder strength
f_{pc} = effective prestress
A_g = pile cross-sectional area

This method is presented in the PCI Design Handbook[24] and recommended for values of h'/r less than 60, where h' is the effective unsupported length of the pile and r is the radius of gyration of the transformed section of the pile.

The second procedure is based on the ultimate strength of the pile under combinations of axial load and moment and is recommended for values of h'/r greater than 60.

2.4.1 PCI Handbook Design

Table 2.4 provides section properties and allowable concentric service loads based on the formula:

$$N = (0.33\, f'_c - 0.27\, f_{pc})A_g$$

The table is based on a 700 psi (4.8 MPa) effective prestress and concrete strengths of 5000 to 8000 psi (34.5 to 55.2 MPa).

2.4.2 Ultimate Strength Method

2.4.2.1 Introduction

Example design aids have been presented which deal with the ultimate strength of prestressed concrete pile sections subject to axial loads and bending. Although the diagrams developed are applicable to prestressed concrete columns in general,[13] the potential for practical application is mainly related to the design of prestressed concrete piles. Application of this method is intended to help the designer take advantage of the full potential of prestressed concrete piles.

Separate interaction diagrams have been developed for the ultimate capacity of common pile sizes for strengths of 5000, 6000, 7000 and 8000 psi (34.5, 41.4, 48.3 and 55.2 MPa), and for effective prestress levels of 700 and 1200 psi (4.8 and 8.3 MPa). A full set of interaction diagrams is available from the Precast/Prestressed Concrete Institute. Note that the ultimate load, force or moment is referred to in the ACI 318-89 Building Code as design load, force or moment. Sample interaction diagrams are shown in Figs. 2.3 through 2.8. Pile section properties are the same as shown in Table 2.4.

2.4.2.2 Description of the Interaction Diagrams

These diagrams are essentially three-dimensional charts in which the parameters are: (1) axial load strength; (2) bending moment strength; and (3) slenderness ratio, h'/r.

The piles are assumed to act as prestressed concrete columns subjected to combinations of axial force and bending. A strength reduction factor, ϕ, of 0.7 has already been included in the diagrams for axial loads in the range from 100 percent down to 10 percent of the ultimate load, and below this level the ϕ factor increases linearly to a maximum of 0.9 for the case of pure flexure and axial tension. The sudden break in the curves results from this variation in ϕ.

Fig. 2.2. Driving a batter pile to support a pier.

2.4.3 Design Examples

Example 2.4.3.1 —
Pile-Supported Building

A building over a lake is to be supported on piles with an effective height, h', of 18 ft (5.49 m). The piles are required to carry a dead load of 80 kips (356 kN) and a live load of 80 kips (356 kN). In addition, each pile must resist a moment of 50 ft-kips (67.8 kN-m) due to earthquake.

Using the load factors from ACI 318-89 and a load factor reduction of 0.75 for combined dead load plus earthquake, determine acceptable pile sizes from interaction diagrams.

Condition I: ACI 318-89, Eq. (9-2), modified by Section 9.2.3:

$$P_u = [(1.4)(80) + (1.7)(80)]0.75 = 186 \text{ kips } (827 \text{ kN})$$
$$M_u = (1.7)(1.1)(50)(0.75) = 70 \text{ ft-kips } (94.9 \text{ kN-m})$$

Condition II: ACI 318-89, Eq. (9-3), modified by Section 9.2.3:

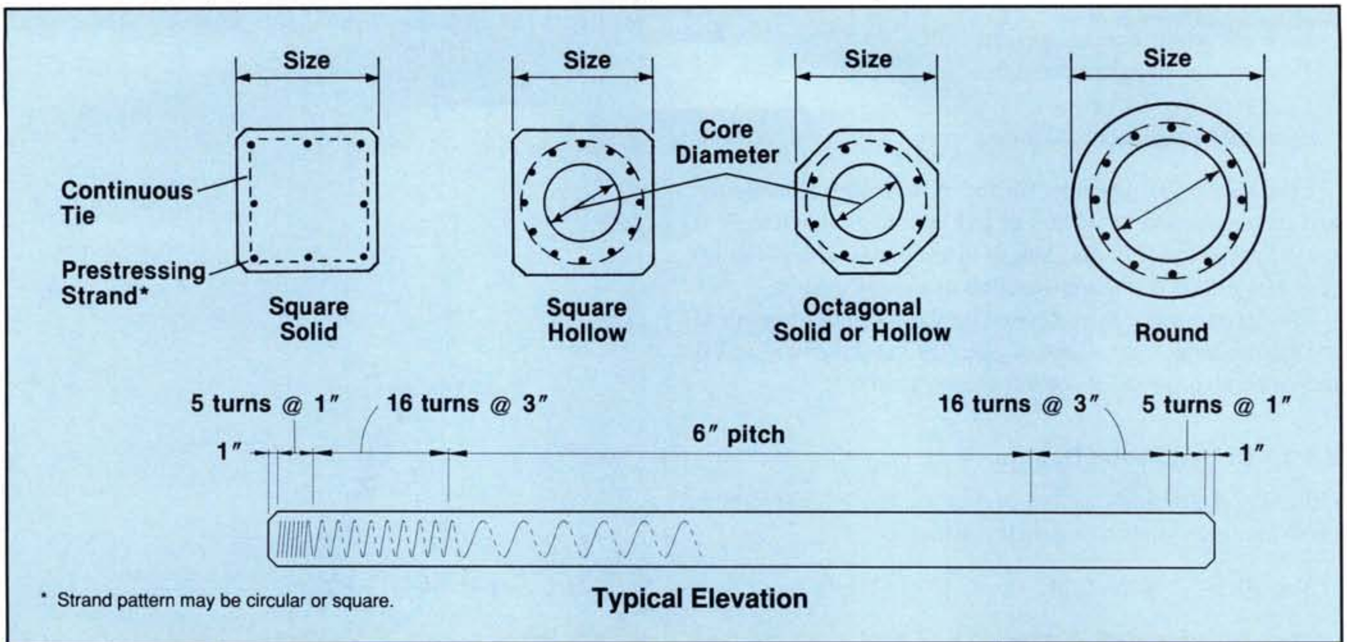
$$P_u = (0.9)(80) = 72 \text{ kips } (320 \text{ kN})$$
$$M_u = (1.3)(1.1)(50) = 71.5 \text{ ft-kips } (97 \text{ kN-m})$$

For a 16 in. octagonal pile (see Table 2.4):

$h'/r = (18)(12)/4.12 = 52.5$
f'_c = 5000 psi (34.5 MPa), 6 strands OK per Fig. 2.5
f'_c = 5000 psi (34.5 MPa), 10 strands OK per Fig. 2.6

Note: Special attention must be given to reinforcing steel details for moment in the transfer zone.

Table 2.4. Section properties and allowable service loads of prestressed concrete piles.



Size	Core Diameter	Section Properties[1]						Allowable Concentric Service Load, Tons[2]			
		Area	Weight	Moment of Inertia	Section Modulus	Radius of Gyration	Perimeter	f'_c			
(in.)	(in.)	(in.^{2})	(plf)	(in.^{4})	(in.^{3})	(in.)	(ft)	5,000	6,000	7,000	8,000
				Square Piles							
10	Solid	100	104	833	167	2.89	3.33	73	89	106	122
12	Solid	144	150	1,728	288	3.46	4.00	105	129	152	176
14	Solid	196	204	3,201	457	4.04	4.67	143	175	208	240
16	Solid	256	267	5,461	683	4.62	5.33	187	229	271	314
18	Solid	324	338	8,748	972	5.20	6.00	236	290	344	397
20	Solid	400	417	13,333	1,333	5.77	6.67	292	358	424	490
20	11 in.	305	318	12,615	1,262	6.43	6.67	222	273	323	373
24	Solid	576	600	27,648	2,304	6.93	8.00	420	515	610	705
24	12 in.	463	482	26,630	2,219	7.58	8.00	338	414	491	567
24	14 in.	422	439	25,762	2,147	7.81	8.00	308	377	447	517
24	15 in.	399	415	25,163	2,097	7.94	8.00	291	357	423	488
30	18 in.	646	672	62,347	4,157	9.82	10.00	471	578	685	791
36	18 in.	1,042	1,085	134,815	7,490	11.38	12.00	761	933	1,105	1,276
				Octagonal Piles							
10	Solid	83	85	555	111	2.59	2.76	60	74	88	101
12	Solid	119	125	1,134	189	3.09	3.31	86	106	126	145
14	Solid	162	169	2,105	301	3.60	3.87	118	145	172	198
16	Solid	212	220	3,592	449	4.12	4.42	154	189	224	259
18	Solid	268	280	5,705	639	4.61	4.97	195	240	284	328
20	Solid	331	345	8,770	877	5.15	5.52	241	296	351	405
20	11 in.	236	245	8,050	805	5.84	5.52	172	211	250	289
22	Solid	401	420	12,837	1,167	5.66	6.08	292	359	425	491
22	13 in.	268	280	11,440	1,040	6.53	6.08	195	240	283	328
24	Solid	477	495	18,180	1,515	6.17	6.63	348	427	506	584
24	15 in.	300	315	15,696	1,308	7.23	6.63	219	268	318	368
				Round Piles							
36	26 in.	487	507	60,007	3,334	11.10	9.43	355	436	516	596
42	32 in.	581	605	101,273	4,823	13.20	11.00	424	520	616	712
48	38 in.	675	703	158,222	6,592	15.31	12.57	493	604	715	827
54	44 in.	770	802	233,373	8,643	17.41	14.14	562	689	816	943
66	54 in.	1,131	1,178	514,027	15,577	21.32	17.28	826	1,013	1,199	1,386

(1) Form dimensions may vary with producers, with corresponding variations in section properties.

(2) Allowable loads based on $N = A_c\,(0.33\,f'_c - 0.27\,f_{pc})$; f_{pc} = 700 psi. Check local producer for available concrete strengths.

