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UNIFIED FACILITIES CRITERIA (UFC)

DEEP FOUNDATIONS

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UNIFIED FACILITIES CRITERIA (UFC)

DEEP FOUNDATIONS

U.S. ARMY CORPS OF ENGINEERS (Preparing Activity)

NAVAL FACILITIES ENGINEERING COMMAND

AIR FORCE CIVIL ENGINEER SUPPORT AGENCY

Record of Changes (changes are indicated by **1**\ ... /**1**/)

Change No.	Date	Location

This UFC supersedes TI 818-02, dated 3 August 1998. The format of this UFC does not conform to UFC 1-300-01; however, the format will be adjusted to conform at the next revision. The body of this UFC is the previous TI 818-02, dated 3 August 1998.

FOREWORD

\1\
The Unified Facilities Criteria (UFC) system is prescribed by MIL-STD 3007 and provides planning, design, construction, sustainment, restoration, and modernization criteria, and applies to the Military Departments, the Defense Agencies, and the DoD Field Activities in accordance with USD(AT&L) Memorandum dated 29 May 2002. UFC will be used for all DoD projects and work for other customers where appropriate. All construction outside of the United States is also governed by Status of forces Agreements (SOFA), Host Nation Funded Construction Agreements (HNFA), and in some instances, Bilateral Infrastructure Agreements (BIA.) Therefore, the acquisition team must ensure compliance with the more stringent of the UFC, the SOFA, the HNFA, and the BIA, as applicable.

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- Whole Building Design Guide web site http://dod.wbdg.org/.

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US Army Corps
of Engineers®

TI 818-02
3 August 1998

Technical Instructions

Design of Deep Foundations

Headquarters
U.S. Army Corps of Engineers
Engineering Division
Directorate of Military Programs
Washington, DC 20314-1000

TECHNICAL INSTRUCTIONS

Design of Deep Foundations

Record of Changes (changes indicated by \1\..../1/)
No. Date Location

This Technical Instruction supersedes EI 02C097, dated 1 July 1997.
(EI 02C097 text is included in this Technical Instruction and may carry EI 02C097 identification.)

FOREWORD

These technical instructions (TI) provide design and construction criteria and apply to all U.S. Army Corps of Engineers (USACE) commands having military construction responsibilities. TI will be used for all Army projects and for projects executed for other military services or work for other customers where appropriate.

TI are living documents and will be periodically reviewed, updated, and made available to users as part of the HQUSACE responsibility for technical criteria and policy for new military construction. CEMP-ET is responsible for administration of the TI system; technical content of TI is the responsibility of the HQUSACE element of the discipline involved. Recommended changes to TI, with rationale for the changes, should be sent to HQUSACE, ATTN: CEMP-ET, 20 Massachusetts Ave., NW, Washington, DC 20314-1000.

TI are effective upon issuance. TI are distributed only in electronic media through the TECHINFO Internet site *http://www.hnd.usace.army.mil/techinfo/index.htm* and the Construction Criteria Base (CCB) system maintained by the National Institute of Building Sciences at Internet site *http://www.nibs.org/ccb/*. Hard copies of these instructions produced by the user from the electronic media should be checked against the current electronic version prior to use to assure that the latest instructions are used.

FOR THE DIRECTOR OF MILITARY PROGRAMS:

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CEMP-E	DEPARTMENT OF THE ARMY U.S. Army Corps of Engineers Washington, DC 20314-1000	EI 02C097

Engineering Instructions
No. 02C097

01 July 1997

DESIGN OF DEEP FOUNDATIONS

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Chapter 1
Introduction

1. Purpose

This publication presents data, principles, and methods for use in planning, design, and construction of deep foundations. Deep foundations are literally braced (supported) column elements transmitting structure loads down to the subgrade supporting medium.

2. Applicability

These instructions are applicable to all HQUSACE elements and USACE comands.

3. Scope

General information with respect to the selection and design of deep foundations is addressed herein. Single and groups of driven piles and drilled shafts under axial and lateral static loads are treated. Some example problems and the most widely accepted computer methods are introduced. This publication is not intended for hydraulic structures; however, it does provide the following:

a. Guidance is provided to assist the efficient planning, design, and quality verification of the deep foundation.

b. Guidance is not specifically provided for design of sheet piles used as retaining walls to resist lateral forces or for the design of stone columns. Other foundation structures may be designed as discussed below:

(1) Shallow foundations will be designed using TM 5-818-1, “Soils and Geology; Procedures for Foundation Design of Buildings and Other Structures (Except Hydraulic Structures).”

(2) Refer to *Foundations* (Pile Buck Inc. 1992) and *Pile Foundations in Engineering Practice* (Prakash and Sharma 1989) for guidance on design of deep foundations subject to dynamic load.

c. Guidance for construction of deep foundations is provided only in minor detail. For construction of deep foundations, the following references are offered:

(1) Some guidance for selection of pile driving equipment and construction of driven piles is provided in TM 5-849-1, “Pile Driving Equipment.”

(2) Guidance for construction of drilled shafts is available in FHWA-HI-88-042, “Drilled Shafts: Construction Procedures and Design Methods” and Association of Drilled Shaft Contractors (ADSC) Publication, “Drilled Shaft Inspector's Manual.”

4. References

Appendix A contains a list of references used in this publication.

5. General Design Methodology

A single drilled shaft or a group of driven piles is typically designed to support a column load. The number of driven piles in a group is determined by dividing the column load by the design load of a single pile. The piles should be arranged in the group to provide a spacing of about three to four times the pile diameter B up to $6B$. The diameter of the piles may be increased to reduce the size of the pile cap if appropriate. Table 1-1 describes a general design methodology. Other design methodology aspects are the following:

a. *Load factor design*. This publication applies load factors for design (LFD) of the structural capacity of deep foundations. The sum of the factored loads shall not exceed the structural resistance and the soil resistance. The LFD, the structural resistance, and the soil resistance are all related to the load factors as follows:

(1) Definition. The LFD may be defined as a concept which recognizes that the different types i of loads Q_i that are applied to a structure have varied probabilities of occurence. Examples of types of loads applied to a structure include the live load Q_{LL}, dead load Q_{DL}, wind load Q_{WL}, and earthquake load Q_{EL}. The probability of occurrence of each load is accounted for by multiplying each Q_i by a load factor $F_i >$ 1.0. The value of F_i depends on the uncertainty of the load.

(2) Structural resistance. The sum of the factored loads shall be less than the design strength

Table 1-1
General Design Methodology for Deep Foundations

Step	Evaluate	Description
1	Soil profile of selected site	Develop depth profiles of water content, liquid and plastic limits, unit weight and overburden pressure, and unconsolidated-undrained shear strength to a depth of a least twice the width of a pile group or five times the tip diameter of drilled shafts. Estimate shear strength and elastic soil modulus from results of in situ and laboratory triaxial tests. Determine water table depth and extent of perched water. Perform consolidation/swell tests if soil is potentially expansive or collapsible and plot compression and swell indices and swell pressure with depth. Evaluate lateral modulus of subgrade reaction profile. Compare soil profile at different locations on the site. See Chapters 1-6 for further details.
2	Group similar soils	Group similar soils and assign average parameters to each group or strata.
3	Depth of base	Select a potentially suitable stratum that should support the structural loads such as a firm, nonswelling, and noncollapsing soil of low compressibility.
4	Select type of deep foundation	Select the type of deep foundation such as driven piles or drilled shafts depending on requirements that include vertical and lateral load resistance, economy, availability of pertinent construction equipment, and experience. Environmental considerations include allowable noise level, vibrations, overhead clearance, and accessibility of equipment to the construction site. Soil conditions such as potential ground rise (heave) or loss and expansion/collapse also influence type of foundation. See Chapter 1 for further information on type and selection of deep foundations.
5	Check Q_a with structural capacity	Allowable pile or shaft load Q_a shall be within the structural capacity of the deep foundation as described by methodology in Chapter 2.
6	Design	The design procedure will be similar for most types of deep foundations and requires evaluation of the ultimate pile capacity $Q_u = Q_{su} + Q_{bu}$ where Q_{su} = ultimate skin friction resistance and Q_{bu} = ultimate end bearing capacity. Reasonable estimates of vertical and lateral displacements under the probable design load Q_d are also required. Q_d should be within levels that can be tolerated by the structure over its projected life and should optimize operations. $Q_d \le Q_a$ where Q_a = allowable pile capacity. $Q_a = Q_u/FS$ = factor of safety. A typical $FS = 3$ if load tests are not performed or if the deep foundation consists of a group of driven piles. $FS = 2$ if load tests are performed or 2.5 if wave equation analyses of the driven piles calibrated with results of pile driving analyzer tests. Design for vertical loads is given in Chapter 3, lateral loads in Chapter 4, and pile groups in Chapter 5.
7	Verify the design	The capability of the deep foundation to support the structure shall be verified by static load and dynamic tests. These tests are usually nondestructive and allow the tested piles or drilled shafts to be used as part of the foundation. See Chapter 6 for further details.
8	Addition to existing structure	Calculate displacements of existing deep or shallow foundations to determine the ability to carry existing and additional loads and to accommodate new construction.
9	Effect on adjacent structure	Evaluate changes in bearing capacity and groundwater elevation and effect of any action which can result in settlement or heave of adjacent structures.

$$\phi_{pf} Q_{cap} \geq \sum F_i Q_i \tag{1-1}$$

where

ϕ_{pf} = performance factor for structural capacity

Q_{cap} = nominal structural capacity, kips

F_i = Load factor of type i

Q_i = applied load of type i

Guidance for analysis of structural capacity is given in Chapter 2.

