CHAPTER 16

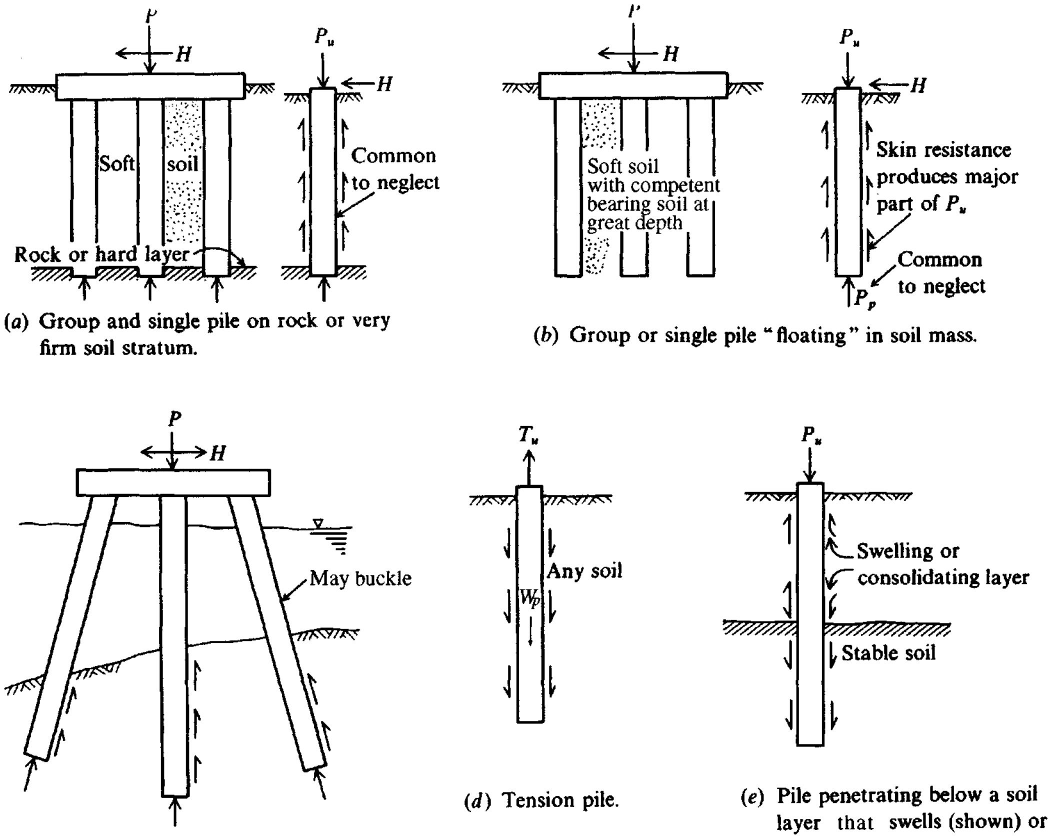
SINGLE PILES—STATIC CAPACITY AND LATERAL LOADS; PILE/POLE BUCKLING

16-1 INTRODUCTION

Piles are structural members of timber, concrete, and/or steel that are used to transmit surface loads to lower levels in the soil mass. This transfer may be by vertical distribution of the load along the pile shaft or a direct application of load to a lower stratum through the pile point. A vertical distribution of the load is made using a *friction* (or *floating*) pile and a direct load application is made by a point, or *end-bearing*, pile. This distinction is purely one of convenience since all piles carry load as a combination of side resistance and point bearing except when the pile penetrates an extremely soft soil to a solid base.

Piles are commonly used (refer to Fig. 16-1) for the following purposes:

1. To carry the superstructure loads into or through a soil stratum. Both vertical and lateral loads may be involved.
2. To resist uplift, or overturning, forces, such as for basement mats below the water table or to support tower legs subjected to overturning from lateral loads such as wind.
3. To compact loose, cohesionless deposits through a combination of pile volume displacement and driving vibrations. These piles may be later pulled.
4. To control settlements when spread footings or a mat is on a marginal soil or is underlain by a highly compressible stratum.
5. To stiffen the soil beneath machine foundations to control both amplitudes of vibration and the natural frequency of the system.
6. As an additional safety factor beneath bridge abutments and/or piers, particularly if scour is a potential problem.
7. In offshore construction to transmit loads above the water surface through the water and into the underlying soil. This case is one in which partially embedded piling is subjected to vertical (and buckling) as well as lateral loads.



(*a*) Group and single pile on rock or very firm soil stratum.

(*b*) Group or single pile "floating" in soil mass.

(*c*) Offshore pile group.

(*d*) Tension pile.

(*e*) Pile penetrating below a soil layer that swells (shown) or consolidates.

Figure 16-1 Typical pile configurations. Note that, whereas analysis is often for a single pile, there are usually three or more in a group. Typical assumptions for analysis are shown. Lateral load H may not be present in (*a*) or (*b*).

Piles are sometimes used to control earth movements (for example, landslides). The reader should note that power poles and many outdoor sign poles may be considered as partially embedded piles subject to lateral loads. Vertical loads may not be significant, although buckling failure may require investigation for very tall members.

A pile foundation is much more expensive than spread footings and likely to be more expensive than a mat. In any case great care should be exercised in determining the soil properties at the site for the depth of possible interest so that one can as accurately as possible determine whether a pile foundation is needed and, if so, that neither an excessive number nor lengths are specified. A cost analysis should be made to determine whether a mat or piles, in particular the type (steel, concrete, etc.), are more economical. In those cases where piles are used to control settlement at marginal soil sites, care should be taken to utilize both the existing ground and the piles in parallel so that a minimum number are required.

Piles are inserted into the soil via a number of methods:

1. Driving with a steady succession of blows on the top of the pile using a pile hammer. This produces both considerable noise and local vibrations, which may be disallowed by local codes or environmental agencies and, of course, may damage adjacent property.

2. Driving using a vibratory device attached to the top of the pile. This method is usually relatively quiet, and driving vibrations may not be excessive. The method is more applicable in deposits with little cohesion.
3. Jacking the pile. This technique is more applicable for short stiff members.
4. Drilling a hole and either inserting a pile into it or, more commonly, filling the cavity with concrete, which produces a pile upon hardening. A number of methods exist for this technique, and the reader is referred to Table 16-1 and Fig. 16-7 for typical installations.

When a pile foundation is decided upon, it is necessary to compute the required pile cross section and length based on the load from the superstructure, allowable stress in the pile material (usually a code value), and the in situ soil properties. These requirements allow the foundation contractor to order the necessary number and lengths of piles. Dynamic formulas, pile-load tests, or a combination are used on-site to determine if the piles are adequately designed and placed. It is generally accepted that a load test is the most reliable means of determining the actual pile capacity.

Pile capacity determinations are very difficult. A large number of different equations are used, and seldom will any two give the same computed capacity. Organizations that have been using a particular equation tend to stick with it—particularly if a successful data base has been established. It is for this reason that a number of what are believed to be the most widely used (or currently accepted) equations are included in this text. In a design situation one might compute the pile capacity by several equations using the required empirical factors suitably adjusted (or estimated) and observe the computed capacity. From a number of these computations some "feel" for the probable capacity will develop so that a design recommendation/proposal can be made.

Note that, although all the pile capacity equations are for a single pile, rarely is a single pile used; rather two or three (or more) piles are used in a group. Further note that the soil properties used in the design are those from the initial soil exploration program, and the soil properties that exist when the foundation is in service may be very different depending on how the piles have been installed and the number of piles in the group.

This chapter will be concerned with the methods of static pile capacity determination as well as an introduction to materials and methods to produce pile members. Methods to analyze lateral pile response to loads and to pile buckling will also be presented. Chapter 17 will take up the problem of estimating pile capacity based on the field driving resistance (dynamic capacity) and pile hammer energy.

16-2 TIMBER PILES

Timber piles are made of tree trunks with the branches carefully trimmed off, usually treated with a preservative, and driven with the small end as a point. Occasionally the large end is driven for special purposes as in very soft soil where the soil will flow back against the shaft and with the butt resting on a firm stratum for increased bearing. The tip may be provided with a metal driving shoe when the pile is to penetrate hard or gravelly soils; otherwise it may be cut either square or with some point.

Generally there are limitations on the size of the tip and butt end as well as on the misalignment that can be tolerated. The Chicago Building Code (in Chap. 13-132–190) requires that the tip have a minimum diameter of 150 mm and the butt 250 mm if the pile is under

TABLE 16-1
Typical pile characteristics and uses

Pile type	Timber	Steel	Cast-in-place concrete piles (shells driven without mandrel)	Cast-in-place concrete piles (shells withdrawn)
Maximum length	35 m	Practically unlimited	10–25 m	36 m
Optimum length	9–20 m	12–50 m	9–25 m	8–12 m
Applicable material specifications	ASTM-D25 for piles; P1-54 for quality of creosote; C1-60 for creosote treatment (Standards of American Wood Preservers Assoc.)	ASTM-A36, A252, A283, A572, A588 for structural sections ASTM-A1 for rail sections	ACI	ACI†
Recommended maximum stresses	Measured at midpoint of length: 4–6 MPa for cedar, western hemlock, Norway pine, spruce, and depending on Code. 5–8 MPa for southern pine, Douglas fir, oak, cypress, hickory	$f_s = 0.35\text{–}0.5 f_y$	$0.33 f_c'$; $0.4 f_c'$ if shell gauge $\leq$ 14; shell stress $= 0.35 f_y$ if thickness of shell $\geq$ 3 mm $f_c' \geq 18$ MPa	$0.25\text{–}0.33 f_c'$
Maximum load for usual conditions	450 kN	Maximum allowable stress × cross section	900 kN	1300 kN
Optimum load range	80–240 kN	350–1050 kN	450–700 kN	350–900 kN
Disadvantages	Difficult to splice Vulnerable to damage in hard driving Vulnerable to decay unless treated Difficult to pull and replace when broken during driving	Vulnerable to corrosion HP section may be damaged or deflected by major obstructions	Hard to splice after concreting Considerable displacement	Concrete should be placed in dry More than average dependence on quality of workmanship

TABLE 16-1 (*continued*)

Pile type	Timber	Steel	Cast-in-place concrete piles (shells driven without mandrel)	Cast-in-place concrete piles (shells withdrawn)
Advantages	Comparatively low initial cost Permanently submerged piles are resistant to decay Easy to handle	Easy to splice High capacity Small displacement Able to penetrate through light obstructions	Can be redriven Shell not easily damaged	Initial economy
Remarks	Best suited for friction pile in granular material	Best suited for end bearing on rock Reduce allowable capacity for corrosive locations or provide corrosion protection	Best suited for friction piles of medium length	Allowable load on pedestal pile is controlled by bearing capacity of stratum immediately below pile

