

I/10

Steel Piling

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I. Introduction

A. General

Piling is used in highway bridge construction when footings cannot economically be founded either directly on rock or soil of adequate bearing capacity. Piling transfers foundation loads to underlying strata that are able to support the loads. In certain cases scour and erosion make it necessary to penetrate otherwise adequate soils to reach deeper materials not subject to erosion or scour.

The load which can be carried safely by a pile depends on three principal factors:

- (1) Capacity of the supporting soil;
- (2) Capacity of the pile to transfer its load to the soil; and
- (3) Strength of the pile as a structural member.

Design of foundations must start with an adequate study of subsurface soil conditions at the site. It is important that this study include an investigation of soil and rock strata to a sufficient depth below the point of proposed piles to determine whether there are any weak layers that could affect the future stability of the foundation. A sufficient number of test borings should be made to identify the different soil strata at the site being considered. This information is imperative in order to select the type of pile and method of installation.

B. Classification of Support

Piles may be divided into two general classes based on the principal manner in which they develop support:

(1) End-bearing piles driven into hard material such as rock, shale, cemented-sand and gravel, dense sand or gravel and very stiff clays.

(2) Friction piles deriving their support principally from the strength of the soil surrounding the pile through the development of shearing resistance between soil and the pile.

Most support conditions combine end bearing and friction—even on relatively short end-bearing piles the full pile load is rarely transmitted to the tip of the pile.

Generally, foundation engineers permit higher loads per pile where piles are classified as the end-bearing type.

For this reason it may be advisable to consider the economic advantages of extending the piles to end-bearing material permitting higher loads rather than to use shorter piles with lighter load capacity.

II. Characteristics of Steel Piles

A. Advantages

Millions of lineal feet of steel piles have been used in highway bridge construction for the following reasons:

- Steel piles can penetrate to bed rock or firm strata where other kinds of piles could not penetrate or would be destroyed in driving. Steel H-piles are particularly advantageous for penetrating fills to reach natural formations.

- Steel piles are particularly useful where great depths of unstable material must be penetrated before reaching the desired load-carrying strata.

- Steel H-piles or open-end steel pipe piles can be driven where piles are closely spaced to carry heavy loads, with much smaller resultant displacement or disruption of the soil than is possible with any other type of pile.

- Steel piles perform extremely well where piles must act both as foundation and as columns of pile bents.

- They possess high resistance to horizontal loads above the ground surface—important when supporting highway or railroad bridges.

- Steel piles are excellent for very high unit loads per pile.

- Steel piles are not subject to attack and destruction by organisms such as limnoria, teredo, and termites.

- Steel piles are easier to handle and drive, and can be driven with large hammers unsuitable for other materials.
- Steel piles are easy to splice.
- Steel piles are easily cut to the proper length after driving.
- The low cost of material, the speed and ease of installation and footing design economies make steel piles an economically sound foundation medium.

B. Steel H-Piles

Steel H-piles are wide-flange sections with flanges and web of equal thickness and are available in sizes from 8" x 8" to 14" x 14". Other types of wide-flange sections may also be used as H-piles.

Steel H-piles are available in A36 steel, in high strength steels with yield points up to 60,000 psi, and in special chemistry steel for applications where salt water splash corrosion must be resisted. Significant mechanical properties of the steels used for piles are shown in Figure 1.