(3) Soil resistance. The sum of the factored loads shall be less than the ability of the soil to resist the loads. This evaluation may be determined by factors of safety (*FS*) or by load factors. Factors of safety are often empirical values based on past experience and may lead to a more conservative design than the LFD concept. The *FS* and the LFD are presented as:

(a) Global *FS*. The allowable load may be evaluated with global *FS*

$$Q_a = \frac{Q_u}{FS} \geq \Sigma F_i Q_i \tag{1-2a}$$

where

Q_a = allowable load that can be applied to the soil, kips

Q_u = ultimate pile capacity, kips

FS = global factor of safety

The approach taken throughout this publication is to select a global *FS* for analysis of soil resistance rather than partial *FS* or load factors. Chapters 3 through 5 provide guidance for design of deep foundations to maintain loads within the allowable soil bearing capacity and displacement. Chapter 6 provides guidance for design verification.

(b) Load factor design. Analysis of soil resistance may also be determined by the LFD concept using performance factors

$$\phi_{pfq} Q_u \geq \Sigma F_i Q_i \tag{1-2b}$$

where ϕ_{pfq} = performance factor appropriate to the ultimate pile capacity. Performance factors ϕ_{pfq} depend on the method of evaluating Q_u and the type of soil resistance, whether end bearing, skin friction, uplift, or a group capacity. Values for ϕ_{pfq} and examples of load factor analysis are available in National Cooperative Highway Research Program Report No. 343, "Manuals for the Design of Bridge Foundations" (Barker et al. 1991). Load factors and factors of safety taken in combination can lead to an uneconomical foundation design. The design should be verified by guidance in Chapter 6.

b. Unusual situations. Consideration should be given to obtaining the services and advice of specialists and consultants in foundation design where conditions are unusual or unfamiliar or structures are economically significant. Some unusual situations for deep foundations, discussed below, include expansive clay, underconsolidated soil, and coral sands.

(1) Expansive clay. The swell of expansive clay can cause an uplift force on the perimeter area of deep foundations that can force the foundation to move up and damage the structure connected to the deep foundation.

(2) Underconsolidated soil. The settlement of underconsolidated soil can cause negative skin friction on the perimeter area of the deep foundation that can increase the end-bearing load, which results in an increase in settlement of the foundation.

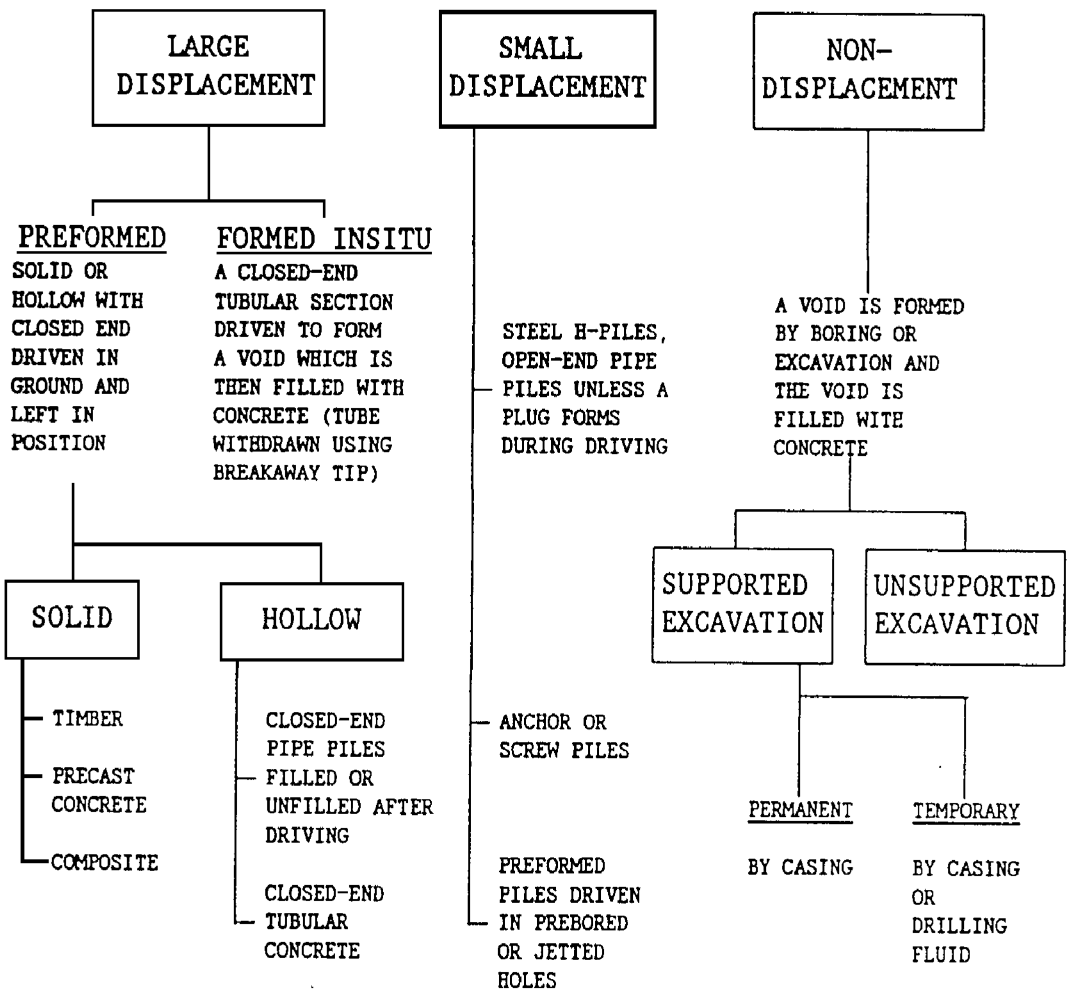
(3) Coral sands. Piles in coral sands may indicate low penetration resistance during driving and an apparent low bearing capacity, but the penetration resistance often increases over time as a result of the dissipation of excess pore pressure. Driving of piles into cemented, calcareous sands can crush the soil and lower the lateral stress, which results in a low value for skin friction and bearing capacity.

c. Computer program assistance. Design of a deep foundation is normally accomplished with the assistance of several computer programs. Brief descriptions of appropriate computer programs are provided in Chapters 3 through 6. Copies of user's manuals and programs are available through the Engineering Computer Programs Library, Information Technology Laboratory, U.S. Army Engineer Waterways Experiment Station, CEWES-IM-DS.

6. Types of Deep Foundations

Deep foundations are classified with respect to displacements as large displacement, small displacement, and nondisplacement, depending on the degree to which installation disturbs the soil supporting the foundation (Table 1-2). Large displacement and small displacement piles are fabricated prior to installation and driven into the ground, while nondisplacement piles are constructed in situ and often are called drilled shafts. Augered cast concrete shafts are also identified as drilled shafts in this publication.

Table 1-2
Types of Deep Foundations



a. Large displacement piles. Driven piles are classified by the materials from which the pile is constructed, i.e., timber, concrete, or filled or unfilled steel pipe.

(1) Timber piles. These are generally used for comparatively light axial and lateral loads where foundation conditions indicate that piles will not be damaged by driving or exposed to marine borers. Overdriving is the greatest cause of damage to timber piles. Pile driving is often decided by a judgment that depends on the pile, soil condition, and driving equipment. Overdriving typically occurs when the dynamic stresses on the pile head exceed the ultimate strength of the pile. Timber piles can broom at the pile tip or head, split, or break when overdriven. Such piles have an indefinite life when constantly submerged or where cut off below the groundwater level. Some factors that might affect the performance of timber piles are the following:

(a) Splicing of timber piles is expensive and time-consuming and should be avoided. The full bending resistance of timber pile splices may be obtained by a concrete cover (Figure 1-1a) (Pile Buck Inc. 1992). Other transition splicers are available to connect timber with cast concrete or pipe piles.

(b) Tips of timber piles can be protected by a metal boot (Figure 1-1b).

(c) Timber piles are normally treated with creosote to prevent decay and environmental attack.

(d) American Society for Testing and Materials (ASTM) D 25 provides physical specifications of round timber piles. Refer to Federal Specifications TT-W-00571J, "Wood Preservation: Treating Practices," for other details.

(2) Precast concrete piles. These piles include conventionally reinforced concrete piles and prestressed concrete piles. Reinforced concrete piles are constructed with an internal reinforcement cage consisting of several longitudinal bars and lateral ties, individual hoops, or a spiral. Prestressed concrete piles are constructed using steel rods or wire strands under tension as reinforcement. Since the concrete is under continuous compression, transverse cracks tend to remain closed; thus, prestressed piles are usually more durable than conventionally reinforced piles. Influential factors for precast concrete piles include splices and steel points.

(a) Various splices are available to connect concrete piles. The splice will provide the tensile strength required during driving when the resistance to driving is low. Figure 1-2a illustrates the cement-dowel splice. Refer to "Foundations" (Pile Buck Inc. 1992) for additional splices.