Typical illustrations

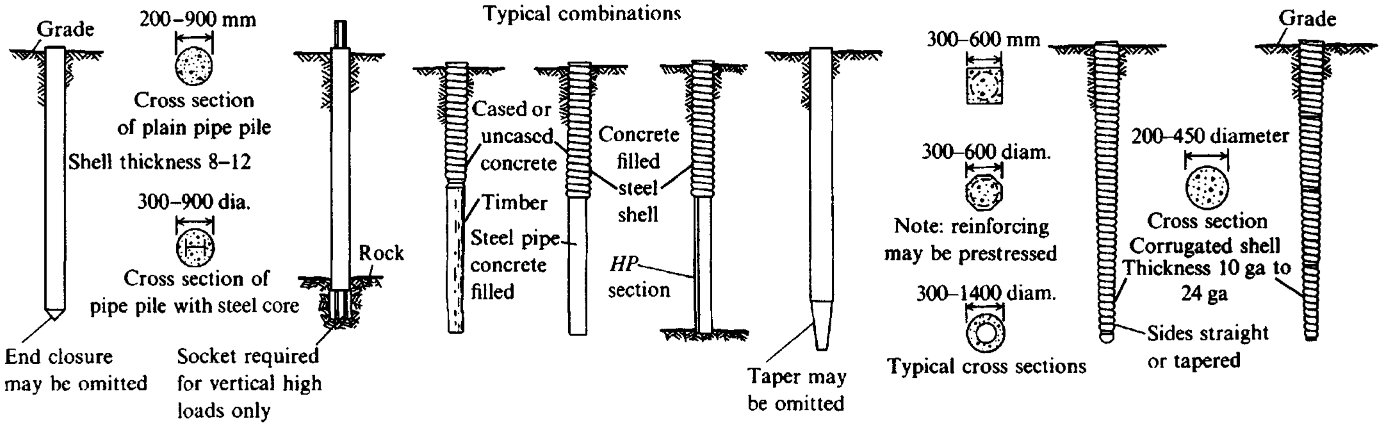


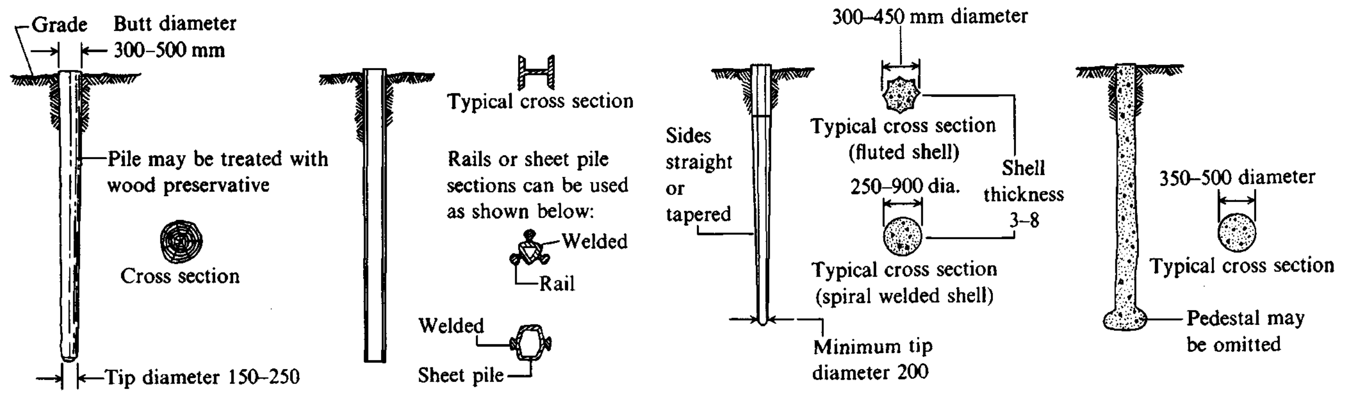
TABLE 16-1 *(continued)*

Pile type	Concrete-filled steel pipe piles	Composite piles	Precast concrete (including prestressed)	Cast in place (thin shell driven with mandrel)	Auger-placed pressure-injected concrete (grout) piles
Maximum length	Practically unlimited	55 m	10–15 m for precast 20–30 m for prestressed	6–35 m for straight sections 12 m for tapered sections	5–25 m
Optimum length	12–36 m	18–36 m	10–12 m for precast 18–25 m for prestressed	12–18 m for straight 5–12 m for tapered	10–18 m
Applicable material specifications	ASTM A36 for core ASTM A252, A283 for pipe ACI Code 318 for concrete	ACI Code 318 for concrete ASTM A36 for structural section ASTM A252 for steel pipe ASTM D25 for timber	ASTM A15 reinforcing steel ASTM A82 cold-drawn wire ACI Code 318 for concrete $f_c' \geq 28$ MPa precast $f_c' \geq 35$ MPa prestressed	ACI	See ACI
Recommended maximum stresses	$0.40 f_y$ reinforcement < 205 MPa $0.35–0.50 f_y$ for shell < 175 MPa $0.33 f_c'$ for concrete	Same as concrete in other piles Same as steel in other piles Same as timber piles for composite	$0.33 f_c'$ unless local building code is less $0.4 f_y$ for reinforced unless prestressed	$0.33 f_c'$; $f_s = 0.4 f_y$ if shell gauge ≤ 14 use $f_y = 0.35 f_y$ if shell thickness ≥ 3 mm	$0.25 f_c'$
Maximum load for usual conditions	1800 kN without cores 18 000 kN for large sections with steel cores	1800 kN	8500 kN for prestressed 900 kN for precast	675 kN	700 kN
Optimum load range	700–110 kN without cores 4500–14 000 kN with cores	250–725 kN	350–3500 kN	250–550 kN	350–900 kN
Disadvantages	High initial cost Displacement for closed-end pipe	Difficult to attain good joint between two materials	Difficult to handle unless prestressed High initial cost Considerable displacement Prestressed difficult to splice	Difficult to splice after concreting Redriving not recommended Thin shell vulnerable during driving Considerable displacement	Dependence on workmanship Not suitable in compressible soil

TABLE 16-1 *(continued)*

Pile type	Concrete-filled steel pipe piles	Composite piles	Precast concrete (including prestressed)	Cast in place (thin shell driven with mandrel)	Auger-placed pressure-injected concrete (grout) piles
Advantages	Best control during installation No displacement for open-end installation Open-end pipe best against obstruction High load capacities Easy to splice	Considerable length can be provided at comparatively low cost	High load capacities Corrosion resistance can be attained Hard driving possible	Initial economy Tapered sections provide higher bearing resistance in granular stratum	Freedom from noise and vibration Economy High skin friction No splicing
Remarks	Provides high bending resistance where unsupported length is loaded laterally	The weakest of any material used shall govern allowable stresses and capacity	Cylinder piles in particular are suited for bending resistance	Best suited for medium-load friction piles in granular materials	Patented method

Typical illustrations



*Additional comments in *Practical Guidelines for the Selection, Design and Installation of Piles* by ASCE Committee on Deep Foundations, ASCE, 1984, 105 pages.

†ACI Committee 543, "Recommendations for Design, Manufacture, and Installation of Concrete Piles," *JACI,* August 1973, October 1974; also in ACI MCP 4 (reaffirmed 1980).

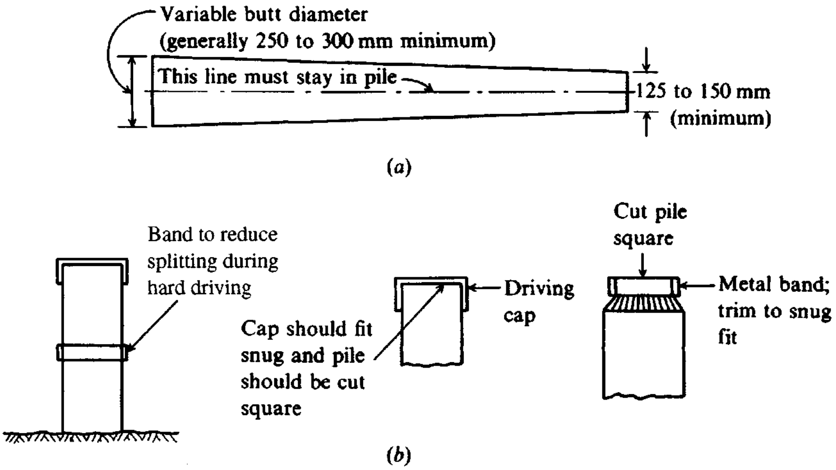


Figure 16-2 (*a*) Alignment criteria for timber piles; (*b*) devices to protect pile during driving operations.

7.6 m and have a 300-mm butt if the pile is more than 7.6 m long. The alignment requirement is that a straight line from the center of the butt to the center of the tip lie within the pile shaft (Fig. 16-2*a*).

ASCE Manual 17 [reprinted ASCE (1959) but now out of print] categorizes timber piles as follows:

Class A: To be used for heavy loads and/or large unsupported lengths. The minimum butt diameter is 360 mm.

Class B: For medium loads. Minimum butt diameter is 300 mm.

Class C: Use below the permanent water table or for temporary works. Minimum butt diameter is 300 mm. Bark may be left on this pile class.

The ASCE manual (and building codes) stipulate minimum quality of the timber concerning defects, knots, holes, and type of wood.

If a timber pile is below the permanent water table, it apparently will last indefinitely. When a timber pile is subjected to alternate wetting and drying, the useful life will be short, perhaps as little as one year, unless treated with a wood preservative. Partly embedded piles and piles above the water table are susceptible to damage from wood borers and other insects unless treated.

The driving end of a timber pile is usually damaged by fiber crushing (called brooming) from the hammer energy. This damage can be somewhat controlled by using a driving cap or metal band around the butt as illustrated in Fig. 16-2. After having been driven to the necessary penetration, the broomed end is cut square and any exposed scars, as well as the fresh end cut, should be coated with a generous application of preservative. A pile may become broken where the soil is very hard or contains boulders. Where a sudden increase in penetration occurs and a soft soil stratum is not expected, a broken pile shaft should be suspected.

Splices in timber piles are undesirable but may be effected as shown in Fig. 16-3. The splice in Fig. 16-3*b* can transmit tension. In both illustrations care should be exercised to get a maximum joint bearing area.

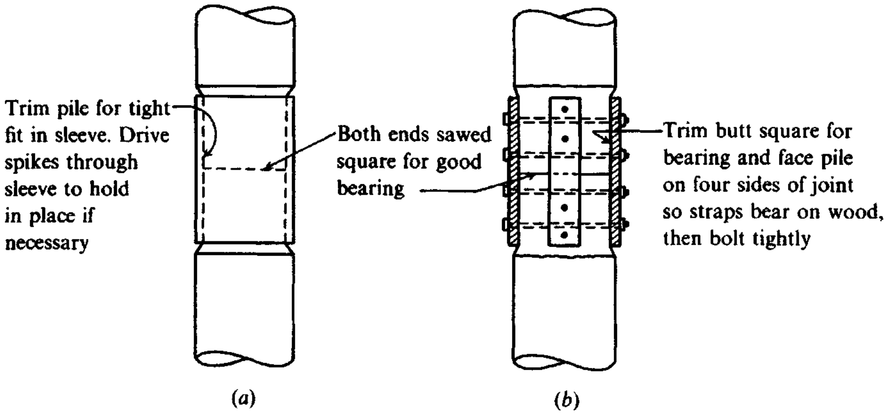


Figure 16-3 Splices in timber piles: (*a*) Using a metal sleeve with ends carefully trimmed for fit and bearing; (*b*) using splice plates. Be sure all exposed cuts are painted or sprayed with preservative.

The allowable design load based on pile material is

$$P_a = A_p f_a \tag{16-1}$$

where A_p = average pile cross-sectional area at the pile cap
f_a = allowable design stress (code) value for the type of timber

The static capacity based on the soil surrounding the pile is computed as for other pile materials and will be taken up in Sec. 16-7 and following. The principal additional factor to consider is that the coefficient of friction between wood and soil may approach tan ϕ' from a combination of soil displacement from the wood volume and from penetration of the wood by the soil grains— particularly in cohesionless soils.