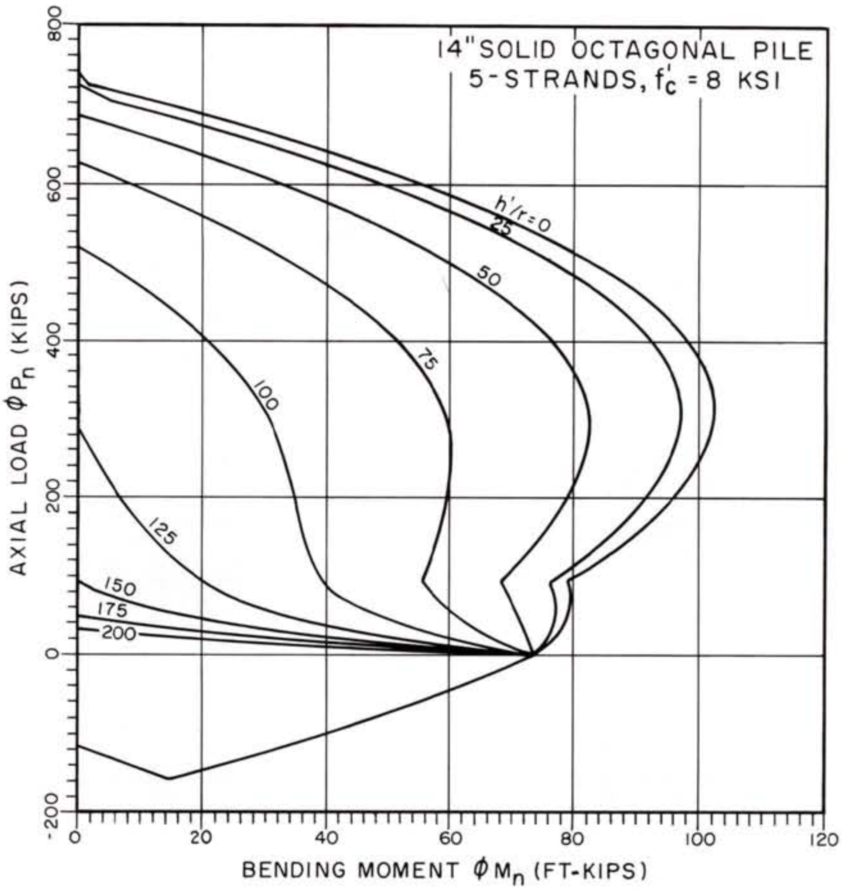


Fig. 2.3. Interaction curve for 14 in. (356 mm) octagonal pile.

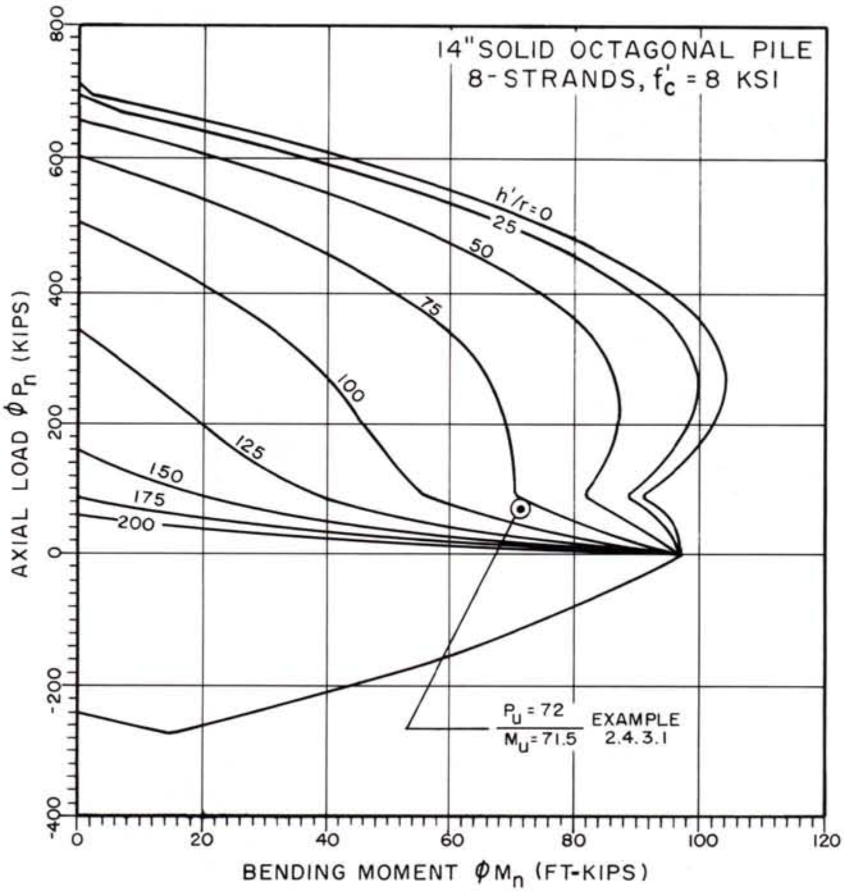


Fig. 2.4. Interaction curve for 14 in. (356 mm) octagonal pile.

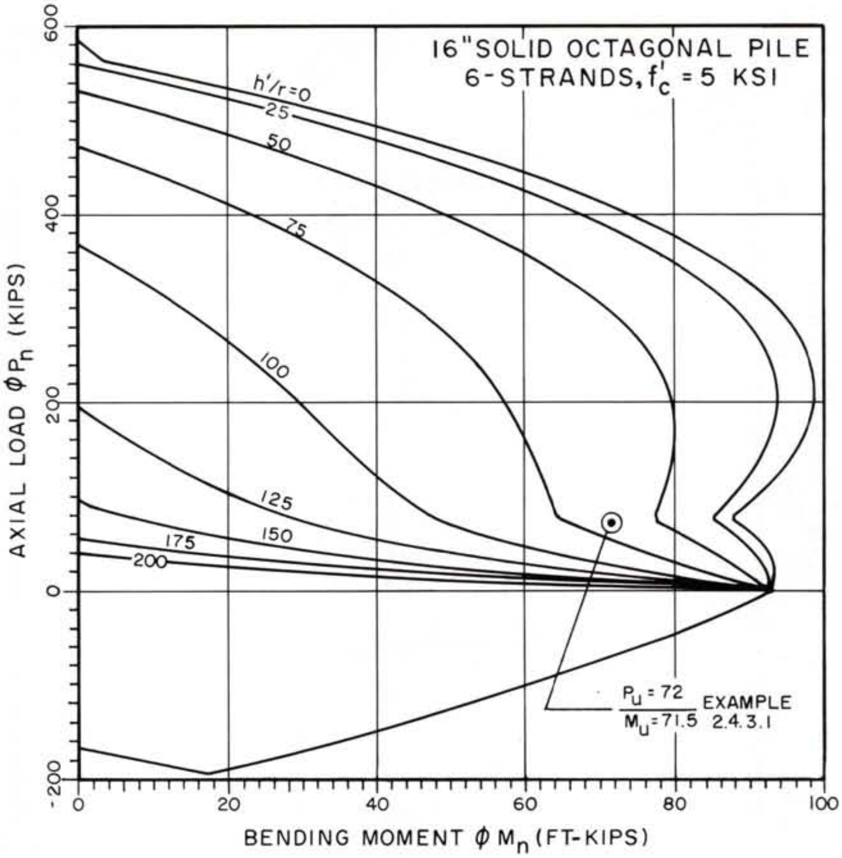


Fig. 2.5. Interaction curve for 16 in. (406 mm) octagonal pile.

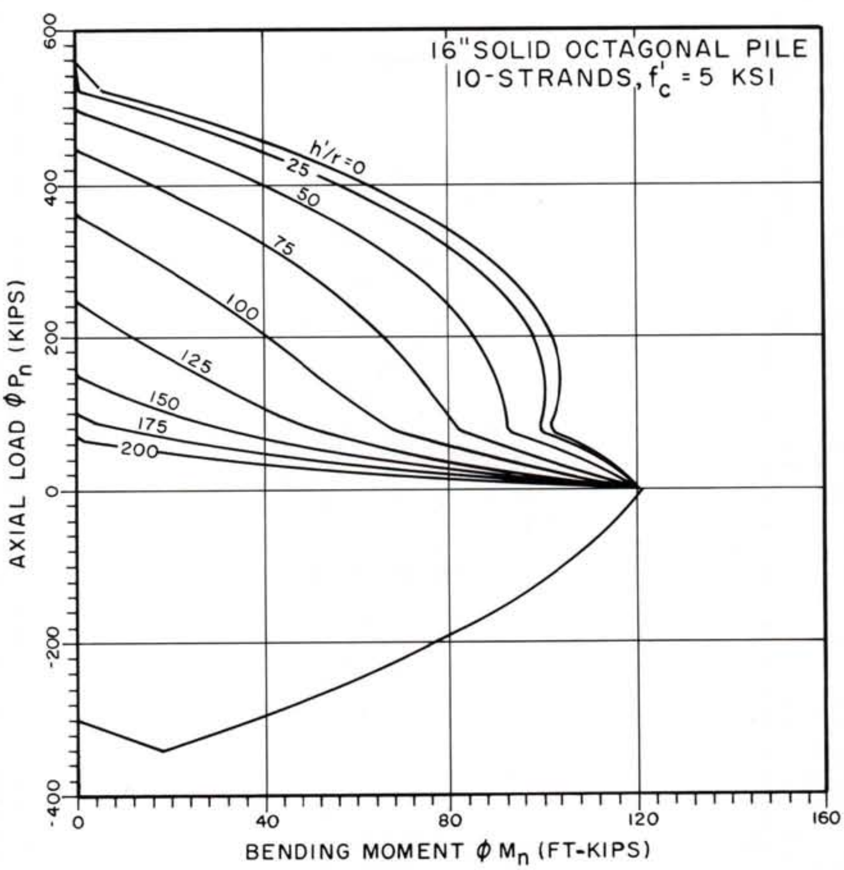


Fig. 2.6. Interaction curve for 16 in. (406 mm) octagonal pile.