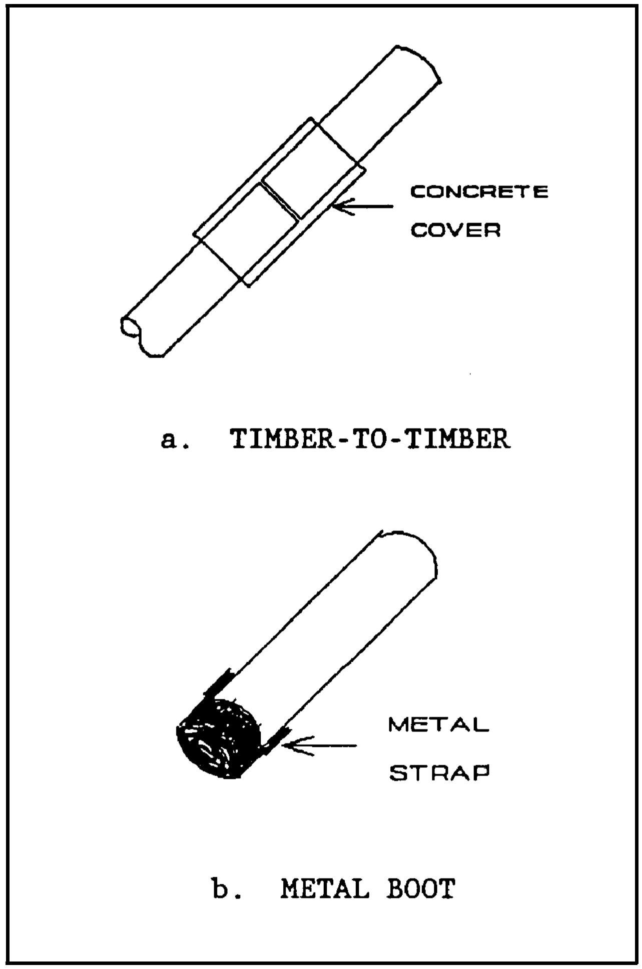


Figure 1-1. Timber pile splice and boot

(b) Special steel points can be attached to precast precast piles during casting of the piles and include steel H-pile tips or cast steel shoes (Figure 1-2).

(3) Raymond step-tapered piles. These consist of a corrugated steel shell driven into the ground using a mandrel. The shell consists of sections with variable diameters that increase from the tip to the pile head. A mandrel is a heavy, rigid steel tube shaped to fit inside the shell. The mandrel is withdrawn after the shell is driven and the shell filled with concrete. Raymond step-tapered piles are predecessors of drilled shafts and are still popular in the southern United States.

(4) Steel piles. These are generally H-piles and pipe piles. Pipe piles may be driven either "open-end" or "closed-end." Steel piles are vulnerable to corrosion, particularly in saltwater; however, experience indicates they are not

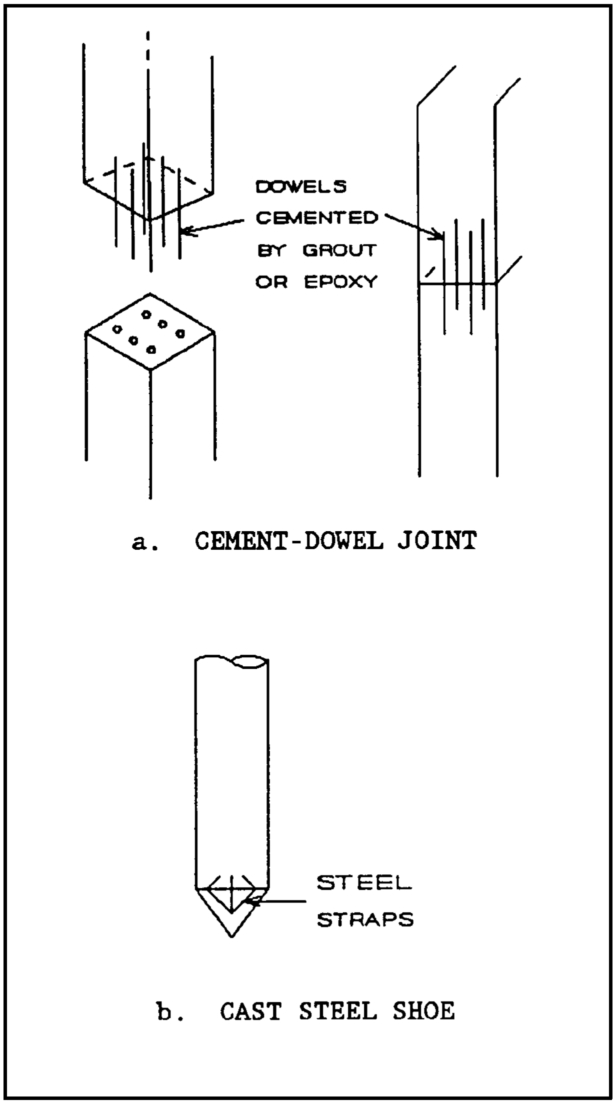


Figure 1-2. Concrete pile splice and boot

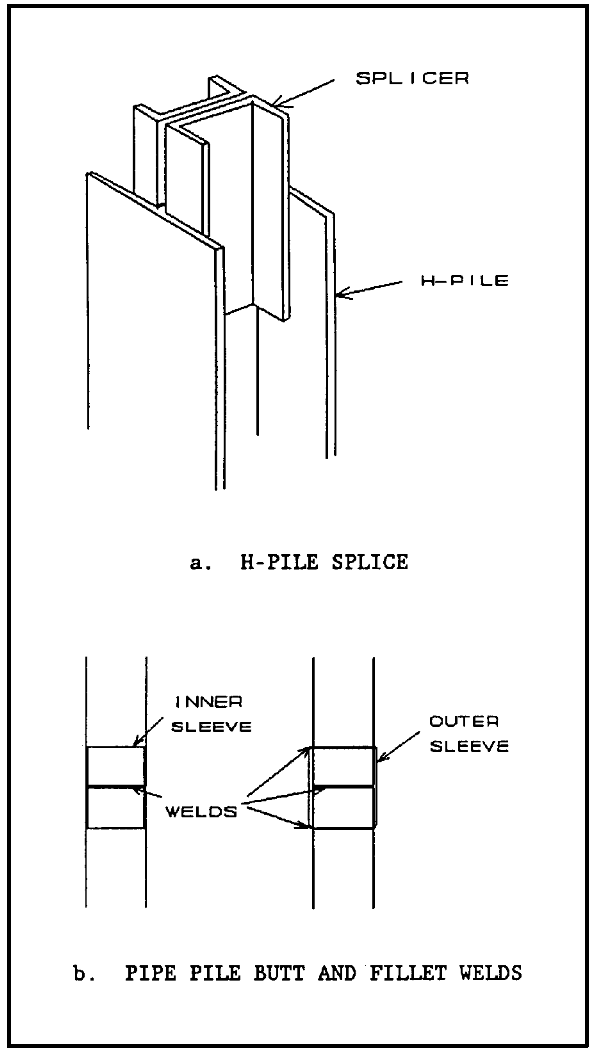


Figure 1-3. Steel pile splices

significantly affected by corrosion in undisturbed soil. Schematics of H-piles and pipe piles are presented in Figure 1-3.

(a) Steel H-piles. This type can carry larger loads, both axially and in bending, than timber piles and can withstand rough handling. H-piles can be driven into dense soil, coarse gravel, and soft rock with minimum damage, and cause minimal displacement of the surrounding soil while being driven. Hardened and reinforced pile tips should be used where large boulders, dense gravel, or hard debris may damage the pile. Splices are commonly made with full penetration butt welds or patented splicers (Figure 1-3a). H-piles can bend during driving and drift from planned location. Thus, H-piles may not be suitable when tolerance is small with respect to location and where absolute plumbness is required. Table 1-3 lists commonly available H-piles together with properties and dimensions.

(b) Steel pipe piles. Commonly used steel pipe piles are listed in Appendix B together with properties and dimensions. Steel pipe piles are generally filled with concrete after driving to increase the structural capacity. If the soil inside the pipe is removed during driving, open-end piles in cohesionless soil will cause less soil displacement and compaction, and in cohesive soils will cause less heaving of adjacent ground and nearby piles. If the soil inside the pipe is not removed during driving, the pipe becomes plugged and acts as a closed-end displacement pile. Criteria are presently unavailable for computing the depth at which a driven, open-end pile will plug. In cases where the foundation contains boulders, soft rock, or other obstructions, the open-end pile permits inspection after removal of the plug material and ensures that the load will be transferred directly to the load-bearing stratum. Splices are commonly made by full penetration butt welds or fillet wells (Figure 1-3b) or patented splicers.

(5) Compaction piles. These are sometimes driven with the objective of increasing the density of loose, cohesionless soils and reducing settlement. Piles with a heavy taper are often most effective in deriving their support from friction.

b. Nondisplacement piles. This pile consists of a drilled shaft with a concrete cylinder cast into a borehole. Normally, the drilled shaft does not cause major displacement of the adjacent ground surface. The hole is usually bored with a short flight or bucket auger. Loss of ground could occur if the diameter of the hole is decreased because of inward displacement of soft soil or if there is caving of soil from the hole perimeter. Such unstable boreholes require stabilization by the use of slurry or slurry and casing. Drilled shafts are not subject to handling or driving stresses and therefore may be designed only for stresses under the applied service loads. Nondisplacement may be categorized as follows:

(1) Uncased shafts. Figure 1-4 illustrates a typical uncased drilled shaft with an enlarged base. The base is not perfectly flat because the shaft is drilled first, then the belling tool rotates in the shaft. Uncased shafts may be constructed in firm, stiff soils where loss of ground is not significant. Examples of uncased shaft are given in the American Concrete Institute (ACI) *Manual of Concrete Practice* (1986). Other terms used to describe the drilled shaft are “pier” or “caisson.” Large shafts greater then 36 inches in diameter are often called caissons. The term “pile” is commonly associated with driven deep foundations of relatively small diameter or cross section.

(2) Cased shafts. A cased shaft is made by inserting a shell or casing into almost any type of bored hole that requires stabilization before placing concrete. Boreholes are caused where soil is weak and loose, and loss of ground into the excavation is significant. The bottom of the casing should be pushed several inches into an impervious stratum to seal the hole and allow removal of the drilling fluid prior to completion of the excavation and concrete placement. If an impervious stratum does not exist to push the casing into, the concrete can be placed by tremie to displace the drilling fluid.

(3) Drilling fluid shafts. Shafts can be installed in wet sands using drilling fluid, with or without casing. This procedure of installing drilled shafts can be used as an alternative to the uncased and cased shafts discussed previously.