HP STEEL H-PILES			
Mechanical Properties			
Steel Designation And USS Brand	Yield Point (f_y) Min. psi	Tensile Strength Min. psi	Elong. in 8" Min.
A36	36,000	58/80,000	20%
A572 GR 42 (EX-TEN 42)	42,000	60,000	20%
A572 GR 45 (EX-TEN 45)	45,000	60,000	19%
A572 GR 50 (EX-TEN 50)	50,000	65,000	18%
A572 GR 55 (EX-TEN 55)	55,000	70,000	17%
A572 GR 60* (EX-TEN 60)	60,000	75,000	16%
A588 GR A (COR-TEN B)†	50,000	70,000	18%
USS MARINER**	50,000	70,000	18%
<p>*HP 14 x 117 not available in A572 Grade 60</p> <p>**USS MARINER has superior corrosion resistance in the splash zone for marine application.</p> <p>†COR-TEN B steel has 2 times the atmospheric corrosion resistance of carbon structural steel with copper.</p> <p>Bearing Capacity—The American Association of State Highway Officials (AASHTO) Standard Specifications permits the point-bearing capacity of steel H-piles and open-end pipe piles to be determined by loading test piles. Where it is not feasible to make the required test loads, the maximum <i>assumed</i> design load for steel point-bearing piles shall be 9,000 psi over the cross sectional area of the pile tip, not including the area of any pile tip reinforcement.</p>			
O STEEL PIPE PILES			
Mechanical Properties			
Steel Designation	Yield Point Min. psi (f_y)	Tensile Strength Min. psi	Elong. in 2" Min.
A252 Grade 2	35,000	60,000	25%

FIGURE 1—SIGNIFICANT MECHANICAL PROPERTIES OF STEEL H-PILES AND PIPE PILES

Steel H-piles have proven capable of satisfying a multitude of design requirements and can be depended on to solve many foundation problems in an efficient manner. They may be utilized either as friction or as end-bearing piles and are suitable for a wide range of loads.

Steel H-piles have been repeatedly driven to refusal in hard rock. In bridge foundations, heavy flange sections have been driven to design loads of 200 tons per pile and test loaded to 400 tons or more. Because of their ability to withstand severe driving, steel H-piles can penetrate strata containing hard lenses, boulders or other obstructions. They have been driven over 200 feet through difficult upper strata to reach deep rock strata. These piles are easy to cut and splice. This has made them especially valuable for driving in fissured limestone and in other strata requiring varying pile lengths.

Steel H-piles are unique in their ability to penetrate compacted sand or sand and gravel which offer great resistance to the driving of other piles. Lateral displacement of the compact materials is held to a minimum with consequent economy in driving costs. Adequate bearing capacity is generally attained with moderate penetration.

When driven into stiff clays, steel H-piles often trap soil between flange and web and compact it so that it becomes a hard core carried down with the pile. This core aids in compressing the surrounding soil and building up resistance to further displacement. Although some point resistance is developed in stiff clays, the principal transfer of stress is through frictional forces—adhesion between the clay and the pile and cohesion in the soil itself. Usually the load-carrying capacity of the pile is proportional to its embedded length.

In soft to medium clays and silts, steel H-piles developed skin friction resistance to penetration under loads dependent upon the embedded surface area of the pile and the shearing strength of the soil. A pile driven into soft clays tends to disturb the clay around the pile resulting in an immediate loss of shear strength of the clay. After driving a pile, the clay begins to consolidate around the pile and recovers most of its original shear strength within a month. For this reason piles driven into soft clays should not be load tested for several weeks to a month after driving depending upon the soil characteristics. Point bearing is negligible in these soils.