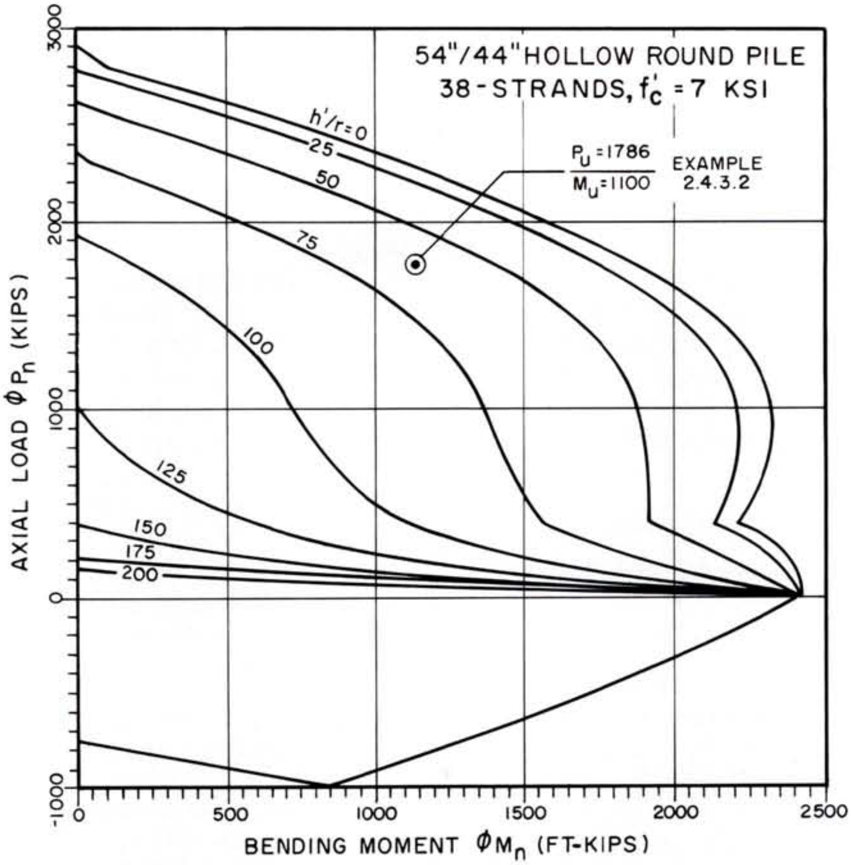


Fig. 2.7. Interaction curve for 54 in. (1372 mm) hollow, round pile.

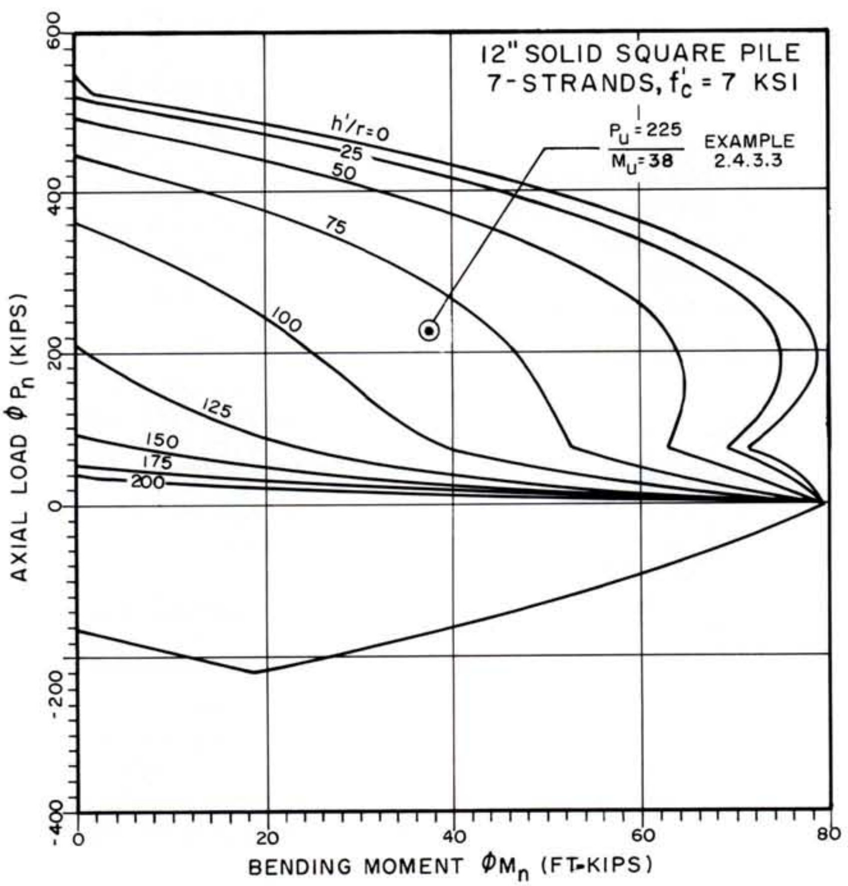


Fig. 2.8. Interaction curve for 12 in. (305 mm) square pile.

For a 14 in. (356 mm) octagonal pile (see Table 2.4):

$h'/r = (18)(12)/3.6 = 60$

$f'_c = 8000$ psi (55.2 MPa), 8 strands OK per Fig. 2.4

Example 2.4.3.2 —
Pile Columns for Long-Span Bridge

The bridge is to be supported on piles with an effective height, h', of 90 ft (27.4 m). The piles are required to carry a dead load of 1200 kips (5338 kN) and a live load of 80 kips (356 kN). Load factors for design are 1.3 times the sum of dead load plus 1.67 times live load including impact. (AASHTO Specifications,[14] Section 3.22). Assume an initial eccentricity of 0.1 times the pile diameter and make provisions for lateral creep deflection to determine effective eccentricity.

For a trial design, the initial ultimate load is:

$$
\begin{aligned}
P_u &= 1.3[D + 1.67 (L + I)] \\
&= (1.3)[(1200) + (1.67)(80 \times 1.3)] \\
&= 1786 \text{ kips } (7944 \text{ kN})
\end{aligned}
$$

The ratio of dead to total load is 1200/1280 = 0.94. Because of the very high ratio of permanent to total design load, assume the effective eccentricity is 50 percent greater than the initial eccentricity due to lateral creep deflection.

From inspection of Fig. 2.7, try a 54 in. (1372 mm) outside diameter, 44 in. (1118 mm) inside diameter hollow cylinder with 7000 psi (48.3 MPa) concrete and 38 strands. The slenderness ratio, h'/r, for this pile is (90)(12)/17.4 = 62, which appears reasonable for the first trial. Then the trial ultimate moment is:

$$
\begin{aligned}
M_u &= (1.5)(P_u)(0.1)(\text{diameter}) \\
&= (1.5)(1786)(5.4)/12 \\
&= 1205 \text{ ft-kips } (1634 \text{ kN-m})
\end{aligned}
$$

Next, investigate the creep deflection due to a sustained dead load, P_d, of 1200 kips (5338 kN) using one-half the elastic modulus (per AASHTO Section 3.22) for 7000 psi (48.3 MPa) concrete (2737 ksi), $h' = 1080$ in. (27.4 m), and $I = 233{,}400$ in.^{4} (9.71×10^{10} mm^4). Using the secant formula:

$$e_{effective} = e_{initial} \sec \sqrt{P_d h'^2 / 4EI}$$

The value under the radical term is:

$$(1200)(1080)^2/(4)(2737)(233{,}400) = 0.55$$

The value of the secant is 1.35 and $e_{effective} = 1.35\ (e_{initial})$. The design requirements are then compared with the capability of the 54 in. (1372 mm) outside diameter, 44 in. (1118 mm) inside diameter hollow cylinder pile with 7000 psi (48.3 MPa) concrete and 38 strands.

Required ultimate moment:

$$
\begin{aligned}
M_u &= 1786\,(0.1)(54/12)(1.35) \\
&= 1085 \text{ ft-kips } (1471 \text{ kN-m})
\end{aligned}
$$

Required capacity:

$P_u = 1786$ kips (7944 kN)
$M_u = 1085$ ft-kips (1471 kN-m)

Available capacity (from Fig. 2.7):

$\phi P_n = 1786$ kips (7944 kN)
$\phi M_n = 1100$ ft-kips (1492 kN-m)

Example 2.4.3.3 —
Overhead Bridge Craneway

An overhead bridge craneway is supported on piles carrying a dead load of 20 kips (89 kN) and a live load of 100 kips (445 kN), with an eccentricity of 0.1 times the pile size. The effective height of the pile is 24 ft (7.32 m).

For this structure, the load factors will be assumed to be 1.25 for dead load and 2.0 for live load. For the first trial, with the ratio of dead load to total load equal to 20/120, or 0.167, assume the ratio of effective to initial eccentricity is 1.2.

The ultimate load is [1.25(20) + (2)(100)] or 225 kips (1001 kN).

From inspection of Fig. 2.8, a 12 in. (305 mm) square pile, with $h'/r = (24)(12)/3.5 = 82$, $f'_c = 7000$ psi (48.3 MPa), and $f_{pc} = 1200$ psi (8.27 MPa), appears satisfactory for a trial design.

$P_u = 225$ kips (1001 kN)
$M_u = (225)(0.10)(1.2) = 27$ ft-kips (36.6 kN-m)

The magnification of eccentricity due to creep under dead load, $P_d = 20$ kips (89 kN), with $h' = 288$ in. (7.32 m), $E = (E_c/1.8) = 3042$ ksi (20975 MPa):

$I = 1728$ in.^{4} (7.19×10^8 mm^4) is next investigated:*

$$e_{effective} = e_{initial} \sec \sqrt{P_d h'^2 / 4EI}$$

The value under the radical term is:

$$(20)(288)^2/(4)(3042)(1728) = 0.08$$

The value of the secant is 1.04, and $e_{effective} = 1.04\ (e_{initial})$. The design requirements are then compared with the capacity of the 12 in. (305 mm) square pile, with $f'_c = 7000$ psi (48.3 MPa) and 7 strands.