(4) Pressure-grouted shafts. A special type of nondisplacement deep foundation is the uncased auger-placed grout shaft. This shaft is constructed by advancing a continuous-flight, hollow-stem auger to the required depth and filling the hole bored by the concrete grout under pressure as the auger is withdrawn. Careful inspection is required during installation, and shaft continuity should be verified by a combination of load tests and nondestructive testing as described in Chapter 6.

7. Selection of Deep Foundations

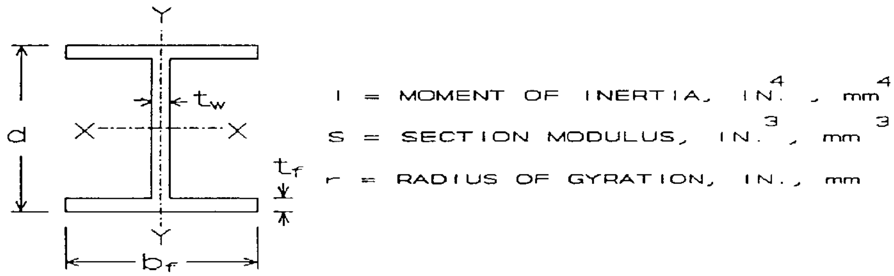
Deep foundations provide an efficient foundation system for soils that do not have a shallow, stable bearing stratum. Selection of a deep foundation requires knowledge of its characteristics and capacity.

a. Characteristics. Information adequate for reaching preliminary conclusions about types of driven piles or drilled shafts to be selected for a project is given in Table 1-4. This table lists major types of deep foundations with respect to capacity, application, relative dimensions, and advantages and disadvantages. Refer to *Foundations* (Pile Buck Inc. 1992) for additional information. Information in the table provides general guidelines in the selection of a type of deep foundation. Relevant codes and standards should be consulted with respect to allowable stresses. A cost analysis should also be performed that includes installation, locally available practices, time delays, cost of load testing program, cost of a pile cap, and other elements that depend on different types of deep foundations.

b. Capacity. Deep foundations transmit structural loads to deep strata that are capable of sustaining the applied loads. Accurate predictions of load capacity and settlement are not always possible. Adequate safety factors are therefore used to avoid excessive movement that would be detrimental to the structure that is supported and to avoid excessive stress in the foundation. Driven piles or drilled shafts are often used to resist vertical inclined, lateral, or uplift forces and overturning moments which cannot otherwise be resisted by shallow footings. These foundations derive their support from skin friction along the embedded length and by end bearing at the tip (base). Both factors contribute to the total ultimate pile capacity, but one or the other is usually dominant depending on the size, load, and soil characteristics. The capacity of deep foundation is influenced by several factors:

(1) Design limits. The limiting design criterion is normally influenced by settlement in soft and moderately stiff soil, and bearing capacity in hard soil or dense sand, and by pile or shaft structural capacity in rock.

Table 1-3
Standard H-piles; Dimensions and Properties (AISC 1969)



a. English Units

Designation	Area A, in.^{2}	Depth d, in.	Flange Width b_f, in.	Flange Thickness t_f, in.	Web Thickness t_w, in.	Axis X-X I, in.^{4}	Axis X-X S, in.^{3}	Axis X-X r, in.	Axis Y-Y I, in.^{4}	Axis Y-Y S, in.^{3}	Axis Y-Y r, in.
HP14 x 117	34.4	14.21	14.885	0.805	0.805	1220	172.0	5.96	443.0	59.5	3.59
x 102	30.0	14.01	14.785	0.705	0.705	1050	150.0	5.92	380.0	51.4	3.56
x 89	26.1	13.83	14.695	0.615	0.615	904	131.0	5.88	326.0	44.3	3.53
x 73	21.4	13.61	14.585	0.505	0.505	729	107.0	5.84	261.0	35.8	3.49
HP13 x 100	29.4	13.15	13.205	0.765	0.765	886	135.0	5.49	294.0	44.5	3.16
x 87	25.5	12.95	13.105	0.665	0.665	775	117.0	5.45	250.0	38.1	3.13
x 73	21.6	12.75	13.005	0.565	0.565	630	98.8	5.40	207.0	31.9	3.10
x 60	17.5	12.54	12.900	0.460	0.460	503	80.3	5.36	165.0	25.5	3.07
HP12 x 84	24.6	12.28	12.295	0.685	0.685	650	106.0	5.14	213.0	34.6	2.94
x 74	21.8	12.13	12.215	0.610	0.610	569	93.8	5.11	186.0	30.4	2.92
x 63	18.4	11.94	12.125	0.515	0.515	472	79.1	5.06	153.0	25.3	2.88
x 53	15.5	11.78	12.045	0.435	0.435	393	66.8	5.03	127.0	21.1	2.86
HP10 x 57	16.8	9.99	10.225	0.565	0.565	294	58.8	4.18	101.0	19.7	2.45
x 42	12.4	9.70	10.075	0.420	0.420	210	43.4	4.13	71.7	14.2	2.41
HP8 x 36	10.6	8.02	8.155	0.445	0.445	119	29.8	3.36	40.3	9.86	1.95

b. Metric Units

Designation	Area A, mm^2	Depth d, mm	Flange Width b_f, mm	Flange Thickness t_f, mm	Web Thickness t_w, mm	Axis X-X I, 10^6mm^4	Axis X-X S, 10^3mm^3	Axis X-X r, mm	Axis Y-Y I, 10^6mm^4	Axis Y-Y S, 10^3mm^3	Axis Y-Y r, mm
HP360 x 174	22200	361	378	20.4	20.4	504	2810	151	184	974	91.0
x 152	19400	356	376	17.9	17.9	439	2470	150	159	846	90.5
x 132	16900	351	373	15.6	15.6	375	2140	149	135	724	89.4
x 108	13800	346	370	12.8	12.8	303	1750	148	108	584	88.5
HP330 x 149	19000	334	335	19.4	19.4	368	2200	139	122	728	80.1
x 129	16400	329	333	16.9	16.9	315	1910	139	104	625	79.6
x 109	13900	324	330	14.4	14.4	263	1620	138	86.3	523	78.8
x 89	11300	319	328	11.7	11.7	211	1320	137	68.9	420	78.1
HP310 x 125	15900	312	312	17.4	17.4	270	1730	130	88.2	565	74.5
x 110	14100	308	310	15.5	15.4	237	1540	130	77.1	497	73.9
x 93	11900	303	308	13.1	13.1	196	1290	128	63.9	415	73.3
x 79	10000	299	306	11.0	11.0	163	1090	128	52.6	344	72.5
HP250 x 85	10800	254	260	14.4	14.4	123	969	107	42.3	325	62.6
x 62	7970	246	256	10.7	10.5	87.5	711	105	30.0	234	61.4
HP200 x 53	6820	204	207	11.3	11.3	49.8	488	85.5	16.7	161	49.5

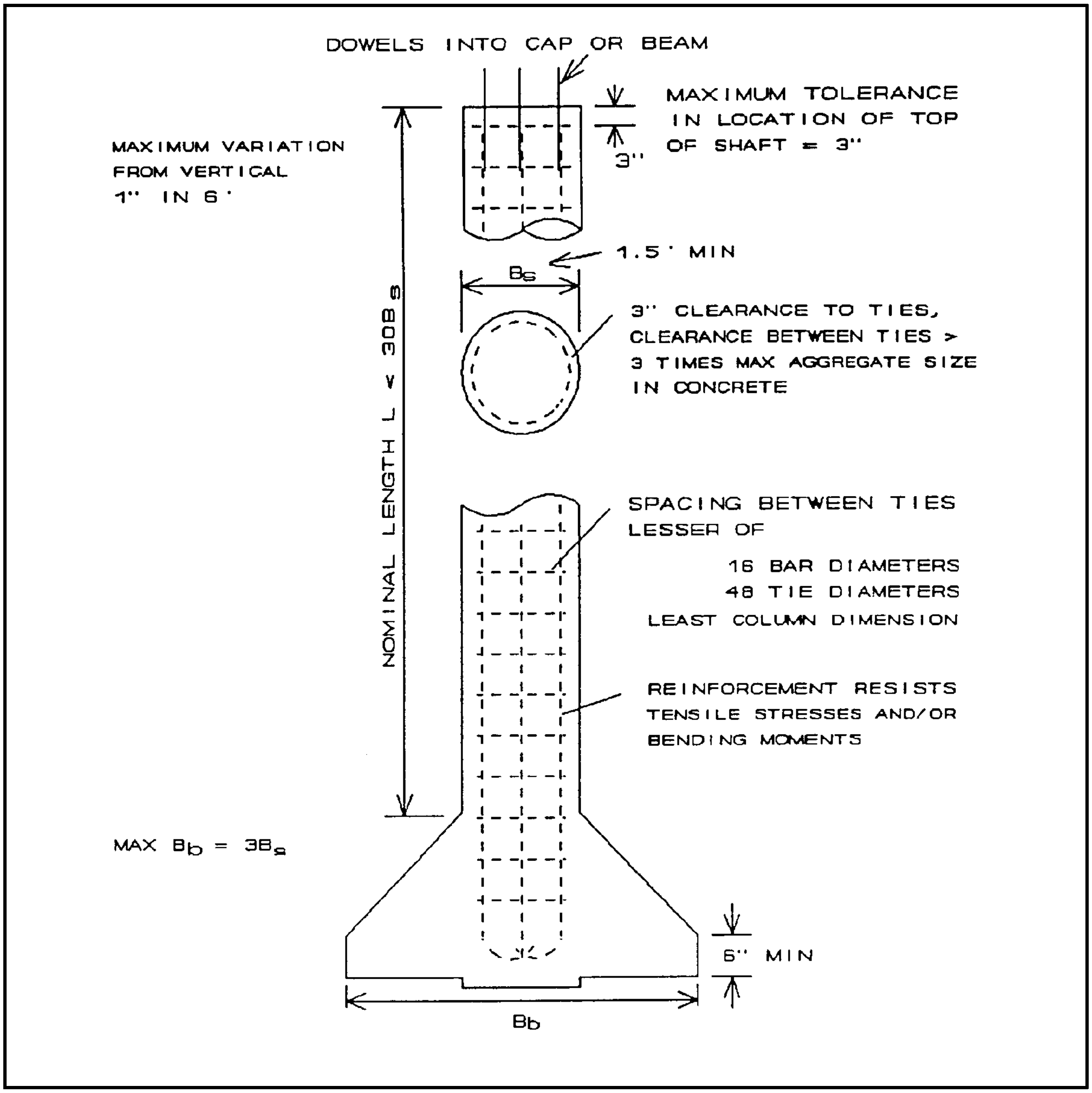


Figure 1-4. Drilled shaft details (1 in. = 25.4 mm)