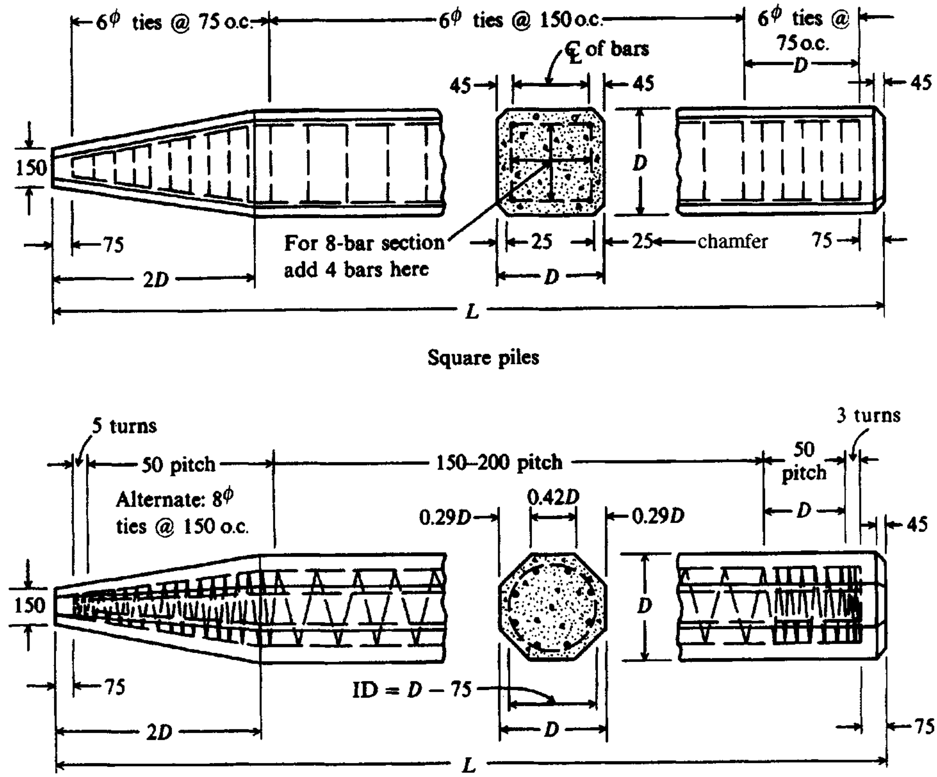
Further information on timber piles may be obtained from American Wood Preservers Institute (AWPI) publications (1966, 1967, 1969, 1981) and ASTM D 25 (Vol. 4.09).

16-3 CONCRETE PILES

Table 16-1 indicates that concrete piles may be precast, prestressed, cast in place, or of composite construction.

Precast Concrete Piles

Piles in this category are formed in a central casting yard to the specified length, cured, and then shipped to the construction site. If space is available and a sufficient quantity of piles needed, a casting yard may be provided at the site to reduce transportation costs. *Precast* piles may be made using ordinary reinforcement as in Fig. 16-4 or they may be prestressed as in Fig. 16-5. Precast piles using ordinary reinforcement are designed to resist bending stresses during pickup and transport to the site and bending moments from lateral loads and to provide sufficient resistance to vertical loads and any tension forces developed during driving. The design procedures can be found in any text on reinforced-concrete design. However,



Spiral wire

D, mm	400	500	600
US bar	#5	#4	#3
SI bar	15	10	10	mm

Octagonal piles

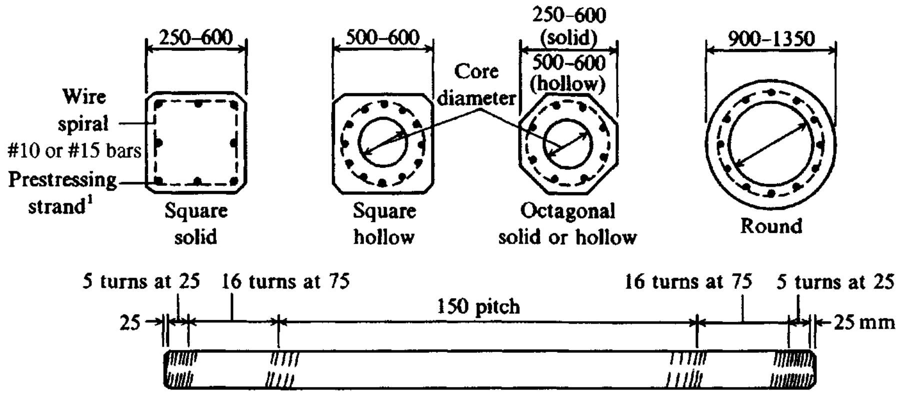
6ϕ = 6 mm diameter (not standard diameter SI bar)

Figure 16-4 Typical details of precast piles. Note all dimensions in millimeters. [*After PCA (1951).*]

temporary stresses from handling and driving (tensile) may be used that are on the order of 50 percent larger than the allowable concrete design stresses. The minimum pile reinforcement should be 1 percent.

Figure 16-6 illustrates typical bending moments developed during pickup depending on the location of the pickup point. The pickup point should be clearly marked since the bending moments depend heavily on its location.

Prestressed piles are formed by tensioning high-strength steel (f_{ult} of 1700 to 1860 MPa) prestress cables to a value on the order of 0.5 to $0.7f_{\text{ult}}$, and casting the concrete pile about the cable. When the concrete hardens, the prestress cables are cut, with the tension force in the cables now producing a compressive stress in the concrete pile as the steel attempts to return to its unstretched length. The pile shortens under the prestress compression load P_i, and additionally the concrete undergoes creep, while simultaneously there is some relaxation in the steel, so the end result is an overall reduction of prestress force (and stress) that cannot be precisely evaluated. One may attempt a refined analysis of this loss, but about the same result is obtained by lumping the losses into a value of 240 MPa (i.e., $\sigma_{\text{pf}} = P_i/A - 240$). The pile will shorten some additional amount under the working load(s) to reduce the above



[1] Strand: 9.5–12.7 mm ($\frac{3}{8}$ to $\frac{1}{2}$ in.) nominal diam., f_u = 1860 MPa

Figure 16-5 Typical prestressed concrete piles (see also App. A, Table A-5); dimensions in millimeters.

σ_{pf} further to produce a final compressive stress σ_f in the pile. These losses in the absence of refined calculations may be taken as 240 MPa not including axial-shortening loss caused by the applied design loads. Final compressive concrete stresses from prestressing are usually on the order of 4 to 6 MPa. It is common to use higher-strength concrete (35 to 55 MPa) in prestressed piles because of the large initial compressive stresses from prestressing. A modest trade-off is obtained from the lighter-weight pile produced for the same load capacity.

The allowable design load P_a based on pile material for prestressed piles, and including prestress loss due to load and creep, can be computed as

$$P_a = A_g(0.33 f_c' - 0.27 f_{\text{pe}}) \tag{16-2}$$

where A_g = gross (total) concrete area

f_{pe} = effective prestress after all losses (about 5 MPa is usual)

Pickup points should be placed so that the computed bending stress has $f = M/S \leq f_{\text{pe}}$, where M is from Fig. 16-6. If this is done the pile should not develop tension cracks during handling. Prestressing the pile tends to counteract any tension stresses during either handling or driving. This latter is particularly important since a pile is often placed in a hostile environment. If tension stresses during driving are large enough transient tension cracks are produced. During the time the crack is open foreign matter can enter and produce deterioration of the steel, which may not be detected for a long period of time.

Concrete piles are considered permanent; however, certain soils (usually organic) contain materials that may form acids that can damage the concrete. Saltwater may also adversely react with the concrete unless special precautions are taken when the mix proportions are designed. Additionally, concrete piles used for marine structures may undergo abrasion from wave action and floating debris in the water. Alternate freezing and thawing can cause concrete damage in any exposed situation.

Nonprestressed concrete used in marine structures should meet the following criteria:

1. Use nonreactive aggregates.
2. Use $8\frac{1}{2}$ to 10 sacks of cement per cubic meter of concrete.

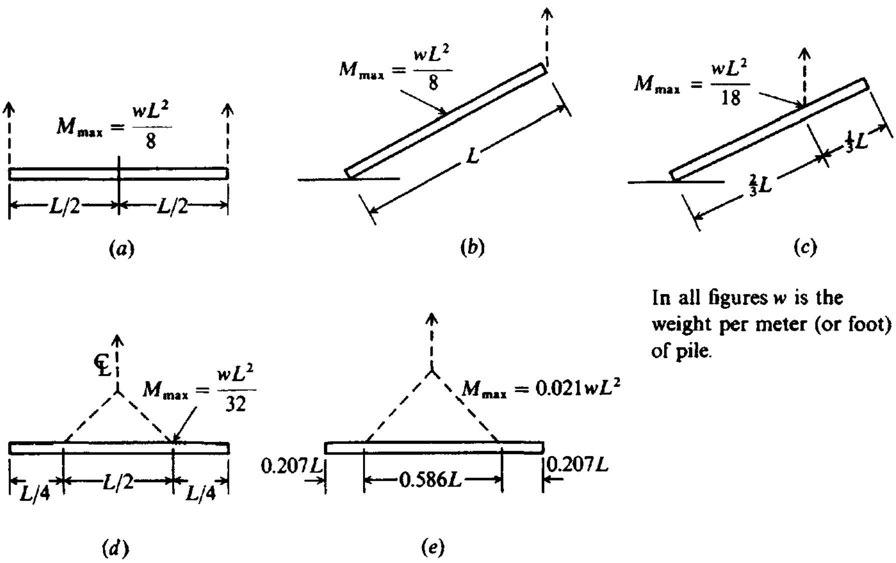


Figure 16-6 Location of pickup points for precast piles, with the indicated resulting bending moments.

3. Use type V cement (has high sulfate resistance).
4. Use a water/cement ratio ≤ 0.53 (by weight).
5. Use air-entrained concrete in temperate and cold regions.
6. Use a minimum of 75 mm of clear cover on all steel reinforcement (normal clear cover is 50 to 70 mm).

Cast-in-Place Piles

A cast-in-place pile is formed by drilling a hole in the ground and filling it with concrete. The hole may be drilled (as in caissons), or formed by driving a shell or casing into the ground. The casing may be driven using a mandrel, after which withdrawal of the mandrel empties the casing. The casing may also be driven with a driving tip on the point, providing a shell that is ready for filling with concrete immediately, or the casing may be driven open-end, the soil entrapped in the casing being jetted[1] out after the driving is completed.

Various methods with slightly different end results are available and patented. Figure 16-7 indicates some of the commonly available patented cast-in-place piles, and is intended to be representative only. Note that they are basically of three types: (1) shell or cased, (2) shell-less (uncased), or (3) pedestal types.

[1]Jetting is a common construction procedure of using a high-velocity stream of water to erode (or wash) a volume of soil into a soil-water suspension. The suspension is pumped or somehow disposed of so that an open cavity is formed. Soil cavities can be jetted into nearly all soils, including those that are very dense and hard.

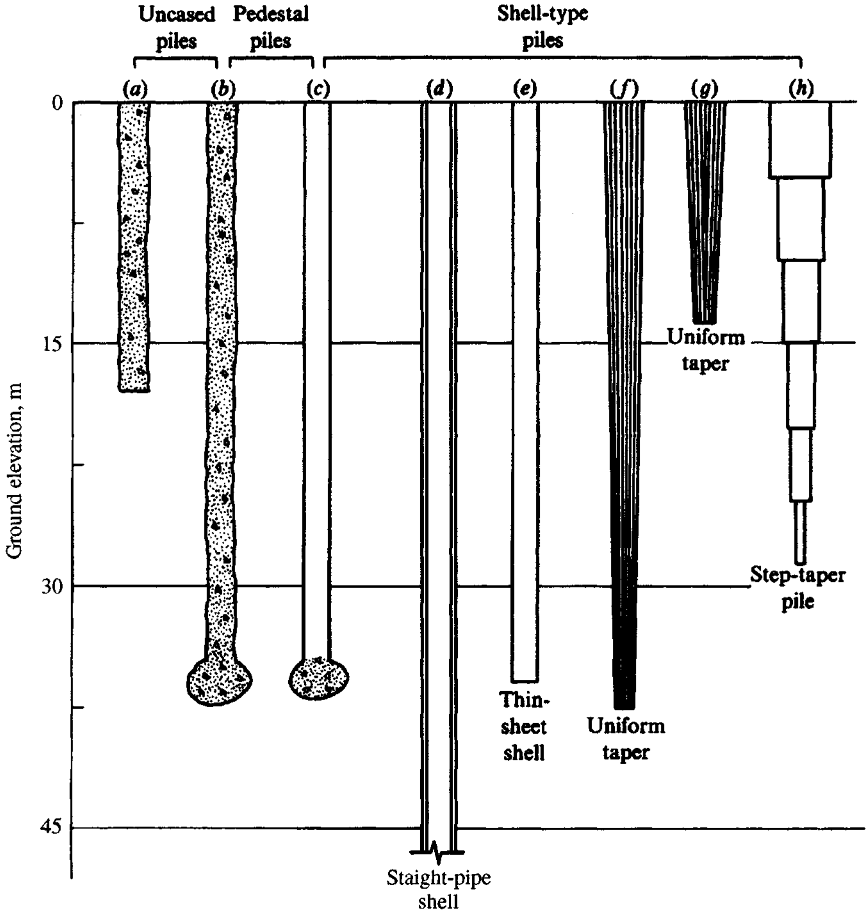


Figure 16-7 Some common types of cast-in-place (patented) piles: (*a*) Commonly used uncased pile; (*b*) Franki uncased pedestal pile; (*c*) Franki cased pedestal pile; (*d*) welded or seamless pipe; (*e*) Western cased pile; (*f*) Union or Monotube pile; (*g*) Raymond standard; (*h*) Raymond step-taper pile. Depths shown indicate usual ranges for the various piles. Current literature from the various foundation equipment companies should be consulted for design data.