Required capacity:

$P_u = 225$ kips (1001 kN)
$M_u = 23.4$ ft-kips (31.7 kN-m)

Available capacity (from Fig. 2.8):

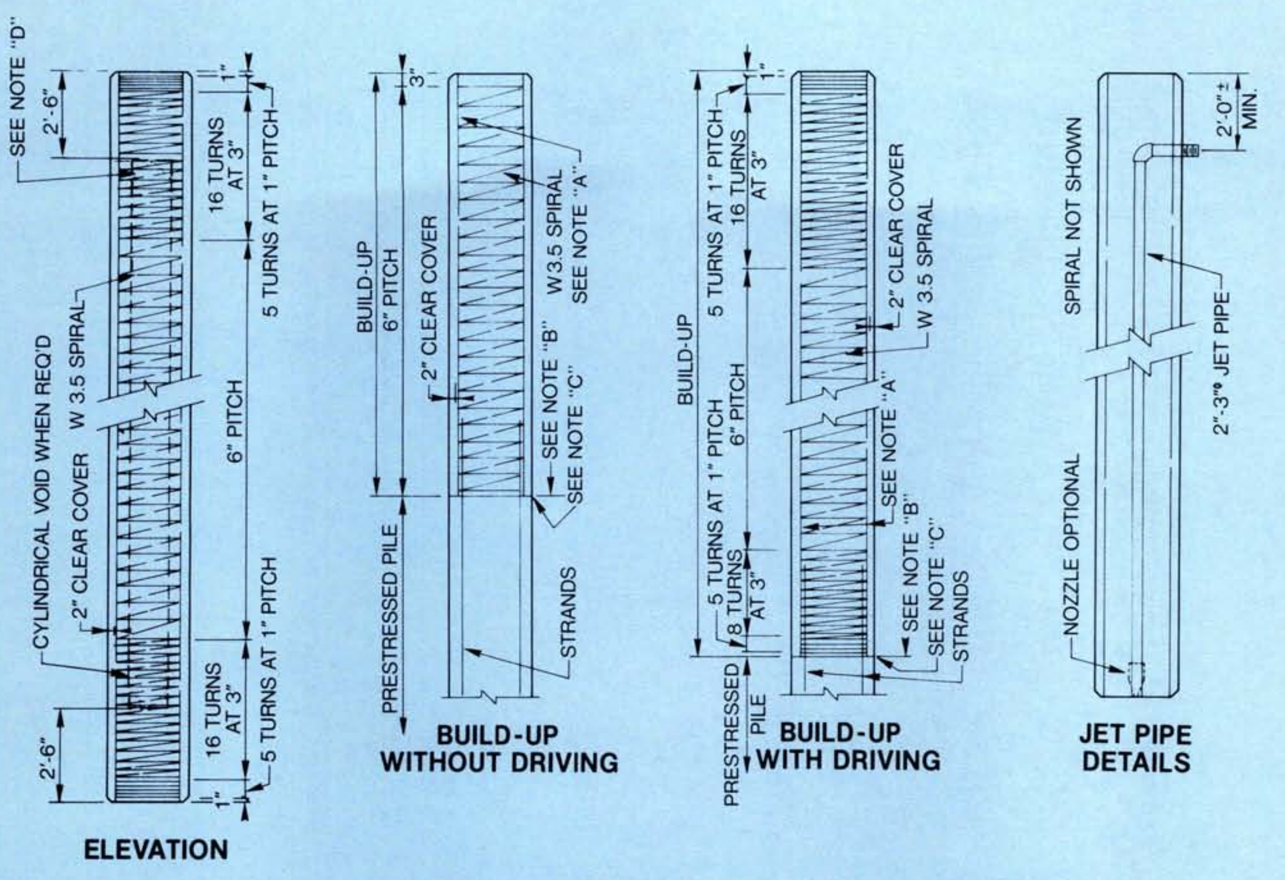
$\phi P_n = 225$ kips (1001 kN)
$\phi M_n = 38$ ft-kips (51.5 kN-m)

2.5 SPECIAL DESIGN CONSIDERATIONS

2.5.1 Spiral Reinforcement

Spiral reinforcement is a requirement for all prestressed concrete piles. Typical spiral details are shown in PCI Pile Standard 112-84 and in Figs. 2.9 and 2.10. For piles up to 24 in. (610 mm) in diameter, standard spiral wire should be W3.5 minimum. For piles having a diameter greater than 24

* For this example, creep is accounted for by dividing E by 1.8, and lateral loads are taken by other systems.



Notes on Build-up

Note A: The minimum area of reinforcing steel shall be 1½ percent of the gross cross-section of concrete. Placement of bars shall be in a symmetrical pattern of not less than four bars.

Note B: Method of attachment of pile to build-up may be by any of the methods given in the notes on alternate pile heads. If mild reinforcing steel is used for attachment, the area shall be no less than that used in the build-up.

Note C: Concrete around top portion of driven pile shall be bush-hammered to prevent feather edges in the cast-in-place build-up.

Note D: End fitting or form may be flat or tapered with proper taping to prevent leakage.

Note E: Additional mechanical and other types of splices are available to designers. Refer to PCI JOURNAL two-part series, "Splicing of Precast Prestressed Concrete Piles," September-October and November-December 1974.

Fig. 2.9. Typical details of pile reinforcement.

in. (610 mm), a larger diameter spiral may be required.

Spiral quantity has been referred to in two ways in the technical literature. Fig. 2.11 illustrates the two methods of nomenclature.

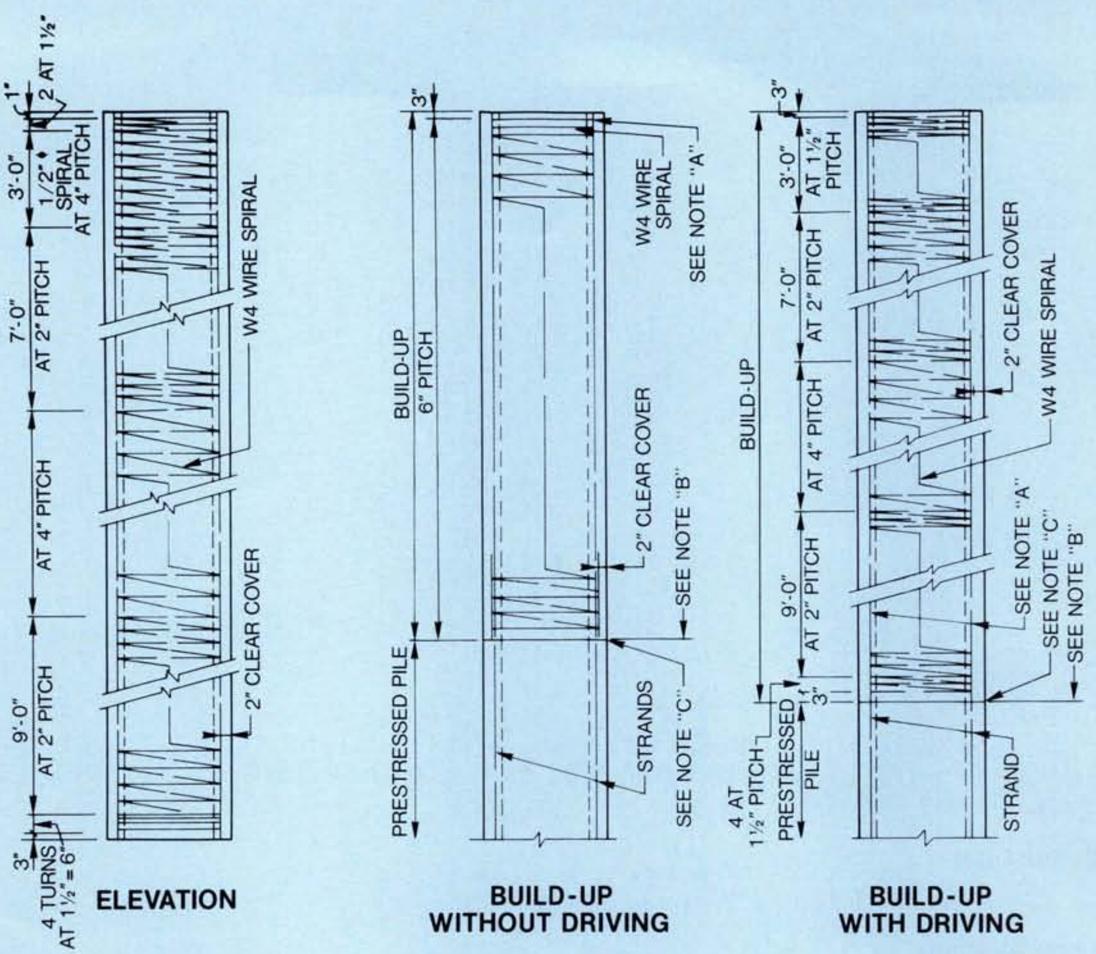
The top of large cylinder piles must be given special consideration depending on driving conditions. Hard driving can cause large bursting forces at the driving end of such piles. Empirically, it has been found that providing about 1 percent spiral area in the top 12 in. (305 mm) has been successful in preventing splitting of the cylinder pile head for diameters up to 54 in. (1372 mm).

Similarly, for hollow-core piles whose upper portions are exposed in water or air, the spiral reinforcement at yield should be adequate to develop the splitting tensile strength of the concrete. This will generally require a spiral area of between 0.6 and 1.0 percent.

If hollow-core piles are driven open ended, potentially damaging bursting stresses due to water hammer can occur if the void becomes filled with liquid or near-liquid material during driving. "Break away" diaphragm plugs to exclude water and mud during early driving through soft clays have been used successfully. Fluid removal by cleaning and pumping is often useful during penetration of water-bearing granular material.

2.5.2 Seismic Design

Prestressed concrete piling, like all other piles, will undergo imposed displacements and curvature under strong seismic action. When soils undergo cyclic movement, embedded piles are bent into curvatures of varying radii. The magnitude of these curvatures can be estimated by studying the acceleration-time history response of the specific soil profile.