(2) Skin resistance mobilization. Full skin resistance is typically mobilized within 0.5 inch of displacement, while end bearing may not be fully mobilized until displacements exceed 10 to 20 percent of the base diameter or underream for drilled shafts, unless the tip is supported by stiff clay, dense sand, or rock. Figure 1-5 illustrates an example of the vertical axial load displacement behavior of a single pile or drilled shaft. The load-displacement behavior and displacements that correspond to ultimate load are site specific and depend on the results of analyses. These analyses are given in Chapter 3.

(3) Lateral loads. Lateral load capacity of a pile or drilled shaft is directly related to the diameter, thus increasing the diameter increases the load-carrying capacity. For a drilled shaft that sustains no axial load, the cost of construction may be optimized by the selection of rigid shafts without underreams and with length/diameter ratios less than 10. The selected shaft dimensions should minimize the volume of concrete required and maximize constuction efficiency. The lateral load capacity of driven piles may be increased by increasing the number of piles

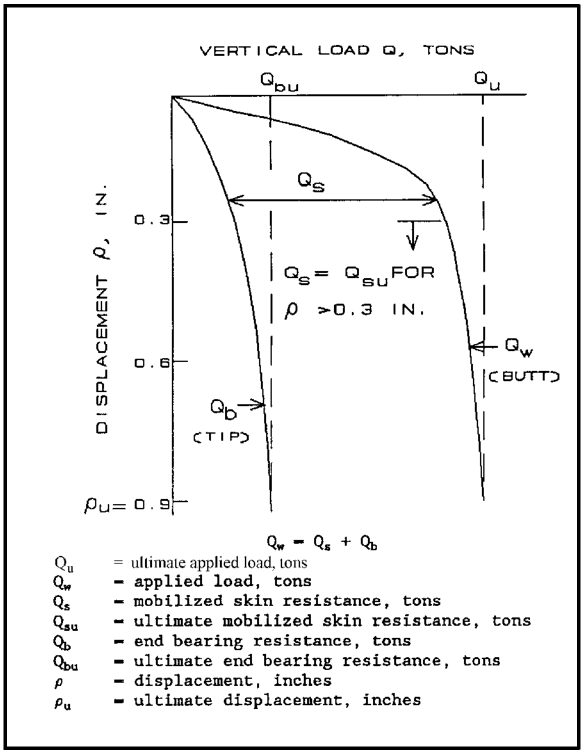


Figure 1-5. Axial-load deflection relationship

and battering piles in a pile group. Batter piles are efficient in resisting lateral loads but significantly reduce ductility of the pile group in the lateral direction, resulting in a brittle failure. Vertical piles, though less efficient in resisting lateral loads, are also less stiff and do not fail suddenly. These conflicting characteristics need to be balanced in design, and they are considered critical where seismic or dynamic lateral loads are involved.

c. Applications. Driven pile groups are typically used by the Corps of Engineers to support locks, dry docks, and other facilities constructed in river systems, lakes, lagoons, and other offshore applications. Drilled shafts typically support many permanent onshore structures such as administrative buildings, warehouses, dormitories, and clinics. Drilled shafts are divided into two groups: displacement and nondisplacement.

(1) Displacement. Driven pile foundations are usually preferable in loose, cohesionless, and soft soils, especially where excavations cannot support fluid concrete and where the depth of the bearing stratum is uncertain. Groundwater conditions can be a deciding factor in the selection of driven piles rather than drilled shafts. Uncased shafts are generally excluded from consideration where artesian pressures are present. Often more than one type of driven pile may meet all requirements for a particular structure. Driven piles according to their application are presented in Figure 1-6.

(a) Figures 1-6a and 1-6b illustrate piles classified according to their behavior as end-bearing or friction piles. A pile embedded a significant length into stiff clays, silts, and dense sands without significant end bearing resistance is usually a friction pile. A pile driven through relatively weak or compressible soil to an underlying stronger soil or rock is usually an end-bearing pile.

(b) Piles designed primarily to resist upward forces are uplift or tension piles (Figure 1-6c), and the resistance to the upward force is by a combination of side (skin) friction and self weight of the pile.

(c) Lateral forces are resisted either by vertical piles in bending (Figure 1-6d) or by batter piles or groups of vertical and batter piles (Figure 1-6e).

(d) Piles are used to transfer loads from above water structures to below the scour depth (Figure 1-6f). Piles are also used to support structures that may be endangered by future adjacent excavations (Figure 1-6g). In order to prevent undesirable movements of structures on shrink/swell soils, a pile anchored as shown in Figure 1-6h can be used.

(2) Nondisplacement. Drilled shafts are especially suitable for supporting large column loads of multistory structures and bridge abutments or piers. They are suitable for resisting large axial loads and lateral loads applied to the shaft butt (top or head) resulting from wind forces; these are also used for resisting uplift thrust applied to the shaft perimeter through soil-shaft interface friction and from heave of expansive soil. Figure 1-7 illustrates example load ranges for drilled shafts in different soils. The loads shown are for guidance only and can vary widely from site to site. Cylindrical shafts are usually preferred to underreamed ones because of ease

in construction and ease in inspection. Table 1-5 provides further details of the applications, advantages, and disadvantages of drilled shafts. Othre aspects of drilled shafts include:

(a) Drilled shafts maysecure much or all of their vertical load capacity from frictional side resistance (Figure1-7a). An enlarged base using a bell or underream may also increase the vertical load capacity, provide uplif resistance to pullout loads, an resist uplift thrust from

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Table 1-4
Characteristics of Deep Foundations

Pile Type	Maximum Length, ft	Optimum Length, ft	Diameter Width, in.	Maximum Allowable Normal Stresses, psi	Maximum Allowable Bending Stresses, psi	Material Specifications Standards	Maximum Load tons	Optimum Load tons	Advantages	Disadvantages	Remarks
Driven Piles Cast-in-place concrete placed without mandrel	150	30-80	Butt: 12-18	Steel shell: 9,000 Concrete: 0.25 f'_c	Compression : 0.40 f'_c Tension: 0	ACI *Manual of Concrete Practice*	150	40-100	Easy to inspect, easy to cut, resistant to deterioration, high lateral capacity, capable of being re-driven, cave-in prevented by shell	Difficult to splice, displacement pile, vulnerable to damage from hard driving	Best suited for medium-length friction pile
Cast-in-place concrete driven with mandrel	Tapered: 40 Step tapered: 120	Tapered: 15-35 Step tapered: 40-60	Tip: 8, Butt: # 23 Step tapered: # 17	Steel: 9,000, $ 1 in. thick Concrete: 0.25 f'_c	Compression: 0.40 f'_c Tension: 0	ACI *Manual of Concrete Practice*	75	30-60	Easy to inspect, easy to cut, easy to handle, resistant to decay, high skin friction in sand, resistant to damage from hard driving	Not possible to re-drive, difficult to splice, displacement pile, vulnerable to collapse while adjacent piles are driven	Best suited for medium-length friction pile
Rammed concrete	60	---	17-26	0.25 f'_c	---	ACI *Manual of Concrete Practice*	300	60-100	Low initial cost, large bearing area, resistant to deterioration, resistant to damage from hard driving	Hard to inspect, displacement pile, not possible to form base in clay	Best suited where layer of dense sand is near ground surface
Composite	180	60-120	Depends on materials	Controlled by weakest materials	---	See Note	200	30-80	Resistant to deterioration, resistant to damage from driving, high axial capacity, long lengths at low initial cost	Hard to inspect, difficult in forming joint	Usual combinations are: cast-in-place concrete over timber o H-steel or pipe pile
Auger Cast Concrete Shafts	60	24	---	0.25 f'_c	---	ACI *Manual of Concrete Practice*	40	---	No displacement, low noise level, low vibration, low initial cost	Construction difficult when soils unfavorable, low capacities, difficult to inspect	Best suited where small loads are to be supported
Drilled Shafts	200	Shaft: # 120 Underreams: # 240	---	0.25 f'_c	---	ACI 318	Soil: 3,000 Rock: 7,000	200-400	Fast construction, high load capacity, no noise or vibration, no displacement, possible to drill through obstruction, can eliminate caps	Field inspection of construction critical, careful inspection necessary for casing method	Best suited for large axial lateral loads and small, isolated loads where soil conditions are favorable

Note: Creosote and creosote treatment: "Standards for Creosoted-Wood Foundation Piles," C1-C12, American Wood-Preservers Institute (1977-1979)
Concrete: ACI Manual of Concrete Practice
Timber: ASTM Annual Book of Standards, Vol 04.09, D 2899, D 3200
Steel: ASTM Annual Book of Standards, Vol 01.01, Vol 01.04, A 252

heave of expansive soil. Shafts subject to pullout loads or uplift thrust must have sufficient reinforcement steel to absorb the tension load in the shaft and sufficient skin friction and underream resistance to prevent shaft uplift movements.