The allowable design load for all concrete piles (not prestressed) is

$$P_a = A_c f_c + A_s f_s \tag{16-3}$$

where A_c, A_s = area of concrete and steel shell, respectively
f_c, f_s = allowable material stresses

Note that Eq. (16-3) does not apply for the aboveground portion of partially embedded piles. A reduction factor may be applied (to either f_c or P_a) for accidental eccentricities. Slenderness effects (l/r ratio) for that portion of the shaft length surrounded by soil are not necessary but may be required for the exposed length above ground.

A pile similar in section to that shown in Fig. 16-7*a* can be formed by using a hollow-stem continuous-flight auger with a diameter of 250 to 400 mm. The hole is excavated to the desired elevation, a hose is connected to the auger, and cement grout (a pumpable mix of water, cement, and sand or sand and small gravel) is pumped under pressure down the auger stem and out the tip into the cavity formed as the auger is slowly withdrawn. The soil on the auger flights prevents the cement mixture from coming up the shaft and allows a modest amount of pump pressure to be exerted to reduce voids and make a solid pile-to-soil contact along the shaft.

A record should be kept of the auger depth and quantity of material pumped to ensure that the hole is filled with grout and that the auger was not withdrawn too rapidly that soil caved into the void such as to produce a discontinuous pile shaft. When the shaft has been filled, the wet concrete, having a greater density than the surrounding soil, will maintain the shaft until the concrete sets.

Reinforcement in the upper part of the shaft can be readily provided by inserting the proper number of reinforcing bars (or dowels) into the wet concrete. Where several soil layers are penetrated, the grout pressure may expand the borehole sufficiently to distort the pile shaft slightly in the soft strata; however, the principal effect of this is to increase the quantity of grout required to fill the shaft.

The Franki pile of Fig. 16-7*b* and *c* is produced by first placing very dry (zero slump) concrete in a cased shaft cavity and ramming it out of the casing base to produce an adequate-sized base enlargement. The shaft cavity is then filled with concrete to complete the pile. The casing may be pulled as the concrete is placed or left if pulling it would be difficult. Both the Franki system (which is patented) and piles formed from the continuous-flight auger method are very economical where cast-in-place procedures can be used.

16-4 STEEL PILES

These members are usually rolled **HP** shapes or pipe piles. Wide-flange beams or **I** beams may also be used; however, the **H** shape is especially proportioned to withstand the hard driving stress to which the pile may be subjected. In the **HP** pile the flanges and web are of equal thickness; the standard **W** and **I** shapes usually have a thinner web than flange. Table A-1 in App. A lists the **HP** pile sections produced in the United States and Canada. Pipe piles are either welded or seamless steel pipes, which may be driven either open-end or closed-end. Closed-end pipe piles are usually filled with concrete after driving. Open-end piles may be filled, but this is often not necessary, because there will be a dense soil plug at some depth below the top (and visible). Here it may only be necessary to jet out some of the upper soil plug to the necessary depth for any reinforcing bars required for bending (and to pump out the water used for jetting), before filling the remainder of the pile cavity with concrete. Concrete in only this shaft depth may be necessary for dowel bars.

The **HP** pile is a small-volume displacement pile since the cross-sectional area is not very large. A plug tends to form between the flanges at greater depths, however, so the bottom several meters may remold the soil on the order of the volume of the plug. An open-end pipe is also considered a small-volume displacement pile; however, a plug also forms inside with a depth one or more meters below the outside ground level—probably from a combination of inside perimeter friction and driving vibrations. From the depth at which the "plug" stabilizes (not visible during driving because of the pile cap and hammer interference) to the final driving depth, the lower soil may be remolded based on the volume of the plug and not the actual area of the pipe section.

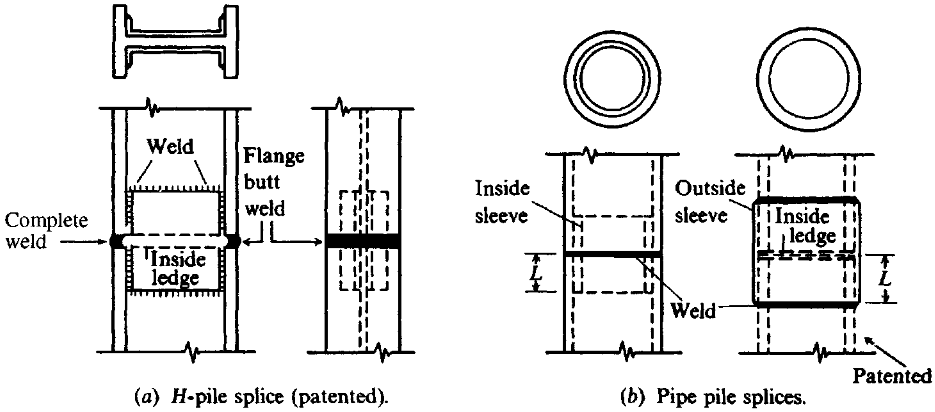


Figure 16-8 Splices for **H** and pipe piles.

HP piles have an advantage of sufficient rigidity that they will either break smaller boulders or displace them to one side. Open-end pipe piles have the advantage of surface entry to break up boulders encountered by either use of a chopping bit or drilling, blasting, and removal of the rock fragments. When large boulders are encountered one should consider the possibility of terminating the pile on (or slightly into) them.

Splices in steel piles (see Fig. 16-8) are made in the same manner as in steel columns, i.e., by welding (most common) or by bolting. Except for small projects involving only a few piles, most splices are made with prefabricated (and patented) splice connectors. For **HP** piles, splices can be prefabricated from two channels of adequate length back-to-back, with a short spacer on which the top pile section rests. The splice is then welded to the web across the ends, and the pile flanges are butt-welded to complete the splice. Pipe pile splicers consist of a ledged ring with an ID slightly larger than the pipe OD. The two sections of pipe to be joined rest against the inside ledge and an end weld is made around the pipe at both ends of the splicer. Generally these splices will develop the strength of the pile in compression, tension, bending, and shear to satisfy most building code requirements.

When a pile must be spliced to develop adequate embedment length, all the necessary equipment should be standing by so that when the hammer is shut off the splice can be quickly made. If this is not done—and sometimes if it is done—the soil tends to set or "freeze" about the pile, and resumption of driving is difficult and sometimes requires changing to a larger hammer. These larger driving stresses may cause considerable damage to the upper part of the pile. This phenomenon is independent of pile material (such as timber, concrete, or steel).

If the top of the steel pile is adequately embedded in the cap (say 150 mm or more) special load transfer plates are not necessary [Ohio (1947)]. Where embedment is limited or for special purposes, steel plates can be welded on the top of the pile to assist in load transfer and ensure that the piles and pile cap act as a unit.

In reference to Fig. 16-9*c* and Fig. 16-10*d*, there is little difference in driving resistance whether a pipe pile has a flat or conical driving point (or shoe). The reason is that a wedge-shaped zone of soil develops in front of the flat point somewhat like zone *abc* of Fig. 4-3*b* beneath a spread footing. It also appears that the later driving resistance of an open-end pipe is about that of a closed-end pile since the plug of soil inside the pipe shell (with friction developed with the wall) behaves similarly to the driving plate.

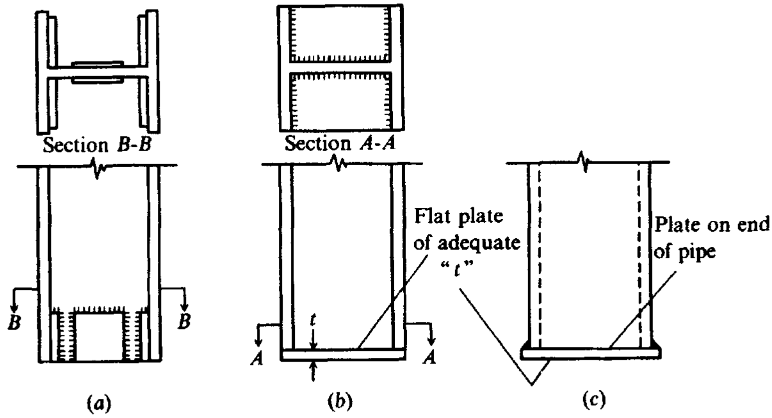


Figure 16-9 Shop- or field-fabricated driving points. Labor costs make this process generally uneconomical except for small numbers of points. Note that (*c*) will damage the perimeter soil so that skin resistance is reduced in stiff clays.

HP piles and pipe piles may require point reinforcement to penetrate hard soils or soils containing boulders without excessive tip damage. Figure 16-9 illustrates field-/shop-fabricated points, and Fig. 16-10 illustrates several that are commercially available. Those commercially available are likely to be more economical due to associated labor and fabrication costs except for isolated cases where only one or two might be needed.

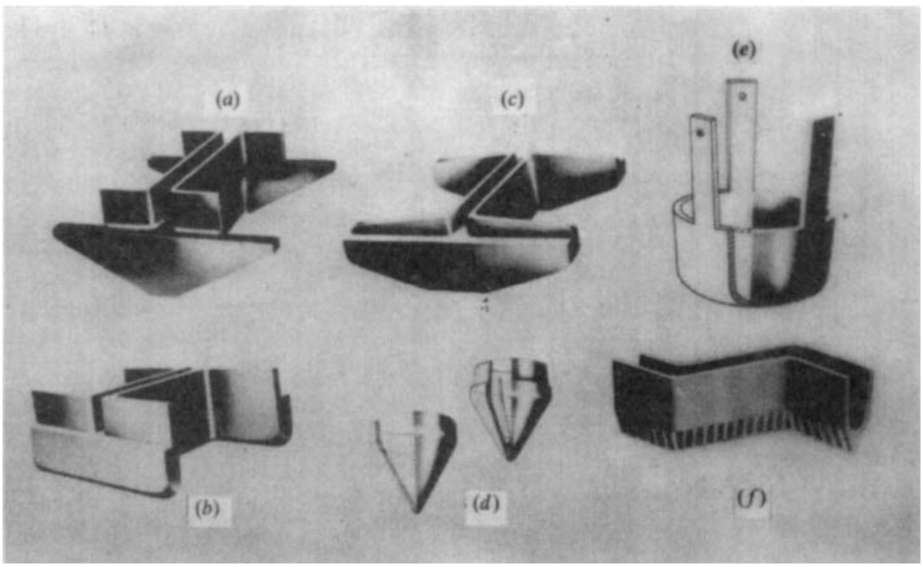


Figure 16-10 Commercially available points for several types of piles. Points are also available in higher-strength steel for very hard driving. Commercial points should be used if a large number of piles are to be driven. Parts (*a*), (*b*), and (*c*) are points for **HP** piles; (*d*) pipe-pile point; (*e*) timber-pile point; (*f*) sheet-pile point. (*Courtesy of Associated Pile and Fitting Corp.*)

The allowable design load for a steel pile is

$$P_a = A_p f_s \tag{16-4}$$

where A_p = cross-sectional area of pile at cap
f_s = allowable steel stress (code or specification); in range of 0.33 to $0.5f_y$

16-5 CORROSION OF STEEL PILES

A corrosion study for the National Bureau of Standards [NBS (1962)] on both sheet-pile and bearing-pile substructures indicated that if piles are driven in *undisturbed natural* soil deposits, pile corrosion is not great enough to affect the strength of the piles significantly. This study encompassed soils with pH (a pH less than 7 is acidic) values from 2.3 to 8.6, and electric resistivities of 300 to 50 200 ohm · cm, from which it was further concluded that as long as the soil was undisturbed, the soil characteristics and properties are not significant. The substructures studied had been in service from 7 to 40 years. The soil resistance probe described by Roy and Ramaswamy (1983) may be used to obtain the soil resistance (ohm·cm) for estimating the probability of pile corrosion in the given site soil.