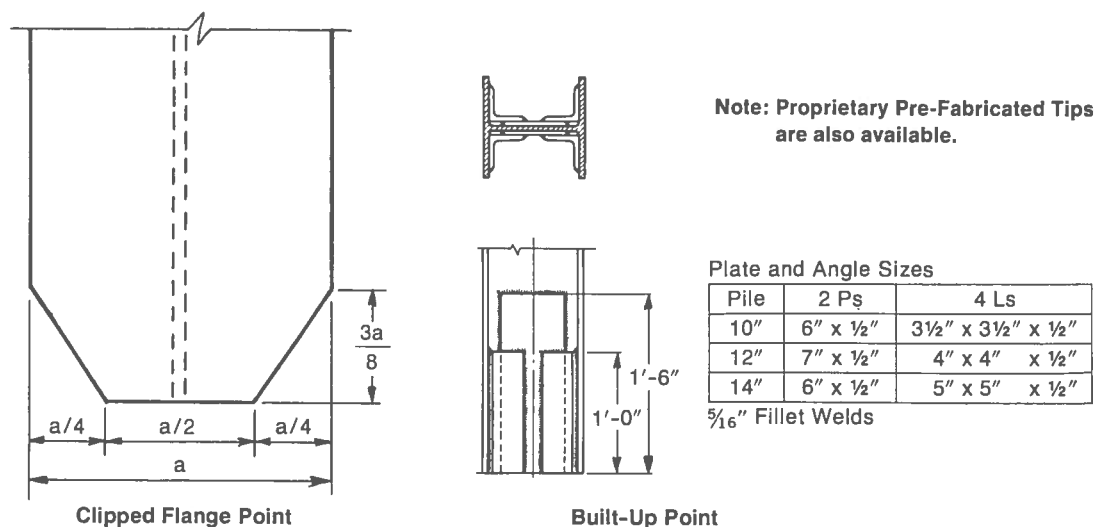


FIGURE 2—TYPICAL H-PILE POINT DETAILS

C. Steel H-Pile Points

The tips of the steel H-piles are usually left square and blunt as received from the mill. Occasionally, pile tips are reinforced if hard driving conditions are expected. Typical tip details are shown in Figure 2. Proprietary tips are also available. An enlarged point, a thick square plate welded across the end of the pile, is useful for developing bearing on or in compacted sand, gravel or hardpan underlying soft clays,

but this type of end may destroy frictional contact between the pile and soil for an appreciable distance above the tip. Thus, H-pile points should not be used on friction piles as the carrying capacity may be lowered by the point.

Caps are not usually required for steel H-piles embedded in concrete. A series of comprehensive tests conducted in 1947 by the State of Ohio, Department of Highways gave evidence that the strength of the connection between properly sized and reinforced concrete caps and steel H-piles was adequate without cap plates. Additional tests by The American Iron and Steel Institute at Lehigh University in 1971-72 verify this conclusion. Steel H-piles without caps embedded only 6 inches into the concrete proved as effective as those with cap plates.

The AASHTO Specification recognizes these tests and indicates that in general cap plates are not required.

D. Steel Pipe Piles

Two types of steel pipe piles are used: Piles driven open end and piles driven with pile driving points (also called closed-end piles). USS Pipe Piling meets the specification for welded and seamless steel pipe piles set forth in ASTM Specification A252. Significant mechanical properties are shown in Figure 1.

E. Open-end Pipe Piles

Open-end piles are used when soil investigations disclose the existence of rock formations at a level reasonably close to the surface, particularly where heavy loads must be supported. This type of pile is normally used to a depth of 40 to 50 feet, although in some instances such piles have been driven to depths of more than 150 feet. After driving, open-end piles are usually blown out by air or water and filled with concrete, although they may also be used as driven with the design load based on the area of steel.

In designing open-end pile foundations, the load bearing capacity of such piles is usually calculated on the basis of the steel and concrete cross sectional area at the point of bearing, without regard for the friction between the surface of the pile and the penetrated material.

F. Closed-end Pipe Piles

Closed-end piles are called for when soil investigations indicate the absence of rock formation, or when it is not practical to drive piles to rock. These piles are usually driven to a desired resistance which has been determined by load tests, theoretical analyses or various currently used pile-driving formulas.

In some cases closed-end rather than open-end piles are used when driving to rock. This procedure avoids the blowing out of confined material and the occasional dewatering necessary when driving open-end piles. Driving to refusal is most commonly specified and, in such driving, the ends of the piles indent or socket themselves into the rock, assuring a positive bearing.