(b) The shaft may pass through relatively soft, compressible deposits and develop vertical load capacity from end bearing on hard or dense granular soil (Fig. 1-7b) or rock (Fig. 1-7c). End-bearing capacity should be sufficient to support vertical loads supplied by the structure as well as any downdrag forces on the shaft perimeter caused by negative skin friction from consolidating soil (Fig. 1-7b).

(c) Single drilled shafts may be constructed with large diameters, typically 10 feet or more, and can extend to depths of 200 feet or more. Drilled shafts can be made to support large loads and are seldom constructed in closely spaced groups.

(d) Drilled shafts tend to be preferred compared with driven piles as the soil becomes harder. Pile driving becomes difficult in these cases, and the driving vibration can adversely affect nearby structures. Also, many onshore areas have noise control ordinances which prohibit 24-hour pile driving (a cost impact).

(e) Good information on rock is required when drilled shafts are supported by rock. Drilled shafts placed in weathered rock or that show lesser capacity than expected may require shaft bases to be placed deeper than anticipated. This may cause significant cost overruns.

d. Location and topography. Location and topo-graphy strongly influence selection of the foundation. Local practice is usually an excellent guide. Driven piles are often undesirable in congested urban locations because of noise, inadequate clearance for pile driving, and the potential for damage caused by vibration, soil densification, and ground heave. Prefabricated piles may also be undesirable if storage space is not available. Other variables may restrict the utilization of deep foundation:

(1) Access roads with limited bridge capacity and head room may restrict certain piles and certain construction equipment.

(2) The cost of transporting construction equip-ment to the site may be significant for small, isolated structures and may justify piles that can be installed using light, locally available equipment.

e. Economy.

(1) Driven piles. Costs will depend on driving rig rental, local labor rates, fuel, tools, supplies, cost and freight of pile materials, driving resistance, handling, cutoffs, caps, splicing, and jetting. Jetting is the injection of water under pressure, usually from jets located on opposite sides of the pile, to preexcavate a hole and to assist pile penetration. Costs are also influenced by downtime for maintenance and repairs, insurance, overhead, and profit margin. An economic study should be made to determine the cost/capacity ratio of the various types of piles. Consideration should be given to including alternative designs in contract documents where practical.

(2) Drilled shafts. Drilled shafts are usually cost effective in soil above the water table and installation in cohesive soil, dense sand, rock, or other bearing soil overlaid by cohesive soil that will not cave when the hole is bored. Drilled shafts, particularly auger-placed, pressure-grouted shafts, are often most economical if the hole can be bored without slurry or casing.

*f. Length.*The length of the deep foundation is generally dependent on topography and soil conditions of the site.

(1) Driven piles. Pile length is controlled by soil conditions and location of a suitable bearing stratum, availability and suitability of driving equipment, total pile weight, and cost. Piles exceeding 300 feet have been installed offshore. Piles up to 150 feet are technically and economically acceptable for onshore installation.

(2) Drilled shafts. Shaft length depends on the depth to a suitable bearing stratum. This length is limited by the capability of the drilling equipment and the ability to keep the hole open for placement of the reinforcement steel cage and concrete.

8. Site and Soil Investigations

The foundation selected depends on functional requirements of the structure and results of the site investigation. Site investigation is required to complete foundation selection and design and to select the most efficient construction method. The first phase of the investigation is examination of site conditions that can influence foundation performance and construction methodology. The seond phase is to evaluate characteristics of the soil profile to determine the design and the construction method. These phases are accomplished bythe following:

a. Feasibility study. A reconnaissance study should be performed to determine the requiriements of a deep

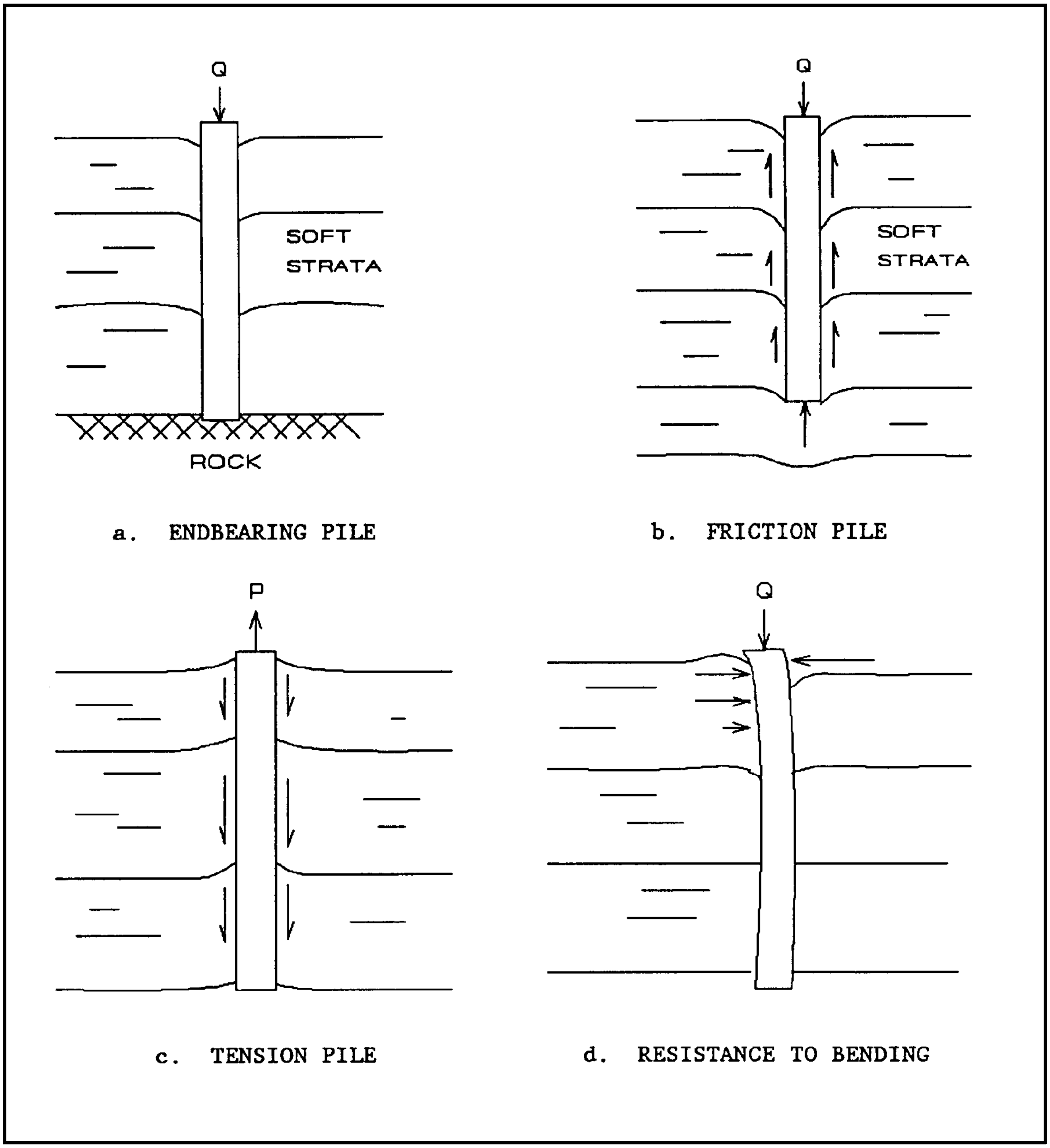
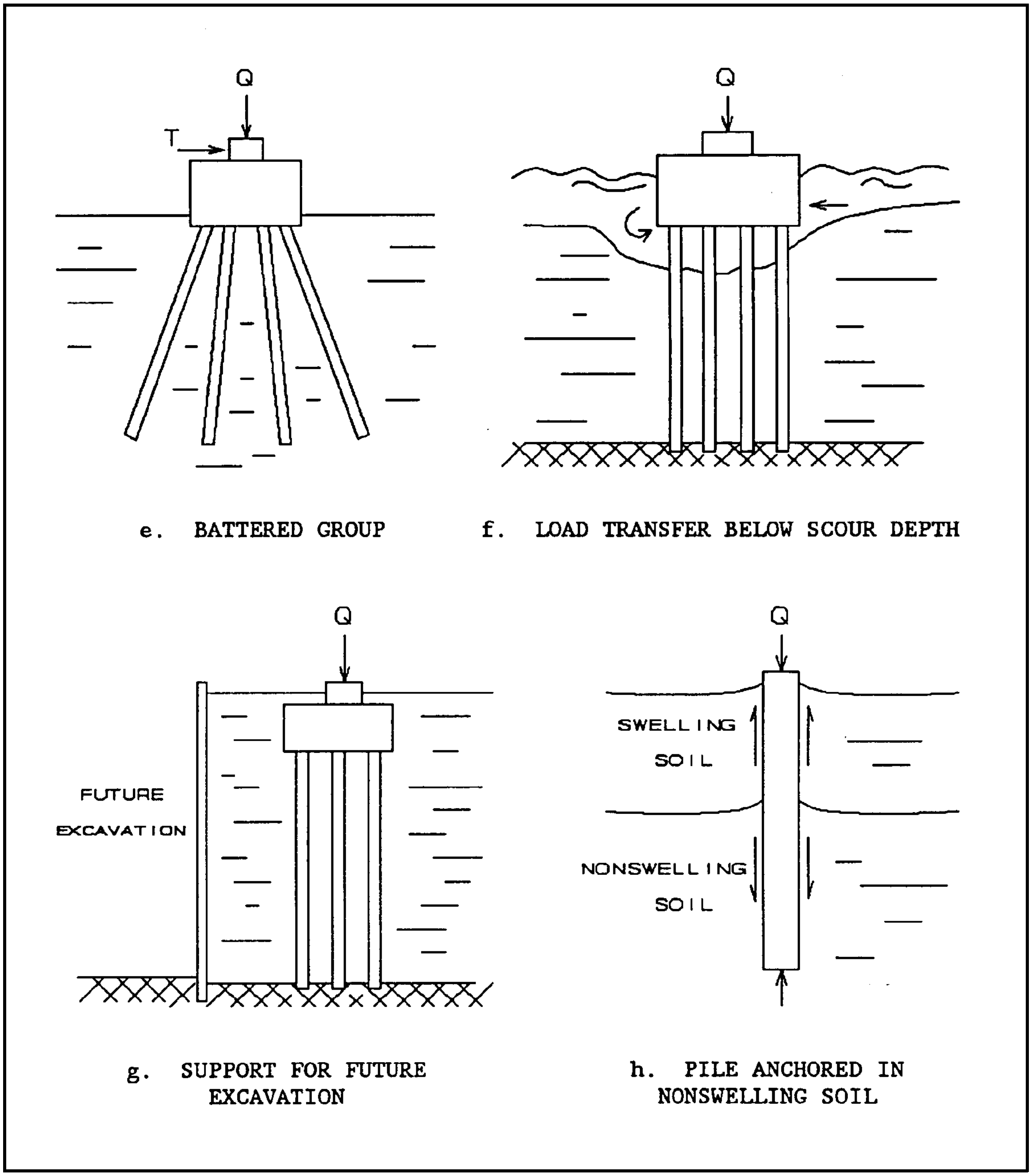