This study also indicated that piles driven in disturbed, or fill, soils will tend to undergo relatively more corrosion and may require painting (i.e., paint the pile, then construct the backfill). This observation was attributed to a higher oxygen concentration in the disturbed soil. Undisturbed soils were found to be oxygen-deficient from a few feet below the ground surface.

Piles exposed to seawater or to effluents with a pH much above 9.5 or below 4.0 will require painting or encasement in concrete to resist corrosion [Watkins (1969)]. This statement also applies to piles in general for the several feet in the zone where the water line fluctuates. As an alternative to painting or concrete encasement, a splice that uses a slightly larger section in the corrosive zone may be made.

Some of the newer grades of high-strength and copper-alloy steels are said to have substantial corrosion resistance. The A690 high-strength low-alloy steel has approximately two to three times more corrosion resistance to seawater than ordinary carbon steel of A36 grade. High-strength steel **HP** piles are seldom needed since the geotechnical considerations (bearing capacity of the rock or soil resistance) are more likely to set the structural stresses required of the steel than the structural considerations as set by codes, which may allow $f_a = 0.33$ to $0.5F_y$. For example, if the maximum allowable rock pressure for a pile founded on rock is 70 MPa, that sets a limit in the **HP** pile of 70 MPa regardless of F_y.

16-6 SOIL PROPERTIES FOR STATIC PILE CAPACITY

For *static pile* (and group) *capacity analysis* the angle of internal friction ϕ and the cohesion c of the soil are needed. Immediate controversy arises since some designers use undrained (or total) stress parameters, whereas others—particularly more recently—use effective stress values.

For *wave equation analysis* a value for the elastic recovery from deformation (quake, Q) and damping constants are needed.

Lateral pile analyses require use of the lateral modulus of subgrade reaction k_s or a lateral stress-strain modulus E_s. The context of usage determines whether the lateral or horizontal value is of interest for these latter two parameters.

The soil parameters may be determined from laboratory triaxial tests on "undisturbed" samples. These are quite satisfactory for piles installed in predrilled holes but may be considerably in error for driven piles.

Laboratory triaxial test parameters are not very reliable for driven piles since the soil in the vicinity of the pile undergoes extensive remolding, a change in water content, and usually an increase in density (or particle packing). Since these changes are highly indeterminate there is no way to duplicate them in any current laboratory test with any confidence. Thus, if laboratory tests are used, they are on the original in situ "undisturbed" samples, with experience used to extrapolate these data to obtain the design parameters. For these reasons the SPT is widely used, although there is movement to more use of CPT or PMT (the vane test is not much used) to obtain in situ parameters.

Most pile design in cohesionless materials (sands, gravelly sands, silty sands, etc.) is based on SPT N values. Pile design in cohesive deposits is usually based on unconfined compression strength q_u tests (pocket penetrometer, compression tests, the laboratory vane, hand-held pocket-sized shear strength test device called a torvane), primarily on very disturbed samples from the SPT. The CPT is, however, being used more in cohesive deposits (and in fine sands and fine silty sands) since those experienced with the procedure believe better design data are obtained.

The SPT N values should be adjusted to a standard energy—either N_{70} or N_{55} depending on the available data base and using the procedures outlined in Secs. 3-7 through 3-9.

Piles driven into the soil mass always result in remolding of the soil in the immediate vicinity of the pile (say, three to five pile diameters). At this instant, undrained soil-strength parameters are produced, which may approach remolded drained values if the degree of saturation S is low and/or the coefficient of permeability k is relatively large.

In general, however, a considerable time lapse (several months to years) occurs before the full design loads are applied. In this interval the excess pore pressures dissipate, and drained (or consolidated-undrained if below the GWT) conditions exist. For these it appears remolded (or residual) soil parameters best describe the soil behavior.

The capacity of piles in soft clays increases with time, with most strength regain occurring in from 1 to 3 months [Flaate and Selnes (1977), Orrje and Broms (1967)], **HP** piles requiring longer times. This increase in capacity is somewhat explained from the pile volume displacement producing high pore pressures that cause a more rapid drainage and consolidation of the soil very near the pile.

There is some opinion that the displaced volume of pipe and similar piles produces so much lateral compression in cohesive soils that a zone of perhaps 50 to 200 mm tends to consolidate to such a high value that the effective pile diameter is increased 5 to 7 percent over the actual value. This increase in "effective" diameter produces a corresponding increase in pile load capacity.

The reduced water content resulting from consolidation in this zone has been observed for some time [see references cited in Flaate (1972)]. The increase in "effective" pile diameter is likely to be marginal (or nonexistent) in very stiff and/or overconsolidated clays. In fact the volume displacement in these clays may produce a reduction in capacity over time as soil creep reduces the lateral pressure produced by the initial volume displacement.

Tavenas and Audy (1972) report an increase in load capacity with time for piles in sand, with the principal regain occurring in about the first month. This strength increase cannot be attributed to dissipation of excess pore pressures but may be due to *aging* from chemical contaminants (primarily carbonates) causing inter-grain and grain-to-pile adhesion. There may be some gain in capacity from dissipation of residual driving stresses; however, this is doubtful since modern methods of driving produce a viscous semi-fluid state in a zone of 6 to 8 mm (at least) around the pile.

The pile capacity in calcareous sands may be considerably less after installation than the design value indicated by conventional design. This material (particularly if the carbonate content is greater than 50 percent) deteriorates rapidly under stress in the presence of water. Since the carbonate content is a byproduct of biological deposition (shells and such), deterioration is more likely to occur along shorelines and coral islands. Unfortunately except for performing tests (ASTM D 4373) for carbonate content (in percent) there is not much that can be done to quantify a design. Murff (1987, with a number of references) noted that some designers simply limit the skin resistance f_s (see Sec. 16.7) to some value on the order 15 to 30 kPa and point bearing q_o in the range of 4000 to 6000 kPa with smaller design values as the percent carbonates increases.

The pile literature contains a great number of conflicting conclusions obtained from both correct and incorrect interpretations of measured load test results and naturally occurring soil anomalies. As a consequence statistical correlations are particularly useful, but only on reliable data. Much of the pile literature (particularly early publications) did not provide enough data so that the reader could arrive at any kind of conclusion. Including these early data in a statistical correlation is not recommended although most publishers of correlations feel the more cases cited the better (or the more confidence the reader will have in the results).

Where piles are placed in predrilled holes, the soil state remains at nearly the existing (drained or consolidated undrained) condition. Possible deterioration of the cohesion at the interface of the wet concrete and soil may occur but this will be partially offset by the small increase in pile diameter when grains in the surrounding soil become part of the pile shaft as the cement hydrates.

The loss of K_o from soil expansion into the cavity may be partially offset by the lateral pressure developed from the wet concrete, which has a higher density than the soil.

Summarizing, for pile design we do not have a very good means to obtain soil parameters except for predrilled piles. For all cases of driven piles we have to estimate the soil parameters. In most cases if there is reasonable correlation between the design and measured load (from a load test) it is a happy coincidence.

16-7 STATIC PILE CAPACITY

All static pile capacities can be computed by the following equations:

$$\left.\begin{aligned} P_u &= P_{\text{pu}} + \sum P_{si} \\ &= P_P + \sum P_{si,u} \end{aligned}\right\} \quad \text{(compression)} \qquad (16\text{-}5a)$$

$$T_u = \sum P_{si,u} + W_P \quad \text{(tension)} \qquad (16\text{-}5b)$$

where P_u = ultimate (maximum) pile capacity in compression—usually defined as that load producing a large penetration rate in a load test

T_u = ultimate pullout capacity

P_{pu} = ultimate pile tip capacity—seldom occurs simultaneously with ultimate skin resistance capacity $\sum P_{si,u}$; neglect for "floating" piles (which depends only on skin resistance)

P_p = tip capacity that develops simultaneously with $\sum P_{si,u}$; neglect for "floating" piles

$\sum P_{si}$ = skin resistance developing with ultimate tip resistance P_{pu}; neglect for point bearing piles

$\sum P_{si,u}$ = ultimate skin resistance developing simultaneously with some tip capacity P_p

W = weight of pile being pulled

$\sum$ = summation process over i soil layers making up the soil profile over length of pile shaft embedment

The allowable pile capacity P_a or T_a is obtained from applying a suitable SF on the contributing parts as

$$P_a = \frac{P_{pi}}{\text{SF}_p} + \frac{\sum P_{si}}{\text{SF}_s} \qquad (a)$$

or using a single value SF (most common practice) to obtain

$$P_a = \frac{P_u}{\text{SF}} \qquad \text{or} \qquad T_a = \frac{T_u}{\text{SF}} \qquad (b)$$

This value of P_a or T_a should be compatible with the capacity based on the pile material (timber, concrete, or steel) considered earlier; and SF_i represents the safety factors, which commonly range from 2.0 to 4 or more, depending on designer uncertainties.

Opinion is mixed whether SF_i should be based on both load-carrying mechanisms [Eq. (*a*)] or be a single value [Eq. (*b*)]. In general, safety factors for piles are larger than for spread foundations because of more uncertainties in pile-soil interaction and because of the greater expense of pile foundations.

Although Eqs. (16-5) are certainly not highly complex in form, using them to arrive at a prediction of capacity that closely compares with a load test is often a fortunate event. A lack of correspondence is attributable to the difficulties in determining the in situ soil properties, which (as previously stated) change in the vicinity of the pile after it is has been installed. Additionally, the soil variability, both laterally and vertically, coupled with a complex pile-soil interaction, creates a formidable problem for successful analysis.

We can readily see from Eq. (16-5*a*) that the ultimate pile capacity P_u is not the sum of the ultimate skin resistance plus the ultimate point resistance but is the sum of one and a portion of the other.

Ultimate skin resistance is produced at some small value of relative slip between pile and soil, where slip is defined at any point along the pile shaft as the accumulated differences in shaft strain from axial load and the soil strain caused by the load transferred to it via skin resistance. This slip progresses down the pile shaft with increasing load.

In the upper regions the slip reaches limiting skin resistance and load is transferred to lower regions, which reach limiting skin resistance, ..., etc., and finally to the tip, which begins to carry load. If pile penetration is rapid at this time the *ultimate* load P_u is reached. If penetration is not rapid the point load increases with further penetration until it also reaches *ultimate,* but with further penetration the slip resistance reduces to some limiting value that is less than the ultimate. We are now at the maximum pile capacity P_u.

The essential difference for tension capacity is that there is no point load, so that the force necessary to initiate a constant withdrawal rate is some limiting skin resistance, plus the pile weight W_p, plus suction at and near the point in wet soils. Suction, however, is seldom considered in design since it is transient. Again the upper pile elements reach the limiting skin resistance first.