Notes on Build-up

Note A: The minimum area of reinforcing steel shall be 1½ percent of the gross cross-section of concrete. Placement of bars shall be in a symmetrical pattern of not less than eight bars.

Note B: Method of attachment of pile to build-up may be by any of the methods given in the notes on alternate pile heads. If mild reinforcing steel is used for attachment, the area shall be no less than that used in the build-up.

Note C: Concrete around top portion of driven pile shall be bush-hammered to prevent feather edges in the cast-in-place build-up.

Fig. 2.10. Typical details of cylinder pile reinforcement.

The analysis method is usually based on the theory of a beam on an elastic foundation including provision for a variable soil modulus determined by pile deflection, depth and stratified soil deposits, pile flexural stiffness changes with depth, pile geometry and pile restraints at the interface to the superstructure. Curvatures may be particularly severe at abrupt changes in soil stiffness and at connections to the pile cap.

Prestressed concrete piling can be designed to accept these extreme conditions and still carry the design load if appropriate design and detailing provisions are made. Such criteria and special detailing are outlined below.

(a) Requirements of ACI 318, Chapter 21, other than Sections 21.0 and 21.1, need not apply unless specifically referenced.

(b) Where the total pile length in the soil is 35 ft (10.7 m) or less, the lateral transverse reinforcement in the ductile region shall occur through the length of the pile. Where the pile length exceeds 35 ft (10.7 m), the ductile pile region shall be taken as the greater of 35 ft (10.7 m) or the distance from the underside of the pile cap to the point of zero curvature plus three times the pile diameter.

(c) In the ductile region, the center to center spacing of the spirals or hoop reinforcement shall not exceed one-fifth of the pile diameter, six times the diameter of the longitudinal strand, or 8 in. (203 mm), whichever is smaller.

(d) Spiral reinforcement may be spliced by lapping one full turn and bending the end of the spiral to a 135 degree seismic hook, by welding or by the use of a mechanical connector which will develop 125 percent of the yield strength of the spiral.

(e) Where pile reinforcement extends into caps or grade beams, the lateral transverse reinforcement in this region may consist of reinforcement other than spiral reinforcing

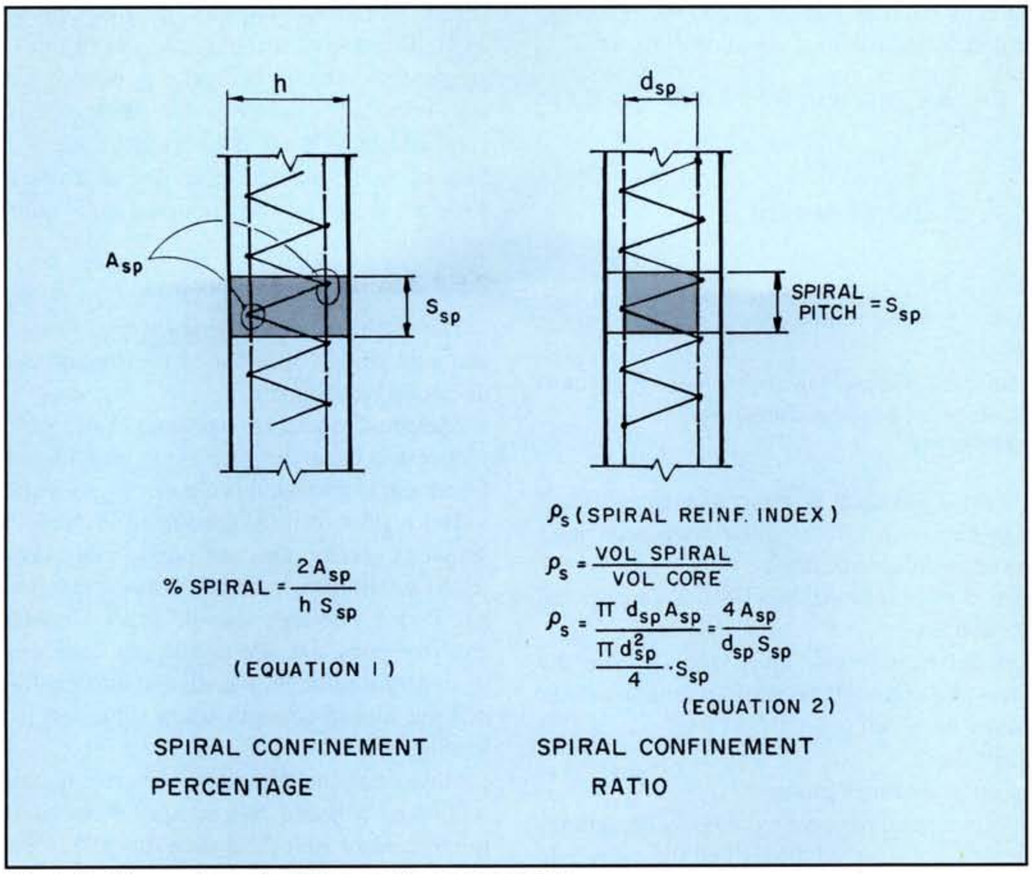


Fig. 2.11. Comparison of spiral quantity nomenclature.

steel. This lateral transverse reinforcement may consist of circular hoops. The hoops shall be deformed bars not less than #3 in size and spaced not greater than 3 in. (76 mm) on center. The circular hoops shall be terminated by a lap splice (Class B) and seismic hooks. The splices shall be staggered 180 degrees.

Certain types of soils may tend to liquefy during earthquakes. In instances where piles are designed as having full lateral support provided by surrounding soil, the likelihood of the surrounding soil liquefying during an earthquake must be considered. If liquefaction could occur, the portion of the pile which will extend through the very fluid soil should be considered unsupported when piles are designed to support lateral seismic loads.

2.5.2.1 Seismic Design of Spiral Reinforcement for Low to Moderate Seismic Risk Areas

The size and pitch of the spiral reinforcement necessary to provide ductility depends upon the amount of curvature anticipated during an earthquake. In the absence of analysis, as described in Section 2.5.2, to determine the magnitude of curvature for a specific site, spiral quantity can be estimated by empirical means. A minimum volumetric ratio of spiral reinforcement in the ductile region equal to 0.007 is recommended. The spiral reinforcement should not be less than the amount required by the following formula:

$$\rho_s = 0.12\, f_c'/f_y$$

where

ρ_s = spiral reinforcement index in Fig. 2.11
f_c' = 6000 psi (41.4 MPa)
f_{yh} = yield strength of spiral reinforcement $\leq$ 85 ksi (586 MPa)

0.45

2.5.2.2 Seismic Design of Spiral Reinforcement for High Seismic Risk Areas

(a) Where the transverse reinforcement consists of circular spirals, the volumetric ratio of spiral transverse reinforcement in the ductile region shall comply with:

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

but not less than

$$\rho_s = 0.12(f_c'/f_{yh})[0.5 + 1.4P/(f_c'A_g)]$$

where

ρ_s = spiral reinforcement index in Fig. 2.11
f_c' $\leq$ 6000 psi (41.4 MPa)
f_{yh} = yield strength of sprial reinforcement $\leq$ 85 ksi (586 MPa)
A_g = pile cross-sectional area
A_{ch} = core area defined by spiral outside diameter
P = axial load on pile

This required amount of spiral reinforcement may be obtained by providing an inner and outer spiral.

(b) When transverse reinforcement consists of rectangular hoops and cross ties, the total cross-sectional area of lateral transverse reinforcement in the ductile region with spacing, s, and perpendicular to dimension, h_c, shall conform to:

$$A_{sh} = 0.3\, sh_c(f'_c / f_{yh})(A_g/A_{ch} - 1.0)[0.5 + 1.4\, P/(f'_c A_g)]$$

but not less than

$$A_{sh} = 0.12\, sh_c(f'_c / f_{yh})[0.5 + 1.4P/(f'_c A_g)]$$

where

s = spacing of transverse reinforcement measured along length of pile
h_c = cross-sectional dimension of pile core measured center to center of hoop reinforcement
$f_{yh} \leq 70$ ksi (483 MPa)

The hoops and cross ties shall be equivalent to deformed bars not less than #3 in size. Rectangular hoop ends shall terminate at a corner with seismic hooks.

(c) The interaction effect of axial and flexural forces shall conform with Section 2.4.2.

(d) Pile splices between precast units shall develop the full strength of the pile and shall be allowed a minimum of 30 ft (9.14 m) below the lesser of:

(i) Bottom of pile cap

(ii) The lowest adjacent finish grade

The relatively heavy reinforcement required by Paragraphs (a) or (b) is generally not required throughout the entire pile length, but only necessary in the ductile region of the pile.

Fig. 2.12 illustrates the extreme congestion that can be created when large diameter spiral is required at a very close pitch. The cost of manufacturing such piles is considerably greater than the cost normally associated with making piles with spiral details discussed in Section 2.5.1. Both material costs and labor costs may be dramatically increased. Only piles subject to lateral loads due to earthquakes should be designed to include this increased spiral reinforcement.

Fig. 2.12. Congestion with heavy spiral reinforcement.

2.5.2.3 Seismic Research

The results of several series of tests to ascertain the elastic and post-elastic behavior of prestressed concrete piles are discussed by Sheppard.[15]

A method to analyze the flexural strength and ductility of prestressed concrete piles containing increased spiral reinforcement is presented in a paper by Joen and Park.[16]

The results of tests conducted in New Zealand on prestressed concrete piles and pile-to-pile cap connections subjected to simulated seismic loads are summarized by Joen and Park.[17] The tests showed that well-detailed prestressed concrete piles and pile-to-pile cap connections are capable of undergoing large post-elastic deformations without significant loss in strength when subjected to severe seismic loading.

Muguruma and Watanabe[18] reported on tests conducted on reinforced concrete column specimens having confinement reinforcement with yield strengths of 114.89 ksi (792 MPa). Test results indicated that a very large ductility could be achieved by using high yield strength lateral reinforcement.