Figure 1-6. Driven pile applications (Continued)



e. BATTERED GROUP

f. LOAD TRANSFER BELOW SCOUR DEPTH

g. SUPPORT FOR FUTURE EXCAVATION

h. PILE ANCHORED IN NONSWELLING SOIL

Figure 1-6. (Concluded)

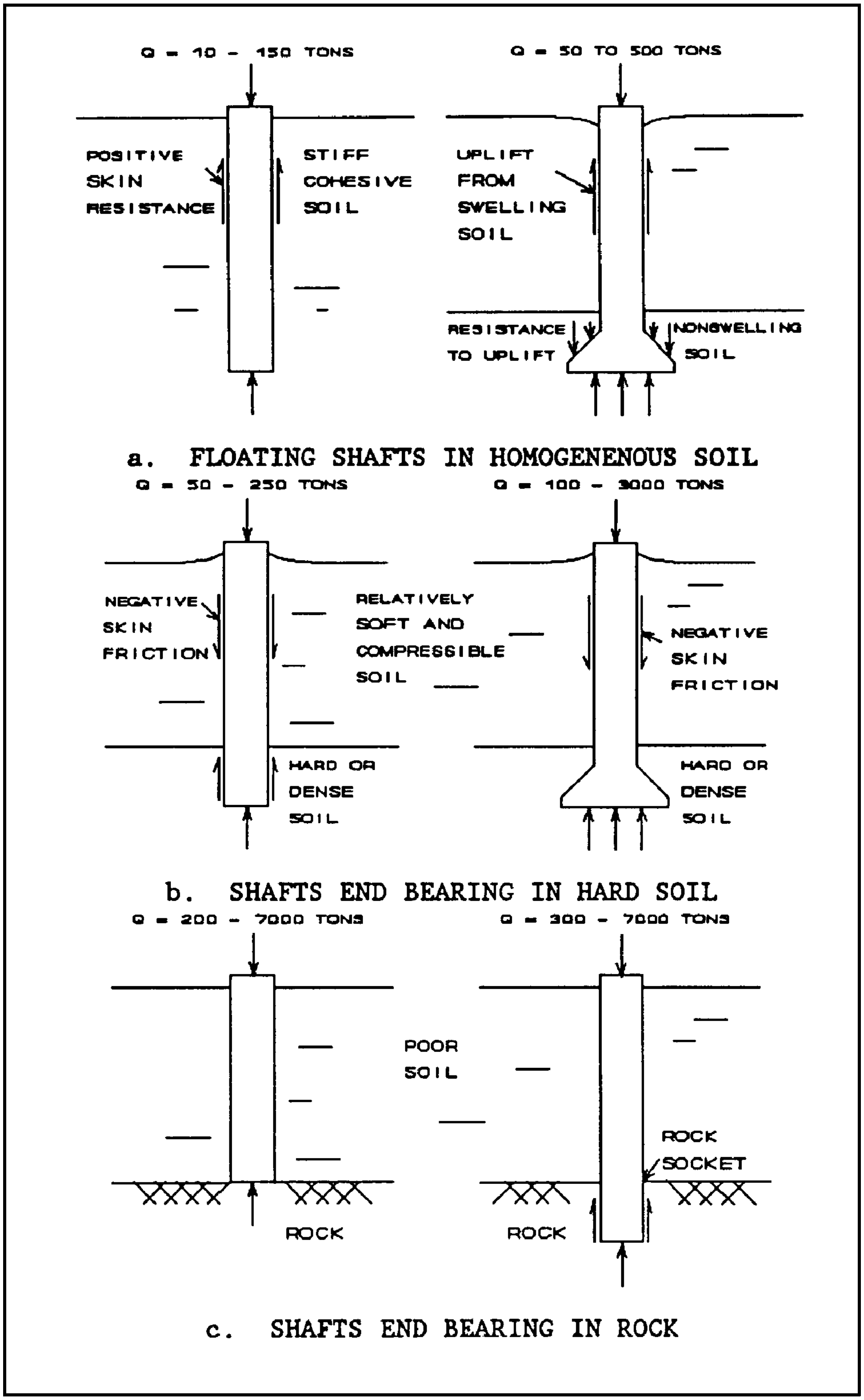


Figure 1-7. Load resistance of drilled shafts in various soils

Table 1-5
Drilled Shaft Applications, Advantages, and Disadvantages

Applications
Support of high column loads with shaft tips socketed in hard bedrock.
Support of moderate column loads with underreams seated on dense sand and gravel.
Support of light structures on friction shafts in firm, nonexpansive, cohesive soil.
Support of slopes with stability problems.
Resists uplift thrust from heave of expansive soil, downdrag forces from settling soil, and pullout forces.
Provides anchorage to lateral overturning forces.
Rigid limitations on allowable structural deformations.
Significant lateral variations in soils.
Advantages
Personnel, equipment, and materials for construction usually readily available; rapid construction due to mobile equipment; noise level of equipment less than some other construction methods; low headroom needed; shafts not affected by handling or driving stresses.
Excavation possible for a wide variety of soil conditions; boring tools can break obstructions that prevent penetration of driven piles; excavated soil examined to check against design assumption; careful inspection of excavated hole usually possible.
In situ bearing tests may be made in large-diameter boreholes; small-diameter penetration tests may be made in small boreholes.
Supports high overturning moment and lateral loads when socketed into rock.
Avoids high driving difficulties associated with pile driving.
Provides lateral support for slopes with stability problems.
Heave and settlement are negligible for properly designed drilled shafts.
Soil disturbance, consolidation, and heave due to remolding are minimal compared with pile driving.
Single shafts can carry large loads; underreams may be made in favorable soil to increase end-bearing capacity and resistance to uplift thrust or pullout forces.
Changes in geometry (diameter, penetration, underream) can be made during construction if required by soil conditions.
Pile caps unnecessary.
Disadvantages
Inadequate knowledge of design methods and construction problems may lead to improper design; reasonable estimates of performance require adequate construction control.
Careful design and construction required to avoid defective shafts; careful inspection necessary during inspection of concrete after placement difficult.

Table 1-5 (Concluded)

Disadvantages (Concluded)
Construction techniques sometimes sensitive to subsurface conditions; susceptible to “necking” in squeezing ground; caving or loss of ground in fissured or cohesionless soil.
Construction may be more difficult below groundwater level; concrete placement below slurry requires careful placement using tremie or pumping artesian water pressure can require weighting additives to drilling fluids to maintain stability; extraction of casing is sensitive to concrete workability, rebar cage placement must be done in a careful, controlled manner to avoid problems; underreams generally should be avoided below groundwater unless “watertight” formation is utilized for construction of underreams.
End-bearing capacity on cohesionless soil often low from disturbance using conventional drilling techniques.
Enlarged bases cannot be formed in cohesionless soil.
Heave beneath base of shaft may aggravate soil movement beneath slab-on-grade.
Failures difficult and expensive to correct.

foundation designs, and the scope of in situ soil and foundation load tests. Required cost estimates and schedules to conduct the soil investigation, load tests, and construction should be prepared and updated as the project progresses.

b. Site conditions. Examination of the site includes history, geology, visual inspection of the site and adjacent area, and local design and construction experience. Maps may provide data on wooded areas, ponds, streams, depressions, and evidence of earlier construction that can influence soil moisture and groundwater level. Existence of former solid waste disposal sites within the construction area should be checked. Some forms of solid waste, i.e., old car bodies and boulders, make installation of deep foundations difficult or result in unacceptable lateral deviation of driven piles. Guidance on determining potential problems of deep foundations in expansive clay is given in TM 5-818-7, “Foundations in Expansive Soils.” Special attention should be payed to the following aspects of site investigation:

(1) Visual study. A visual reconnaissance should check for desiccation cracks and nature of the surface soil. Structural damage in nearby structures which may have resulted from excessive settlement of compressible soil or heave of expansive soil should be recorded. The visual study should also determine ways to provide proper drainage of the site and allow the performance of earthwork that may be required for construction.