Although it is common to compute the skin resistance contribution as an "average" value over one or two depth increments, better correlation is obtained if the summation is made for each stratum penetrated, using the best estimate of the applicable soil parameters for that stratum. The normal increase in soil density with depth will always produce several "soil layers" having values of γ, ϕ, and c that are somewhat different from those obtained using a single layer even for the same soil. It has been popular (and also convenient but not recommended) to use an average value from the several layers making up the site soil profile. A computer program (such as PILCAPAC) makes it a trivial exercise to subdivide the soil penetrated by the pile shaft into several layers for an improved analysis.

A study of load-settlement and load-transfer curves from a number of load tests indicates that slip to develop maximum skin resistance is on the order of 5 to 10 mm [Whitaker and Cooke (1966), Coyle and Reese (1966), AISI (1975)] and is relatively independent of shaft diameter and embedment length, but may depend upon the soil parameters ϕ and c. Note that sufficient slip at any point along the shaft to mobilize the limiting shear resistance is not the same as the butt movement measured in a pile-load test (as illustrated in Fig. 17-6) but is larger than the slip that produces the maximum (or ultimate) skin resistance.

Mobilization of the ultimate point resistance in any soil requires a point displacement on the order of 10 percent of the tip diameter B (see Fig. 16-11*a* for point cross section) for driven piles and up to 30 percent of the base diameter for bored piles and caissons. This is a total point displacement and when the pile point is in material other than rock may include additional point displacement caused by skin resistance stresses transferred through the soil to produce settlement of the soil beneath the point (refer to qualitative stress trajectories on right side of Fig. 16-11*a*). It is highly probable that in the usual range of working loads, skin resistance is the principal load-carrying mechanism in all but the softest of soils.

Since the pile unloads to the surrounding soil via skin resistance, the pile load will decrease from the top to the point. The elastic shortening (and relative slip) will be larger in the upper shaft length from the larger axial load being carried. Examination of a large number of load-transfer curves reported in the literature shows that the load transfer is approximately parabolic and decreases with depth for cohesive soils as shown in Fig. 16-12*a*.

The load transfer may, however, be nearly linear for cohesionless soils, and the shape may be somewhat dependent on embedment depth in all materials. Generally a short pile will display a more nearly linear load-transfer curve than a long pile; however, this conclusion is somewhat speculative since not many very long piles have been instrumented because of both expense and the poor survivability of instrumentation with increased driving effort. The more nonlinear load-transfer curves for long piles may be caused from overburden pressure increasing the soil stiffness with depth. The load-transfer curves for either short end-bearing or long friction piles may be nearly linear and vertical at the butt end where the relative slip and driving whip, or lateral shaft movement under hammer impacts (critical in stiff clays) are so large that the upper soil carries very little load. Figure 16-13 illustrates a case where the upper region also carries very little load at higher pile loads; however, this is in sand fill so that the small load is due more to relative slip than to driving whip damage.

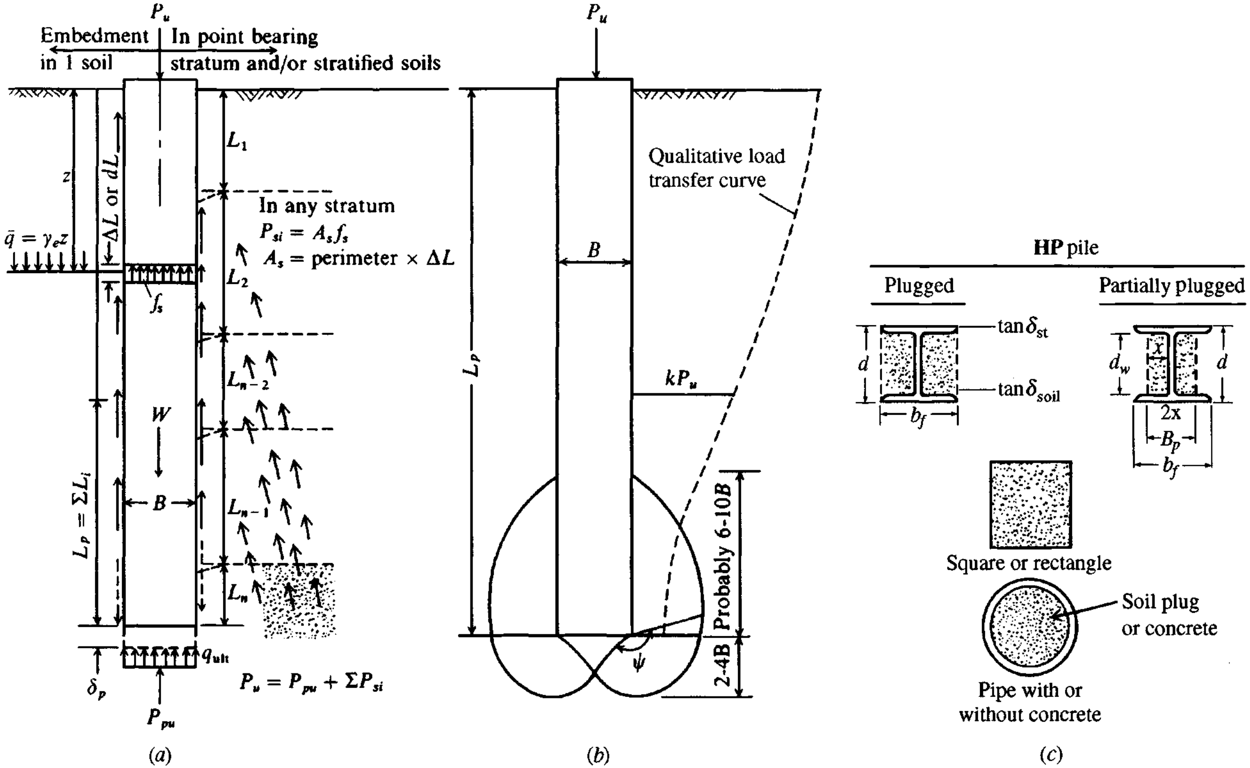


Figure 16-11 Piles in soil. Pile-to-soil friction $\tan\delta$ defined for pile perimeters shown; **HP** pile has two values; all others have a single δ value.

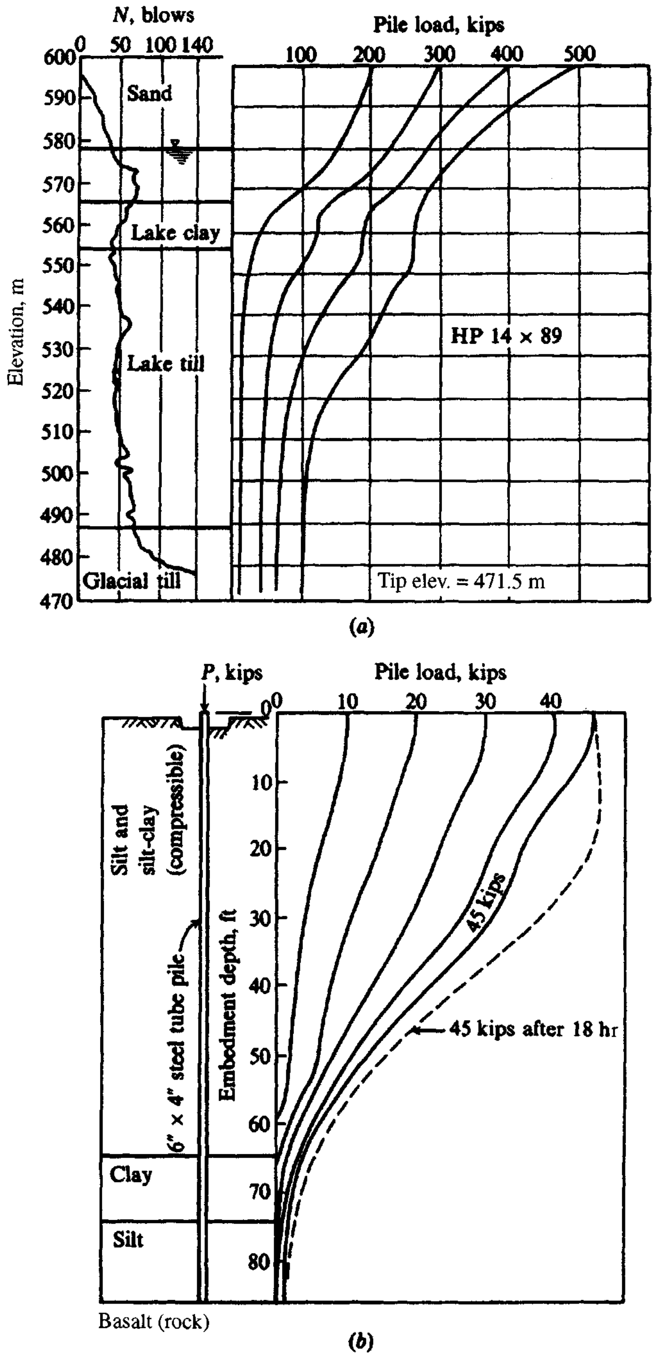


Figure 16-12 (*a*) Load-transfer curves for an **HP** pile in cohesive soil. [*From D'Appolonia and Romualdi (1963).*] (*b*) Load-transfer curves for pile in compressible soil showing transfer to be time-dependent. [*Frances et al. (1961).*]

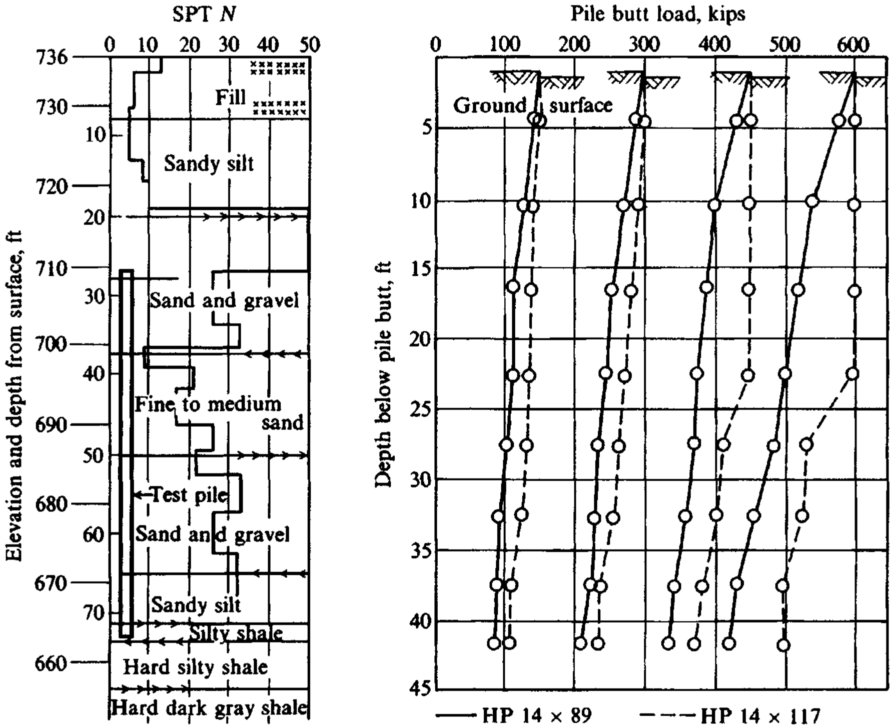


Figure 16-13 Load transfer for long **HP** piles in sand. Note that the behavior of **H** 14 × 117 is considerably different from that of the **H** 14 × 89 at higher loads. [*After D'Appolonia (1968).*]

The amount of load that is carried by the point under any butt loading depends on the surrounding soil, length and stiffness (AE/L) of the pile, and the actual load. Load duration and elapsed time before load application may also be significant factors to increase (or decrease) the point load for end-bearing piles that penetrate soft soils as shown in Fig. 16-12*b*.