Azizinamini, Corley and Johal[19] described the results of a test program which investigated the effects of different transverse reinforcement types and details on hinging of reinforced concrete columns subjected to loads simulating earthquake effects. The tests indicated an advantage in providing continuous square helical spiral over single peripheral hoops.

2.5.3 Splices and Build-ups

For the purpose of this chapter, splices are defined as any method of joining prestressed concrete pile sections in the field during driving so that driving may continue.[20] Fig. 2.13 shows a pile being spliced with a connector ring.

Fig. 2.13. Connector ring pile splice.

A pile may have one or more splices. Each splice must be capable of resisting all subsequent stresses and deformations which may be induced through driving or under service loads. Driving stresses, both tensile and compressive, may be estimated by use of wave equation analysis. The type of splice selected must depend primarily on service loads and conditions. Not all splices will develop moment or uplift.

Commonly used splices can be categorized generally as:

1. Welded
2. Mechanical locking
3. Connector ring
4. Wedge
5. Sleeve
6. Dowel
7. Post-tensioned

Typical splice details for these general types of splices are shown in Fig. 2.14.

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

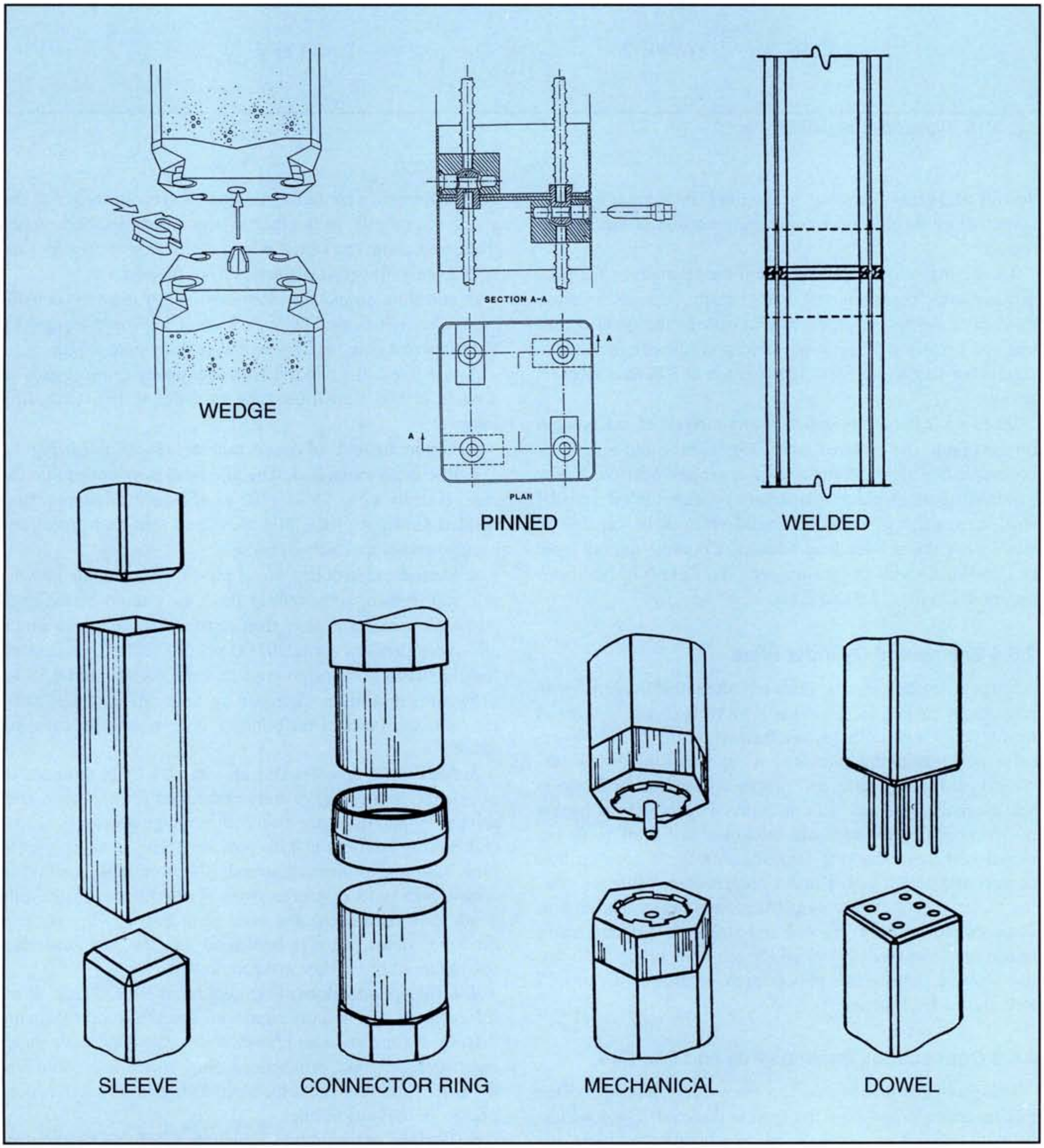


Fig. 2.14. Commonly used prestressed concrete pile splices.

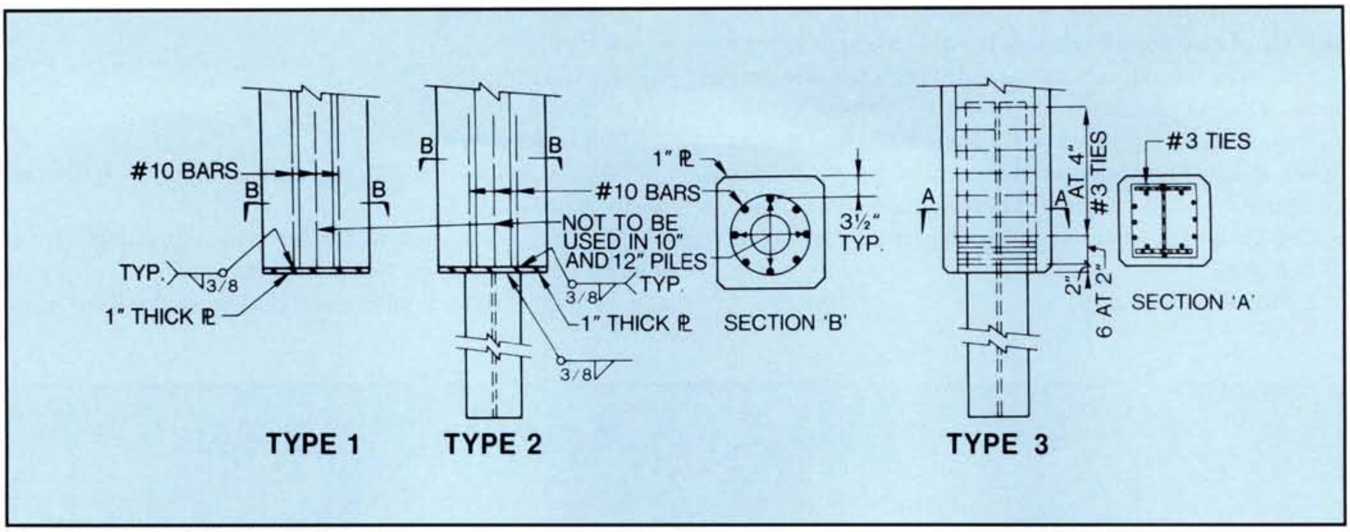


Fig. 2.15. Typical pile tip details.

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.

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 section, but is generally cast in place. Minimum recommended vertical steel area is 1.5 percent. The build-up must be capable of developing the service load stresses. Concrete quality must be compatible with the prestressed pile. Details of build-ups are shown in Figs. 2.9 and 2.10.

2.5.4 Segmented Cylinder Piles

Large diameter cylinder piles are often manufactured with centrifugal casting in segments 8 to 16 ft (2.4 to 4.9 m) in length. Longitudinal holes are formed during casting to receive post-tensioning strands or wires. Stressing 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. Grouting should follow the procedures outlined in the PCI Recommended Practice.[21]

2.5.5 Connections Between Pile and Pile Cap

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 strand diameters from the end.

In addition, strand development length must be considered. The ACI Code 318-89,[22] Section 12.9, and a paper by Shahawy and Issa[23] address strand development length.

A pile fixed to a cap must be adequately reinforced with dowels in this transition zone in order to resist bending moment.

Common methods of connection include the following:

1. Pile head extension: The pile head is extended into the cap generally 12 to 18 in. (305 to 457 mm) minimum. Embedded surfaces of the pile must be clean and preferably roughened before casting concrete.

2. Strand extension: Extend prestressing strands 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 (207 MPa)]. Embedded lengths depend on design requirements but should be 18 in. (457 mm) minimum. A paper by Salmons and McCrate[24] provides design data for bond of untensioned prestressing strand.

3. 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.[25] Holes can also be drilled into the pile head after driving to accommodate grouted dowels.

4. 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 plug, which is grouted into the core. Structural steel members can also be used as connectors in cylindrical piles.

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

2.5.6 Pile Tips

Normally, spiral reinforcement in prestressed piles is symmetrically placed so that the head and tip are equally reinforced. For conditions where exceptionally hard driving may occur, 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, or steel shoes, points, or stubs. Short structural steel stubs at the pile tip for aid in penetration into sloping rock are most common. For details of this tip, see Fig. 2.15. The ability of prestressed concrete piles to withstand very hard driving makes the use of special tips rare.

2.5.7 Cover

The minimum recommended cover for prestressed reinforcement is as follows:

Normal exposure................................ 2 in. (51 mm)
Marine or similar corrosive environment .... 2½ in. (64 mm)

For certain types of centrifugally cast, prestressed, post-tensioned piles, a cover of 1.5 in. (38 mm) has given satisfactory service for 20 years of marine exposure in the Gulf of Mexico. A 1.5 in. (38 mm) cover is recommended only if such piles are manufactured by a process using no-slump concrete containing a minimum of 658 lb of cement per cu yd (390 kg/m^3) of concrete.