(2) Accessibility. Accessibility to the site and equipment mobility also influence selection of construction methods. Some of these restrictions are on access, location of utility lines and paved roads, location of obstructing structures and trees, and topographic and trafficability features of the site.

(3) Local experience. The use of local design and construction experience can avoid potential problems with certain types of foundations and can provide data on successfully constructed foundations. Prior experience with and applications of deep foundations in the same general area should be determined. Local building codes should be consulted, and successful experience with recent innovations should be investigated.

(4) Potential problems with driven piles. The site investigation should consider sensitivity of existing structures and utilities to ground movement caused by ground vibration and surface heave of driven piles. The condition of existing structures prior to construction should be documented with sketches and photographs.

c. Soil investigation. A detailed study of the subsurface soil should be made as outlined in TM 5-818-1. The scope of this investigation depends on the nature and complexity of the soil, and size, functional intent, and cost of the structure. Results of the soil investigation are used to select the appropriate soil parameters for design as applied in Chapters 2 through 5. These parameters are frequently the consolidated-drained friction angle N for cohesionless soil, undrained shear strength C_u for cohesive soil, soil elastic modulus E_s for undrained loading, soil dry unit weight, and the groundwater table elevation. Refer to TM 5-818-1 for guidance on evaluating these parameters.Consolidation and potential heave characteristics may also be required for clay soils and the needed parameters may be evaluated following procedures presented in TM 5-818-7. Other tests associated with soil investigation are:

(1) In situ tests. The standard penetration test (SPT) according to ASTM D 1586 and the cone penetration test (CPT) according to ASTM D 3441 may be performed to estimate strength parameters from guidance in TM 5-818-1.

(2) Soil sampling. Most soil data are obtained from results of laboratory tests on specimens from disturbed and relatively undisturbed samples. Visual classification of soil is necessary to roughly locate the different soil strata as a function of depth and lateral variation.

(3) Location and sampling depth. Borings should be spaced to define the lateral geology and soil nonconformities. It may be sufficient to limit exploration to a depth that includes weathered and fissured material, to bedrock, or to depths influenced by construction. For individual drilled shafts, depths of at least five tip diameters beneath the tip of the deepest element of end-bearing foundations should be investigated. For driven pile groups, a much deeper investigation is appropriate and should extend a minimum of 20 feet or two pile group widths beneath the tip of the longest anticipated pile, or to bedrock, whichever is less. These depths are the minimum required to provide sufficient data for settlement analysis. The potential for settlement should be checked to ensure compliance with design specifications.

(4) Selection of soil parameters. Results of laboratory and in situ tests should be plotted as a function of depth to determine the characteristics of the subsurface soils. Typical plots include the friction angle ϕ for sands, undrained shear strength C_u for clays, and the elastic modulus E_s. These data should be grouped depending on the geological interpretation of the subsoil of similar types. Each soil type may be given representative values of strength, stiffness, and consolidation or swell indexes for estimating soil settlement or heave. Soil strength parameter could be estimated from established correlations from laboratory testing.

(a) Classification. Soil classification characteristics should be applied to estimate soil strength and other parameters from guidance in TM 5-818-1. Data such as gradation from sieve analysis, Atterberg limits, water content, and specific gravity should be determined from tests on disturbed specimens. Refer to ASTM D 2487 for soil classification procedures.

(b) Strength. Soil strength parameters are required to evaluate vertical and lateral load capacity. The strength of cohesive soil may be determined from triaxial test results performed on undisturbed soil specimens at confining pressures equal to the in situ total vertical overburden pressure σ_v. The unconsolidated undrained Q test will determine the undrained shear strength (cohesion) C_u of cohesive soils. The effective friction angle ϕ' and cohesion of overconsolidated soils may be determined from results of R tests with pore pressure measurements using a confining pressure similar to the effective overburden pressure ϕ'_v. However, analyses are usually performed assuming either cohesive or cohesionless soil. Mean strength values within the zone of potential failure may be selected for pile capacity analysis. Refer to TM 5-818-1 and NAVFAC DM-7.1, "Soil Mechanics," for further details.

(c) Elastic modulus. Young's Elastic modulus E_s is required for evaluation of vertical displacements of the deep foundation. The E_s may be estimated as the initial slope from the stress-strain curves of strength test results performed on undisturbed soil specimens. The E_s for clay may be estimated from the undrained shear strength C_u, the overconsolidation ratio, and the plasticity index (PI) shown in Figure 1-8. The E_s typically varies from 100 to 400 kips per square foot (ksf) for soft clay, 1,000 to 2,000 ksf for stiff clay, 200 to 500 ksf for loose sand, and 500 to 1,000 ksf for dense sand.

The E_s may also be estimated from results of

$$E_s = K_{cu} C_u \tag{1-3a}$$

where

E_s = undrained elastic modulus

K_{cu} = factor relating E_s with C_u

C_u = undrained shear strength

The E_s may also be estimated from results of static cone penetration tests (Canadian Geotechnical Society 1985) as:

$$E_s = C_1 q_c \tag{1-3b}$$

where

C_1 = constant depending on the relative compactness of cohesionless soil; i.e., C_1 = 1.5 for silts and sands, 2.0 for compact sand, 3.0 for dense sands, and 4.0 for sand and gravel

q_c = Static cone penetration resistance expressed in similar units of E_s

The E_s for cohesive soil may be estimated from the preconsolidation pressure (Canadian Geotechnical Society 1985) as:

$$E_s = C_2 P_c \tag{1-3c}$$

where

C_2 = constant depending on the relative consistency of cohesive soils; i.e., 40 for soft, 60 for firm, and 80 for stiff clays

P_c = preconsolidation pressure, measured in similar units of E_s

(d) Lateral modulus of subgrade reaction. The modulus of horizontal subgrade reaction E_{ls} is required for evaluation of lateral displacements

$$E_{ls} = \frac{p}{y} \tag{1-4}$$

where

p = lateral soil reaction at a point on the pile per unit length, kips/ft

y = lateral displacement, ft

The E_{ls} is approximately $67C_u$ for cohesive soil (Davisson 1970), and for granular or normally consolidated clays is

$$E_{ls} = K_s Z \tag{1-5}$$

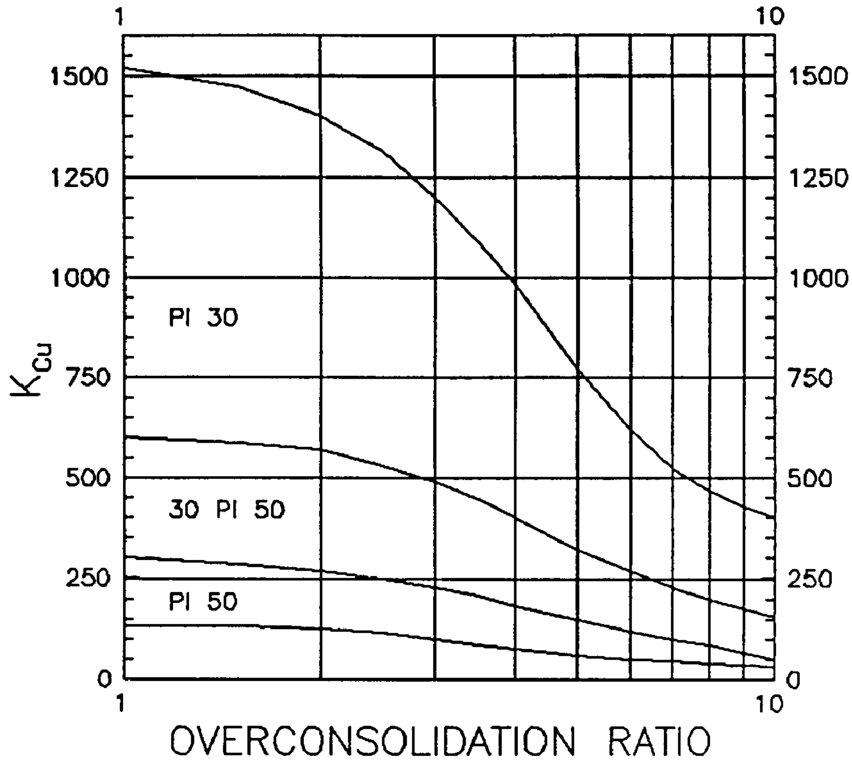
where

k_s = constant relating E_{ls} with depth z, ksf/ft

z = depth, ft

The value of k_s is recommended to be about 40, 150, and 390 ksf/ft for loose, medium, and dense dry or moist sands, respectively, and 35, 100, and 210 ksf/ft for submerged sands after FHWA-RD-85-106, "Behavior of Piles and Pile Groups Under Lateral Load." The value of k_s is also recommended to be about 500, 1,700, and 5,000 ksf/ft for stiff clays with average undrained shear strength of 1 to 2, 2 to 4, and 4 to 8 ksf, respectively. Refer to Chapter 4 for further information on E_{ls}.

(e) Consolidation. Consolidation tests using ASTM D 2435 or swell tests using ASTM D 4546 may be performed to determine preconsolidation pressure, compression and swell indexes, and swell pressures for estimating settlement and downdrag of consolidating soil and uplift forces and heave of expansive soil. Average parameters may be selected for analysis of deep foundations in consolidating or expansive soil by using the computer program Axial Load Transfer (AXILTR) (Appendix C). Refer to TM 5-818-7 for further details on the analysis of the potential heave of expansive soil.



E_s = $K_{Cu}C_u$

E_s = UNDRAINED ELASTIC MODULUS

K_{Cu} = FACTOR RELATING E_s WITH C_u

C_u = UNDRAINED SHEAR STRENGTH

Figure 1-8. Variation K_{cu} for clay with respect to undrained shear strength and overconsolidation ratio