Inspection of Figs. 16-12 and 16-13 indicates some interpretation is required to estimate load transfer at any depth increment. The piles of Fig. 16-13 are in the same site, but at higher loads there is little similarity in the shapes of the load-transfer curves.

When a pile is driven into a soil the response will depend upon several factors:

1. The volume of soil displaced by the pile. Concrete, closed-end pipe, and timber piles displace a large volume of soil relative to open-end pipe and **HP** piles.

   A plug forms on the inside of an open-end driven pipe pile and acts as a part of the pile cross section (including an apparent weight increase) when the friction resistance on the metal perimeter becomes larger than the weight of the plug [see Paikowsky and Whitman (1990)]. The plug is visible at some depth below the ground line in driven pipe piles. This depth represents a volume change due to driving vibrations and compression from friction between the inside pipe perimeter and the soil plug [see Eq. (17-8)].

   Two plugs (or partial plugs) usually form between the flanges of **HP** piles depending on the amount of soil-to-steel friction/adhesion along the inner faces of the two flanges and web and on the amount of soil-to-soil friction/adhesion across the web depth. In clay the "plugged" case of Fig. 16-11*c* will generally form unless the pile is quite short. In sands the full plug may not form [see Coyle and Ungaro (1991)]. You can estimate the amount of plug formation (refer to Fig. 16-11*c*) as follows:

$$(2x_p + d_w)\gamma z K \tan\delta = d_w \gamma z K \tan\phi$$

Canceling $\gamma z K$, using $d_w = d$, and solving for x_p, we obtain

$$x_p = \frac{d}{2}\left(\frac{\tan\phi}{\tan\delta} - 1\right) < \frac{b_f}{2} \qquad \text{(partial plug forms)}$$

$$\geq \frac{b_f}{2} \qquad \text{(full plug forms)}$$

$$\text{Then } A_{\text{point}} = d \times 2x_p \qquad \text{(neglecting web thickness)}$$

$$\text{Perimeter } A_s = 2b_f + 2d \qquad \text{(neglect any inner flange width zones)}$$

The preceding roundings should give adequate computational accuracy since the x_p zone is likely curved inward from the inner flange tips and not the assumed straight line shown. You can refine the foregoing for actual web thickness if desired, but both angles ϕ and δ are usually estimated. You probably should make this check even for point bearing piles in dense sand.

Use these "plug" dimensions to compute the plug weight to add to any pile computations that include a pile weight term W_p.

2. The amount and type of overburden material. Piles penetrating a cohesionless soil into clay will tend to drag sand grains into the cohesive soil to a depth of about 20 pile diameters [Tomlinson (1971)]. This material will be trapped in the void around the perimeter caused by driving whip and tends to increase the skin resistance.
3. The fact that piles penetrating a soft clay layer into a stiffer lower layer will drag (or flow) a film of the softer material into the perimeter void to a depth of about 20 pile diameters. This dragdown may not be serious, however, for the crack closure will consolidate this material so that the resulting adhesion will be much higher than the adhesion in the upper soft layer.
4. The fact that large-volume piles penetrating a stiff clay layer tend to form large surface cracks that radiate out from the pile such that adhesion in the topmost 20 pile diameters is most uncertain. Generally the top 1.2 to 1.8 m of penetration should be neglected in computing the skin resistance in medium stiff or stiff clays and in sand.
5. The fact that soft clay tends to flow and fill any cracks that form during driving. After driving and dissipation of the excess pore pressures, the skin resistance tends to be larger than the initial values. It is believed that considerable consolidation occurs, which produces the higher skin resistances. This is the rationale that the adhesion factor α [of such as Eq. (16-11)] can be larger than 1 when s_u is under about 50 kPa.

16-8 ULTIMATE STATIC PILE POINT CAPACITY

The ultimate static pile point capacity in any soil can be computed using the bearing-capacity equations given in Table 4-1. The N_γ term is often neglected when the pile base width B_p is not large. It may not be neglected where an enlarged pile base or the *piers* of Chap. 19 are used. The computed point bearing capacity varies widely because there is little agreement on what numerical values to use for the bearing-capacity factors N_i.

We will look at several of the more popular values, but no special recommendation is given for the "best" values since local practice or individual designer preference usually governs the values selected/used.

As previously stated, the soil parameters may be derived from laboratory tests on "undisturbed" samples but more often are unconfined compression data from an SPT or cone penetration test data. In general, the point capacity is computed as

$$P_{pu} = A_p(cN_c'd_cs_c + \eta\bar{q}N_q'd_qs_q + \tfrac{1}{2}\gamma' B_pN_\gamma s_\gamma) \tag{16-6}$$

where A_p = area of pile point effective in bearing, i.e., generally include any "plug." Use actual steel area for point bearing **HP** piles founded on rock, giving simply

$$P_{pu} = A_{steel} \times q_{ult}; \text{ see Sec. 4-16 for rock } q_{ult}$$

c = cohesion of soil beneath pile point (or s_u)

B_p = width of pile point (including "plug")—usually used only when point is enlarged

N_c' = bearing capacity factor for cohesion as previously defined in Chap. 4 but not computed the same way. Use

$$d_c = 1 + 0.4\tan^{-1}(L/B)$$

And when $\phi = 0$; $c = s_u$; $N_c' \approx \mathbf{9.0}$

N_q' = bearing capacity factor (may include overburden effects)

$$\text{Use } d_q = 1 + 2\tan\phi(1 - \sin\phi)^2\tan^{-1} L/B$$

The following depth factors are representative:

L/B	d_c	d_q; $\phi = 36°$
10	1.59	$1.36 = 1 + 0.247\tan^{-1} 10$
20	1.61	1.38
40	1.62	1.38
100	1.62	1.39

N_γ' = bearing capacity factor for base width = N_γ since it is not affected by depth

$\bar{q}$ = γL = effective vertical (or overburden) pressure at pile point

η = **1.0** for all except the Vesić (1975*a*) N_i factors where

$$\eta = \frac{1 + 2K_o}{3}$$

K_o = at-rest earth pressure coefficient defined in Chap. 2.

When making point resistance computations, keep in mind that these bearing-capacity factors are based on the initial in situ soil parameters and not on any soil parameters revised to include driving effects. Initially, of course, any revised values would not be known.

Neglecting the N_γ term and making adjustment for pile weight, we may rewrite Eq. (16-6) as follows:

$$P_{\text{pu}} = A_p[cN_c'd_c + \eta\bar{q}(N_q' - 1)d_q] \tag{16-6a}$$

For $c = s_u$ and $\phi = 0$, the value of $N_q' = 1$ and

$$P_{\text{pu}} = A_p(9s_u) \tag{16-6b}$$

Most designers use N_q', not $(N_q' - 1)$, for piles (but not piers of Chap. 19) when $\phi > 0$ since the factor reduced by 1 is a substantial refinement not justified by estimated soil parameters. The ultimate point capacity is divided by an SF on the order of 1.5 to 3.

Based on results obtained by Coyle and Castello (1981), who back-computed point capacities of a large number of piles in sand, the Hansen bearing-capacity factors of Table 4-4 can be used together with the shape and depth factors of Table 4-5 with a reliability about as good as any other procedure.

The Terzaghi bearing-capacity equation and factors (Table 4-3) are often used even though they are strictly valid only for $L \leq B$. They seem to give about the same point capacity as the Hansen equation for pile depths on the order of 10 to 20 m—probably because the Hansen $N_q d_q$ term equates to the larger Terzaghi N_q factor.

The depth factor d_c was previously shown to give a limiting value on the order of 1.62; the depth factor d_q depends on both the pile depth ratio L/B and ϕ but from the typical values previously given we see that it can be computed to give a limit on the N_q' term as well. From this we see that using any of Eqs. (16-6) gives an unlimited ultimate point resistance P_u but at a decreasing rate. The point capacity increase at a decreasing rate with increasing L/B seems to be approximately what occurs with actual piles, and for this reason *critical depth* methods such as that of Meyerhof (1976), which adjusts both bearing-capacity factors N_c', N_q' using a critical depth ratio of L_c/B that was dependent on the ϕ angle of the soil, are not suggested for use.

The Vesić Method

According to Vesić (1975*a*) the bearing-capacity factors N_i' of Eq. (16-6) can be computed based on the following:

$$N_q' = \frac{3}{3 - \sin\phi}\left\{\exp\left[\left(\frac{\pi}{2} - \phi\right)\tan\phi\right]\tan^2\left(45^\circ + \frac{\phi}{2}\right)I_{\text{rr}}^{\frac{1.333\sin\phi}{1+\sin\phi}}\right\} \tag{16-7}$$

The reduced rigidity index I_{rr} in this equation is computed using the volumetric strain ϵ_v [see Eq. (*d*) of Sec. 2-14] as

$$I_{\text{rr}} = \frac{I_r}{1 + \epsilon_v I_r} \tag{c}$$

The rigidity index I_r is computed using the shear modulus G' and soil cohesion and shear strength s (or τ) as

$$I_r = \frac{G'}{c + \bar{q}\tan\phi} = \frac{G'}{s} \tag{d}$$

When undrained soil conditions exist or the soil is in a dense state, take $\epsilon_v = 0.0$ so that $I_{rr} = I_r$. The value of I_{rr} depends on the soil state (loose, dense; low, medium, or high plasticity) and on the *mean normal stress* defined by $\eta\bar{q}$ with lower I_r values in sand when $\eta\bar{q}$ is low. In clay higher I_r values are used when the water content is high and/or together with a high $\eta\bar{q}$. The lowest values of $I_r \approx 10$ are obtained (or used) for a clay with high OCR and low $\eta\bar{q}$. Estimates for I_r may be made as follows:

Soil	I_r
Sand ($D_r = 0.5$–0.8)	75–150
Silt	50–75
Clay	150–250

Use lower I_r values with *higher* average effective mean normal stress $\eta\bar{q}$.

Since the Vesić method is based on cavity expansion theory, the pile tip behavior is similar to that of the CPT. On this basis Baldi et al. (1981) suggest the following equations for I_r:

For *Dutch cone tip* (see Fig. 3-14*a*):

$$I_r = \frac{300}{f_R} \tag{e}$$

For the *electric cone* (see Fig. 3-15*a*):

$$I_r = \frac{170}{f_R} \tag{f}$$

where f_R = friction ratio in percent given by Eq. (3-10).

The Vesić bearing-capacity factor N'_c term can be computed by one of the following equations:

$$N'_c = (N'_q - 1)\cot\phi \tag{16-7a}$$

When $\phi = 0$ (undrained conditions)

$$N'_c = \frac{4}{3}(\ln I_{rr} + 1) + \frac{\pi}{2} + 1 \tag{16-7b}$$

Janbu's Values

Janbu (1976) computes N'_q (with angle ψ in radians) as follows:

$$N'_q = \left(\tan\phi + \sqrt{1 + \tan^2\phi}\right)^2 \exp(2\psi\tan\phi) \tag{16-7c}$$

For either the Vesić or Janbu methods obtain N'_c from Eq. (16-7*a*) for $\phi > 0$, from Eq. (16-7*b*) when $\phi = 0$. The value of ψ for the Janbu equation is identified in Fig. 16-11*b* and may vary from 60° in soft compressible to 105° in dense soils. Table 16-2 gives a selected range of N'_i values, which can be used for design or in checking the Vesić and Janbu equations.

TABLE 16-2

Bearing-capacity factors N_c' and N_q' by Janbu's and Vesić's equations

A shape factor of s_c 1.3 may be used with Janbu's N_c'. Use program FFACTOR for intermediate values.