CHAPTER 3 — MATERIALS

3.1 CONCRETE

3.1.1 Cement

Portland cement should conform to the "Standard Specification for Portland Cement" (ASTM C150). In areas where the soil or ground water contains sulfate concentrations in excess of 0.2 percent, C_3A content of the cement should be limited to 8 percent. ACI recommends a limit of 8 percent C_3A for sulfate concentrations between 0.1 and 0.2 percent and a limit of 5 percent for concentrations over 0.2 percent.

However, for the higher strength concretes [6000 psi (41.4 MPa) and over] employed in the manufacture of prestressed piles, the 8 percent limit on C_3A is considered adequate for sulfate concentrations over 0.2 percent. See ACI 543R-74(80). It is suggested that local prestressed concrete manufacturers be consulted for readily available cements as special cement requirements will affect cost and performance.

3.1.2 Aggregates

Concrete aggregates should conform to the "Standard Specification for Concrete Aggregates" (ASTM C33). Aggregates failing to meet this specification, but which have been shown by special test or actual service to produce concrete of adequate strength and durability, may be used with approval of the governing authority.

3.1.3 Water

Water used in mixing concrete should be clean and free from injurious amounts of oils, acids, alkalis, salts, organic materials, or other substances that may be deleterious to concrete or steel. Mortar cubes made with nonpotable mixing water should have 7- and 28-day strengths equal to at least 90 percent of the strengths of similar specimens made with potable water.

3.1.4 Admixtures

Air-entrained concrete is recommended for use in piles which will be subjected to cycles of freezing and thawing or wetting and drying. Air-entraining admixtures should be used where concrete piles are exposed to these conditions. Air-entraining admixtures should conform to the "Standard Specification for Air-Entraining Admixtures for Concrete" (ASTM C260).

The amount of air entrainment and its effectiveness depend on the admixture used, the size and type of the coarse aggregate, moisture content, 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 in the range of 4 to 7 percent by volume, depending on the size of the coarse aggregate.

Air-entraining admixtures are less effective when used in low slump, high strength concrete. Furthermore, the need for air entrainment is reduced in high strength concrete because of its high density and low porosity. For this reason, the site conditions should be weighed against the quality of the pile concrete before specifying air entrainment.

When used, water-reducing admixtures, retarding admixtures, accelerating admixtures, water-reducing and retarding admixtures, and water-reducing and accelerating admixtures should conform to the "Standard Specification for Chemical Admixtures for Concrete" (ASTM C494). Calcium chloride or admixtures containing calcium chloride should not be used.

3.1.5 Concrete Quality

Minimum concrete strength should be 3500 psi (24.1 MPa) for concentrically prestressed piles at the time of prestress transfer.

Concrete in precast prestressed piles and build-ups to be driven should have a minimum compressive cylinder strength, f'_c, of 5000 psi (34.5 MPa) at 28 days. Economy in handling and driving along with higher load capacity can be achieved with concrete strengths up to 8000 psi (55.2 MPa). Designers should check with local pile manufacturers to determine attainable strengths.

For the sake of durability, concrete piles should have at least 564 lbs of cement per cu yd of concrete (335 kg/m^3). The water-cement ratio (by weight) should correspond to the least water which will produce a plastic mix and provide the desired workability for the most effective placement of the concrete.

3.2 REINFORCEMENT

All steel wires, prestressing strand and reinforcement, unless otherwise stipulated, shall conform to the applicable ASTM standards (see Section 1.2).

3.3 GROUT

Cement grout, where used in prestressed piles, should be of materials which conform to the requirements stipulated herein for cement, sand, admixtures and water. Approved expanding admixtures or expansive cements may be used. Sand and cement grouts are generally used when grouting dowels into holes in heads of piles.

Some expanding admixtures contain calcium chloride and should not be used. Neat cement grout is frequently used to grout dowels in pile heads. Neat cement grouts used in bonding post-tensioned tendons should follow the PCI Recommended Practice.[21]

3.4 ANCHORAGES

Anchorage fittings for post-tensioning assemblies should conform to the latest requirements of the ACI 318 Building Code[22]. Also, post-tensioning specifications from the American Concrete Institute and Post-Tensioning Institute may be used for guidance.

CHAPTER 4 — MANUFACTURE AND TRANSPORTATION OF PRESTRESSED CONCRETE PILES

4.1 MANUFACTURING PLANTS

Manufacturers should be regularly engaged in the production of prestressed concrete piles or other prestressed concrete products for a period of not less than three years. Proven capability should be shown through certification in the PCI Plant Certification program.

4.2 HANDLING AND STORAGE

Damage to piles can occur during the handling, storage, and transporting stages. Handling should be done using the designed lifting points. On many long slender piles, three, four, or five pick-up points are required. This is done through the use of 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 piling 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.

Fig. 4.1. Expandable trailer for transporting piles up to 110 ft (33.5 m) long.

4.3 TRANSPORTING

Piles up to approximately 50 ft (15.2 m) long can be carried on flat bed trailers. Piles over this length are generally carried on expandable flat bed trailers or telescoping pole trailers. Fig. 4.1 shows an expandable trailer rigged to transport piles up to 110 ft (33.5 m) long.

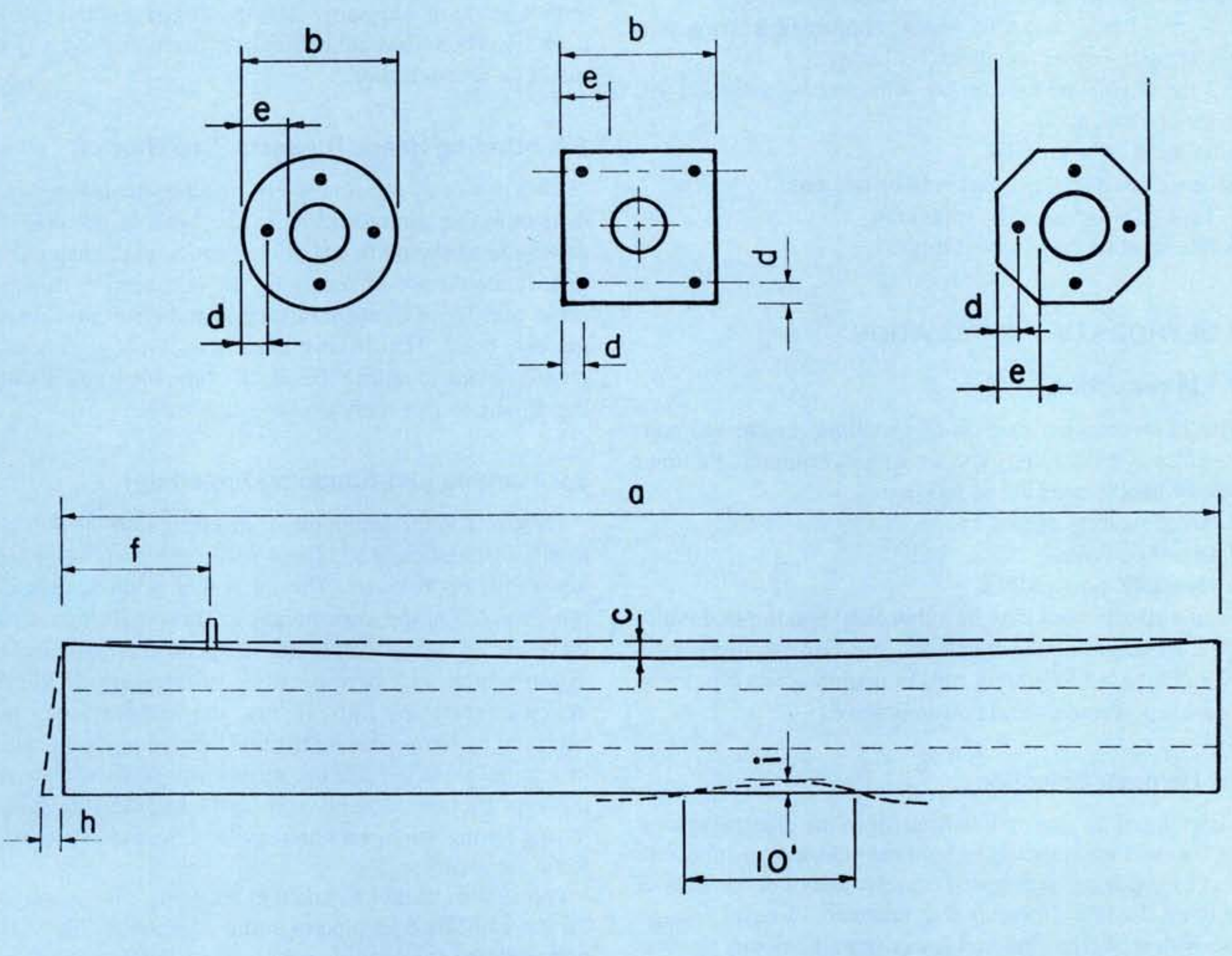
For piles requiring more than two support points, special supports should be articulated to avoid excessive bending stress in the pile. One method is to build "A" frames, one at the tractor end and one at the dolly end. On top of the "A" frames, a long steel support will provide the pile two or more 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 prior to delivery and all obstructions, ruts, holes or dangerous conditions corrected.

4.4 TOLERANCES

Piles should be fabricated in accordance with the generally accepted tolerances found in PCI MNL 116-85. These tolerances are reproduced in Fig. 4.2.

Closer tolerances are required when using mechanical splices, as recommended by the splice manufacturer.



a = Length* .... ± 1 in.

b = Width or diameter .... ± 3/8 in.

c = Sweep (variation from straight line parallel to centerline of member) (considered to be a form tolerance) .... ± 1/8 in. per 10 ft

d = Position of tendons .... ± 1/4 in.

e = Wall thickness .... − 1/4 in., +1/2 in.

f = Position of handling devices .... ± 6 in.

g = Position of steel driving tips .... ± 1/2 in.

h = Variation from specified end squareness or skew .... ± 1/4 in. per 12 in., ± 1/2 in. max.

i = Local smoothness any surface .... ± 1/4 in. per 10 ft

j = Longitudinal spacing of spiral reinforcement .... ± 3/4 in.

* In most cases, controlling pile length to + 6 in., − 2 in. is functionally acceptable.
Note: 1 ft = 0.305 in.; 1 in. = 25.4 mm.

Fig. 4.2. Prestressed concrete pile production tolerances.

CHAPTER 5 — INSTALLATION OF PRESTRESSED CONCRETE PILES

5.1 SCOPE

5.1.1 Structural Integrity

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 pile should be installed so as to ensure the structural integrity of the pile itself, and ensure that it will properly resist the imposed design loads.

5.1.2 Factors Affecting Installation

Installation methods may vary with such factors as:

1. Type of pile: bearing, sheet, combined bearing and sheet, tension, fender, trestle and columns

2. Type of soil and intervening substance between soil and structure (water, air)

3. Vertical or batter piles

4. Design purpose, i.e., forces to be resisted

5. Type of structure to be supported

6. Site location and accessibility

5.2 METHODS OF INSTALLATION

5.2.1 Introduction

The most common method of installing prestressed concrete piles is by driving with an impact hammer. Hammer types commonly used are as follows:

1. Air or steam powered

2. Diesel powered

3. 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.

5.2.2 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 and/or depth without damaging the pile. In comparing hammers of equal energy, those with a heavier ram and lower impact velocity 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 in evaluating the effectiveness of the pile installation system selected.

Driving stresses caused by hammer impact must be maintained below levels which 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. The following maximum driving stresses are recommended as a guide to minimize the risk of pile damage:

Allowable compression: $0.85\, f_c'$ – effective prestress
Allowable tension: $6\sqrt{f_c'}$ + effective prestress

5.2.2.1 Dynamic Pile Testing

Dynamic pile testing consists of monitoring production and/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 the pile load capacity. These data are particularly useful for capacity estimates on restruck piles.

5.2.3 Driving Heads (Helmets, Cap Blocks)

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 also holds or retains the cushion block to reduce the shock of the blow and distribute the driving force evenly over the pile head.

5.2.4 Jetting (Jet-Spudding Prejetting)

Prejetting is the technique of inserting into the soil, prior to 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 liquified condition which will permit easier penetration of the pile. When penetrating hard layers, the same effect can be achieved by jetting during the pile driving using an external or internal jet at the pile tip. Jetting will temporarily reduce the skin friction in sands and sandy material. Precautions during jetting are as recommended in Section 5.4 (see also Refs. 5 and 6).

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 pipe with corresponding damage to piles.

5.2.5 Predrilling

Predrilling a starter hole may be appropriate to locate and/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 in order 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 geotechnical and structural engineers.

5.2.6 Spudding

Spudding is the technique of inserting into the soil, prior to 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 when the difficult material may tend to deflect the pile when driven.

5.2.7 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 prior to pile installation.

5.3 PREVENTION OF DAMAGE

5.3.1 Types of Damage

In some cases, prestressed concrete piles have cracked and/or spalled during driving. The damage or failure of such concrete piles occurring during driving can be classified into three types:

1. Spalling of concrete at the head of the pile due to high compressive stress

2. Spalling of concrete at the point of the pile due to hard driving resistance at the point

3. Transverse cracking or breaking of the pile due to torsion and/or reflected tensile stress sometimes accompanied by spalling at the crack

5.3.1.1 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:

1. 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.

2. 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. (See Section 4.4 on head tolerances.)

3. Improper fit (generally too tight) of the driving head on top of the pile.

4. 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.

5. Lack of adequate spiral reinforcing steel at the pile head or pile point 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. (See Section 2.5.1 for spiral steel requirements.)

6. If the top edges and corners of the concrete pile are not adequately chamfered, they are likely to spall on impact of the ram.

5.3.1.2 Compression Damage at Pile Tip

Spalling of concrete at the point of a pile can be caused by extremely hard driving resistance at the point. Such resistance may be encountered when founding the pile point 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, overdriving 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.

5.3.1.3 Transverse Cracking

Transverse cracking of a pile due to reflected tensile stress can lead to failure of the pile during driving.

To quote Gerwick,[26] "Horizontal cracks appear about one-third of the length down from the head and are spaced about 1.6 ft (0.5 m) apart. Puffs of concrete dust are produced, followed by spalling and widening of the crack. Repeated driving produces fatigue in the concrete adjacent to the crack including a definite rise in temperature, and eventually brittle fracture of the tendons. The actual point or points of cracking tend to occur initially at a point of discontinuity, such as a lifting point, insert, form mark or honeycomb."

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 per second (3660 to 4570 m/second). 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.

Since, in a given pile, the stress wave 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 point. 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 point, 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 in order to prevent damaging the pile.

If the soil resistance at the point 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:

1. 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 percent 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. See Section 5.4 for cushion material recommendations.

2. 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.

3. When little or no soil resistance at the point of long piles [50 ft (15.2 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.

4. When very hard driving resistance is encountered at the point of piles [50 ft (15.2 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. A comprehensive study of dynamic driving stresses is included in Ref. 27.

5.3.2 Bursting of Hollow Prestressed Piles

Longitudinal splits due to internal bursting pressure may occur with open-ended hollow prestressed piles. When driving in extremely soft, semifluid soils, the fluid pressure builds up and a hydraulic ram effect known as "water hammer" occurs. The precautions outlined in Section 2.5 should be followed.

When driving open-ended precast 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 spirals or ties.

Use of a solid or steel armored tip will eliminate the splitting problems mentioned, but may not be compatible with other installation requirements.

5.4 REPAIR OF DAMAGED PILES

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

5.4.1 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 the closely spaced spiral at the head of the pile has been cut off.

5.4.2 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.

5.5 GOOD DRIVING PRACTICES

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

1. Use the proper hammer as discussed in Section 5.2.2.

2. Use adequate cushioning material between the driving head and the concrete pile. Additional discussion relative to pile cushioning is provided in Section 5.5.1.

3. 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.

4. 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.

5. If predrilling or jetting is permitted in placing the piles, ensure that the pile point is well seated with moderate soil resistance at the point before full driving energy is used.

6. 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.

7. Ensure that the driving head fits loosely around the pile top so that the pile may rotate easily within the driving head.

8. Ensure that bearing piles are straight and not cambered. High flexural stresses may result during driving of an initially bent pile. (See Section 4.4 — Tolerances.)

9. 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 corners of the pile head.

10. Use adequate spiral reinforcement throughout the pile, particularly near the head and tip.

11. 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 (4.8 to 8.3 MPa) after losses. Very short piles have been installed with lower prestress levels [350 to 400 psi (2.4 to 2.8 MPa)]. Where moment resistance in service is a requirement, effective prestress levels up to $0.2 f_c'$ and even higher have been used without difficulty.

5.5.1 Pile Cushioning

A wood cushioning material of 3 or 4 in. (76 or 102 mm) may be adequate for short piles [50 ft (15.2 m) or less] with moderate point resistances. A wood cushioning material of 6 in., 8 in., 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 burns, 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.

Fig. 5.1. Pile location established with the use of templates.

5.6 HANDLING AND TRANSPORTATION

Prestressed concrete piles should be picked up, handled and transported to avoid tensile cracking and any impact damage. Piles cracked due to mishandling cannot be relied upon for resisting driving tensile stresses which may develop. Refer to Section 2.2.1 for handling stress criteria.

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

5.7 POSITIONING AND ALIGNMENT

Correct position can best be assured by accurate setting of the pile. Removal of surface obstructions will aid in attainment of 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.

Fig. 5.1 shows piling being driven in a template.

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 vertical 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 the 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, current, 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.

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Design-Construction of Precast Segmental Elevated Metro Line for Monterrey, Nuevo León, México

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Paul E. Mondorf obtained his M.Sc. engineering degree from the Technical University of Denmark, Copenhagen, where he was also assistant professor for 4 years. Over the major part of his career, Mr. Mondorf has been active in the design and construction of concrete structures around the world, such as concrete platforms for the North Sea, containment vessels for nuclear plants in France and Spain, and a large variety of bridges, including cast-in-place and precast segmental cantilever bridges. Recently, he was field construction engineer for the third Lake Washington Bridge, Washington, resident engineer for the CSX Escambia Bay Crossing, Florida and advisor to the general contractor who built the Metro in Monterrey, Mexico. Currently, he is resident engineer for the construction of the Los Angeles Metro Green Line segmental bridges.

In April 1991, the first line of a mass transit rail system (Metro) for Monterrey, Nuevo León, México, was brought into service. The 18.7 km (11.6 mile) line is an elevated bridge structure stretching over existing streets, consisting of segmental precast concrete spans that vary in length up to a maximum of 47 m (154 ft). There are 17 elevated stations on the line, combining precast and cast-in-place concrete in the column-beam-slab structures and in the adjacent platforms. More than 6500 precast concrete bridge segments and 2700 other elements were cast in a specially designed, state-of-the-art plant located north of the city. Segments were match-cast on concrete beds long enough for each complete span, then delivered by trucks to the site, erected span-by-span on movable steel trusses and post-tensioned. The precast concrete box girder segments have pretensioned top slabs, 7.40 m (24 ft 3 in.) wide, that support two parallel, standard-gauge tracks. The electrified metro trains have up to four cars and operate at a design speed of 70 km/hr (43 miles/hr). The line was built by a consortium of three Monterrey contractors in just under 40 months from start of design to its opening.

Monterrey is a rapidly growing industrial state capital of northern Mexico, 240 km (150 miles) south of the United States-Mexican border. Its present population is estimated at 3½ million inhabitants. The city is fairly spread out among the foothills of the Sierra-Madre Mountains, and most residential areas are one-story family dwellings or generally not more than four- to five-story multi-family houses.

The city has a good system of radial and belt roads for rapid vehicular traffic. However, the larger part of the population is totally dependent on public transportation, consisting mostly of buses which increase air and noise pollution and inner city congestion.

The construction of a mass transit