	Janbu			Vesić				
ϕ	ψ = 75°	90	105	I_{rr} = 10	50	100	200	500
0°	N_q' = 1.00	1.00	1.00	N_q' = 1.00	1.00	1.00	1.00	1.00
	N_c' = 5.74	5.74	5.74	N_c' = 6.97	9.12	10.04	10.97	12.19
5	1.50	1.57	1.64	1.79	2.12	2.28	2.46	2.71
	5.69	6.49	7.33	8.99	12.82	14.69	16.69	19.59
10	2.25	2.47	2.71	3.04	4.17	4.78	5.48	6.57
	7.11	8.34	9.70	11.55	17.99	21.46	25.43	31.59
20	5.29	6.40	7.74	7.85	13.57	17.17	21.73	29.67
	11.78	14.83	18.53	18.83	34.53	44.44	56.97	78.78
30	13.60	18.40	24.90	18.34	37.50	51.02	69.43	104.33
	21.82	30.14	41.39	30.03	63.21	86.64	118.53	178.98
35	23.08	33.30	48.04	27.36	59.82	83.78	117.34	183.16
	31.53	46.12	67.18	37.65	84.00	118.22	166.15	260.15
40	41.37	64.20	99.61	40.47	93.70	134.53	193.13	311.50
	48.11	75.31	117.52	47.04	110.48	159.13	228.97	370.04
45	79.90	134.87	227.68	59.66	145.11	212.79	312.04	517.60
	78.90	133.87	226.68	53.66	144.11	211.79	311.04	516.60

The American Petroleum Institute [API (1984)] has formulated recommendations for pile design in the form of design parameters for piles in sands, silts, sand silts, and gravels based on a soil description ranging from very loose to very dense. This publication suggests using N_q' ranging from a low of 8 for very loose sand to 50 for a dense gravel or very dense sand. The table is footnoted that the values are intended as guidelines only. These values seem rather low compared to recommendations by most authorities, particularly when considering that piles driven into loose sand will densify it a modest amount in almost all circumstances.

A study of a number of pile load tests by Endley et al. (1979) indicated the 1979 API [reissued as API (1984)] recommendations for N_q' were about 50 percent too low. Be aware that recommended values are not requirements; however, if they are not followed, one must be prepared to justify the use of any alternative values.

Using Penetration Test Data for Pile Point Resistance

For standard penetration test (SPT) data Meyerhof (1956, 1976) proposed

$$P_{\text{pu}} = A_p(40N)\frac{L_b}{B} \leq A_p(380N) \qquad \text{(kN)} \tag{16-8}$$

where N = statistical average of the SPT N_{55} numbers in a zone of about $8B$ above to $3B$ below the pile point (see Fig. 16-11b). Use any applicable SPT N corrections given in Chap. 3.

B = width or diameter of pile point

L_b = pile penetration depth into point-bearing stratum

L_b/B = average depth ratio of point into point-bearing stratum

According to Shioi and Fukui (1982) pile tip resistance is computed in Japan as

$$P_{pu} = q_{ult}A_p \tag{16-9}$$

with the ultimate tip bearing pressure q_{ult} computed from the SPT based on the embedment depth ratio L_b/D into the point-bearing stratum as follows:

Driven piles	$q_{ult}/N = 6L_b/D$	≤ 30 (open-end pipe piles)
	$q_{ult}/N = 10 + 4L_b/D$	≤ 30 (closed-end pipe)
Cast-in-place	$q_{ult} = 300$	(in sand)
	$q_{ult} = 3s_u$	(in clay)
Bored piles	$q_{ult} = 10N$	(in sand)
	$q_{ult} = 15N$	(in gravelly sand)

where this SPT N should be taken as N_{55}.

For *cone penetration data* with $L/B \geq 10$ the point load is estimated by the Japanese as

$$P_{pu} = A_p q_c \qquad \text{(in units of } q_c\text{)} \tag{16-9a}$$

where q_c = statistical average of the cone point resistance in a zone similar to that for N_{55} of Eq. (16-8).

Summarizing Pile Point Capacity

We can compute the ultimate pile point capacity by using Eqs. (16-6), (16-8), or (16-9), depending on the data available. The major problem in using Eq. (16-6) is having access to a reliable angle of internal friction ϕ and soil unit weight γ. We have at least three methods of obtaining the N factors: Table 4-1, Vesić, or Janbu. We should note that Fig. 2-31 indicates that ϕ is pressure-dependent, so laboratory values in the common range of triaxial cell test pressures of 70 to 150 kPa may be several degrees larger than field values at the pile point, which may be 20 or 30 meters down where there is a substantially larger effective normal stress.

In Table 4-4, N_q more than doubles going from $\phi = 34°$ to $40°$; thus, even small variations of 1 or 2° can produce a significant change in the pile point capacity.

The following example will illustrate how some of the methods given here are used.

Example 16-1. The point of a pile of $L = 25$ m is founded into a dense medium-coarse sand deposit, which has an average $N_{70} = 30$ in the zone of influence of about 1.5 m above the tip to 3 m below. The pile is an **HP** 360 × 174 with $d \times b = 361 \times 378$ mm. The GWT is 5 m below the ground surface.

Required. Estimate the point capacity P_u using the several methods presented in this section.

Solution.

$$A_p = d \times b \text{ (including the plug between flanges)} = 0.361 \times 0.378 = \mathbf{0.136}\ \text{m}^2$$

$$N_{55} = N_{70}(70/55) = 30(70/55) = 38$$

With a 1.5 m embedment into dense bearing sand, $L_b = 1.5$ m. We estimate the overburden unit weight $\gamma_s = 16.5$ kN/m^3 since we have no N values or other data.

By Meyerhof's Eq. (16-8). From this we directly obtain

$$P_{\text{pu}} = A_p(40 \times N_{55})L_b/B = 0.136(40 \times 38)(1.5/0.361) = \mathbf{859}\ \text{kN}$$

The maximum recommended limit for the preceding equation is

$$P_{\text{pu}} = A_p 400 N_{55} = 0.136(380 \times 38) = 1964 > 859 \rightarrow \text{use 859 kN}$$

We will also use the other equations for a comparison.

By Hansen's Eq. (16-6)

$$P_{\text{pu}} = A_p(cN_c d_c + \eta \bar{q} N_q' d_q + \tfrac{1}{2}\gamma' B_p N_\gamma)$$

For sand the cN_c term is 0. We can estimate for the medium coarse sand with $N_{70} = 30$ a value of $\phi \approx 36°$ (range from 36 to 50°) from Table 3-4 and in the tip zone $\gamma_{\text{sand}} = \mathbf{17.0}$ kN/m^3. From Table 4-4 we obtain $N_q = \mathbf{37.7}$; $N_\gamma = \mathbf{40.0}$; depth factor $= \mathbf{0.247}$. We then compute

$$\begin{aligned}
d_q &= 1 + 0.247\tan^{-1}(L/B) = 1 + 0.247\tan^{-1}(25/0.361) = \mathbf{1.38}\\
\bar{q} &= 5 \times 16.5 + 18.5(16.5 - 9.807) + 1.5(17.0 - 9.807)\\
&= \mathbf{217.1}\ \text{kPa} \qquad (16.5\ \text{kN/m}^3 \text{ above tip zone and } 17.0\ \text{kN/m}^3 \text{ in tip zone})\\
P_{\text{pu}} &= 0.136\left[217.1 \times 37.7 \times 1.38 + \tfrac{1}{2}(17.0 - 9.807)(0.361 \times 40)\right]\\
&= 0.136(11\,295 + 52) = \mathbf{1543.2}\ \text{kN}
\end{aligned}$$

By Vesić's Method for N_q', N_γ'. Estimate $K_o = 1 - \sin 36° = 0.412$:

$$\eta = \frac{1 + 2 \times 0.412}{3} = 0.61 \rightarrow \eta\bar{q} = 0.61 \times 217.1 = \mathbf{132.4}\ \text{kPa}$$

Based on using $I_{\text{rr}} = 100$, Eq. (16-7), and program FFACTOR (option 10), we obtain

$$\begin{aligned}
N_q' &= 93.2 \ (N_c' \text{ is not needed})\\
N_\gamma &= 40 \text{ from Hansen equation (and Table 4-4)}\\
d_q &= 1.38, \text{ as before}
\end{aligned}$$

Substituting values into Eq. (16-6), we obtain

$$\begin{aligned}
P_{\text{pu}} &= 0.136(132.4 \times 93.2 \times 1.38 + \tfrac{1}{2} \times 7.2 \times 0.361 \times 40)\\
&= 0.136(17\,028.8 + 52.0) = \mathbf{2323}\ \text{kN}
\end{aligned}$$

By Janbu's method [Eq. (16-6) but using N_q' from Eq. (16-7*d*)]. Using program FFACTOR (option 10), for $\phi = 36°$ and estimating $\psi \approx 90°$, we obtain $N_c' = 37.4$; $d_q = 1.38$ as before; $\bar{q} = 217.1$ kPa (as before)

$$N_\gamma' = 40.0 \text{ as in Hansen equation also}$$

Substituting values into Eq. (16-6), we obtain

$$\begin{aligned}
P_{\text{pu}} &= 0.136(217.1 \times 37.4 \times 1.38 + \tfrac{1}{2} \times 7.2 \times 0.361 \times 40)\\
&= 0.136(11\,205.0 + 52.0) = \mathbf{1531}\ \text{kN}
\end{aligned}$$

 Next Page

By Terzaghi's method [$P_{pu} = 0.136(\bar{q}N_q + \frac{1}{2}\gamma' N_\gamma s_\gamma)$], **equation from Table 4-1.** Using $N_q = 47.2$; $N_\gamma = 51.7$; $s_\gamma = 0.8$; $L = 25$ m; $B = 0.361$ m; $A_p = 0.136\ \text{m}^2$; $\bar{q} = 217.1$ kPa; $\gamma' = 17.0 - 9.807 = 7.2$, we obtain

$$P_{pu} = 0.136(217.1 \times 47.2 + \tfrac{1}{2} \times 7.2 \times 0.361 \times 51.7 \times 0.8)$$
$$= 0.136(10\,300.87) = \mathbf{1401}\ \text{kN}$$

A good question is what to use for P_{pu}. We could, of course, average these values, but there are too many computations involved here for a designer to compute a number of point resistances and obtain their average.

Let us instead look at a tabulation of values and see if any worthwhile conclusions can be drawn:

Method	P_{pu}, kN
Hansen	1543.2
Terzaghi	1401.0
Janbu	1531.0
Meyerhof	859.0
Vesić	2323.0

From this tabulation it is evident that the Meyerhof value is too conservative; the Vesić may be too large; but almost any value can be obtained by suitable manipulation of I_{rr} and, similarly with the Janbu equation, with manipulation of the ψ angle.

From these observations it appears that the Hansen equation from Chap. 4 using values from Table 4-4 provides as good an estimate of point capacity as the data usually available can justify. As a consequence that is the only method used in the rest of this text and is included as one of the point capacity contribution methods in the computer program PILCAPAC noted on your diskette and described further in the next section concerning skin resistance.

////

16-9 PILE SKIN RESISTANCE CAPACITY

The skin resistance part of Eq. (16-5) is currently computed using either a combination of total and effective, or only effective, stresses. Some evidence exists that use of only effective stresses gives a better correlation of prediction to load tests; however, both methods are widely used. Preference will depend on the data base of successful usage in a given locale/design office.

Three of the more commonly used procedures for computing the skin resistance of piles in cohesive soils will be given here. These will be called the α, λ, and β methods for the factors used in the skin resistance capacity part of Eq. (16-5). The β method is also used for piles in cohesionless soils. In all cases the skin resistance capacity is computed as

$$\sum_{1}^{n} A_s f_s \qquad \text{(in units of } f_s\text{)} \tag{16-10}$$

where A_s = effective pile surface area on which f_s acts; computed as perimeter $\times$ embedment increment ΔL. Refer to Fig. 16-11a for pile perimeters.

ΔL = increment of embedment length (to allow for soil stratification and variable pile shaft perimeters in the embedment length L)

f_s = skin resistance to be computed, using one of the three methods previously cited