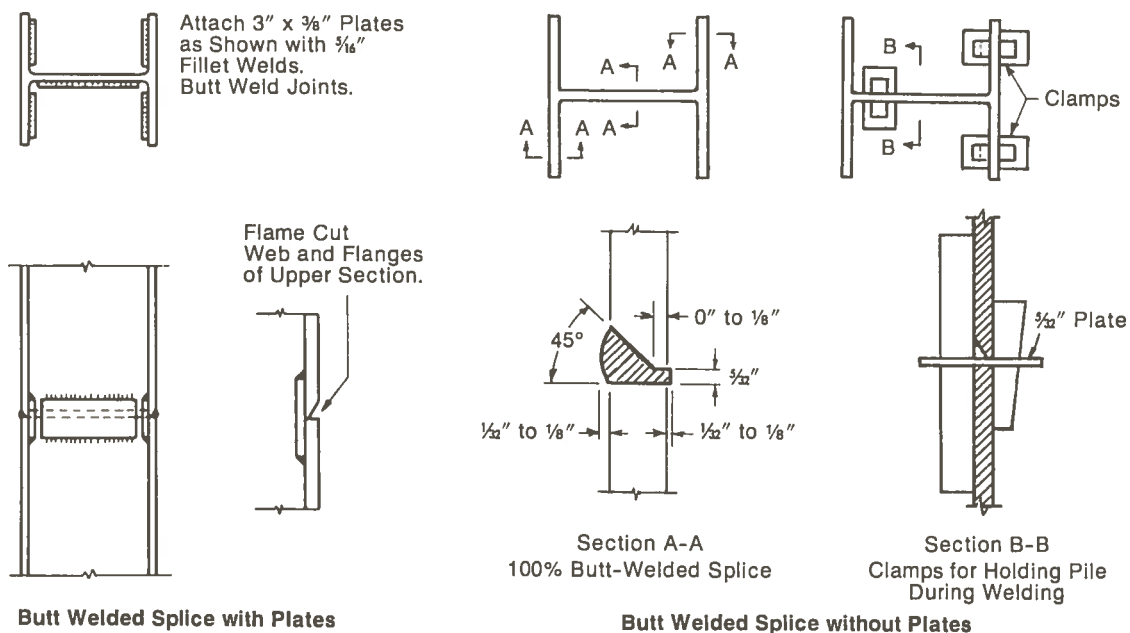
G. Closed-end Pipe Pile Points

The tips of pipe piles usually are closed with steel plates with diameters about $\frac{1}{2}$ inch greater than the diameter of the pile. The added $\frac{1}{2}$ inch allows for a fillet weld attaching the plate to the pile. Proprietary pointed tips also are available.

H. Steel Pile Splices

Splices should be designed and constructed to maintain the true alignment and position of the pile sections. Splices should develop the full strength of the pile in both bearing and bending.

Typical butt-welded splice details for H-piles are shown in Figure 3. Either simple positioning plates welded to flanges and web can be used as back-up plates for welding, or the ends of H-piles can be clamped in position while welding without back-up plate. External collars fillet welded to both the lower and upper pile sections are occasionally used. In addition proprietary splices are available for both H and pipe piles that position the upper pile section before welding.



Note: Proprietary Pre-Fabricated Splices
are also available.

FIGURE 3—TYPICAL WELDED SPLICE DETAIL FOR H-PILES

I. Drilled-in Caisson

The drilled-in caisson is another steel pipe application. Here the pipe is fitted with a hardened steel cutting edge and driven open end to a positive seat on sound rock. A rock socket is then drilled to a predetermined depth below the tip of the pile. After thorough cleaning and dewatering (if necessary), a steel H-beam core may be placed in the pipe shell to the bottom of the rock socket and the socket and pipe are filled with concrete. This combination of steel pipe, steel H-beam, and concrete can be used to carry loads of more than 1000 tons per caisson.

III. Subsurface Investigation

A. General

The subsurface investigation for a pile foundation should determine the ability of the bearing strata to support imposed loadings from the viewpoint of both strength and settlement. Borings throughout the area are required to determine general sub-soil profiles, and to establish the properties of the soil.

B. Soil Properties

Most available pile design concepts require the classification of soils as either fine-grained or coarse-grained. This is most appropriately accomplished in accordance with the Unified Soil Classification System presented on Figure 4. In this system the fine-grained category encompasses all predominantly clayey and silty soils. These soils exhibit the general characteristics of homogeneous appearance, low permeability, potentially high compressibility, deformability without rupture (plasticity), moderate tensile strength, and resistance to shear stress on planes with no normal stress, termed cohesive shear resistance. The shear strength of fine-grained soils is approximately equal to the cohesion c , and to one-half the unconfined compressive strength, thus:

$$s = c = \frac{1}{2}q_u$$

The coarse-grained soil category may be seen from Figure 4 to primarily comprise

sand, gravel and rock fragments. These soils, and mixtures thereof, exhibit the general characteristics of grainy appearance, high permeability, low compressibility, rupture deformation (non-plastic), no tensile strength, and no significant resistance to shear stresses on planes without normal stresses. In practical situations the shear strength, s , of coarse-grained soils may be assumed to be directly proportional to the intergranular or effective normal stress, σ' in the soil, thus:

$$s = \sigma' \tan \phi$$

where ϕ is termed the angle of internal friction. The value of ϕ varies directly with the soil's density, and the common designations of coarse-grained soils as loose, medium, or dense may be interpreted as implying values of ϕ of less than 30° , 30° to 36° and greater than 36° , respectively.

Fine-grained soils of little or no plasticity, ML in the Unified System, may act as a cohesionless coarse-grained soil with low ϕ value or as a cohesive soil with relatively low shear strength. In general, ML materials tend to perform poorly as a pile-bearing stratum.

When a soil includes a mixture of both fine and coarse particles, a judgment must be made as to whether it will act as a cohesive or cohesionless material.

Disturbance created by pile installation alters the characteristics of adjacent soils. Fine-grained soils generally undergo a reduction in shear strength, but this is regained to some degree with time. Loose coarse-grained soils are densified as the particles are shaken together during pile installation, thereby eventually yielding a stronger soil, while dense coarse-grained soils become less dense as the constituent soil particles are disturbed, resulting in a weaker soil. Some allowance for this strength variation is inherent in the design procedures presented in this chapter.

The driving of a large number of closely spaced piles in a foundation can produce displacements of piles previously driven or alter the soil strength or stress conditions at piles previously driven. In saturated clays, heave of the ground or rise in pore pressures may occur. In loose sands, settlement of the ground may be expanded over fairly wide areas as pile driving continues. The bearing qualities of stiff clays or clay-shales or dense coarse-grained soils at piles previously driven may be damaged by installation of nearby piles.

C. Preliminary Investigation

The preliminary site investigation should determine the most appropriate type of foundation and provide guidelines for a more detailed investigation. Geologic and soil maps can give an indication of the soil and rock conditions to be encountered at a specific location. In developed areas there is frequently sufficient information from immediately adjacent construction to permit rapid selection of the best foundation type with only a minimum of additional investigation.

Where adjoining construction is absent, or too distant to provide reliable data, the preliminary investigation should include field study. At large sites where it is anticipated that bedrock or some other relatively dense bearing stratum may be present at depths suitable for structural support, the surface of such materials may be approximately determined by seismic methods. In general, however, a few borings are taken to tentatively set the type of footing.

D. Detailed Investigation

In the detailed subsurface investigation, test borings should provide complete descriptions of all soil strata at all footing locations so as to permit drawing accurate soil profiles and to provide sufficient soil samples for laboratory testing when required. Borings should extend not only to the greatest depth likely to be attained by the piles, but also into underlying strata that will be stressed by the piles. Rock coring is required where boulders or bedrock are encountered. To ensure that bedrock has been reached, coring should extend at least 5 feet into sound rock that shows no sign of increased deterioration with depth.

When piles will be end bearing on rock, complete descriptions and samples of the rock core are of major importance. The percentages of rock core recovered should be recorded as an indication of the quality of the rock. In some cases, laboratory tests giving the unconfined compressive strength of the rock will be required.

Standard penetration tests should be made at regular intervals in depth (usually 3 or 5 feet). The standard penetration resistance or "N value" is the number of blows of a 140 pound hammer, falling 30 inches, required to drive a standard 2-inch split-spoon sampler a distance of one foot. Since it is not practical to obtain undisturbed samples of coarse-grained soil, their relative density and angle of internal friction ϕ are related to the standard penetration resistance as shown in Figure 5.^{10*} The values given are more reliable for sand than for gravel.

Compactness	Very Loose	Loose	Medium	Dense	Very Dense	
Relative density	0	15 %	35 %	65 %	85 %	100 %
Standard penetration resistance, N =no. of blows per foot		4	10	30	50	
Angle of internal friction, ϕ (degrees)		28	30	36	41	
Unit weight, pcf						
moist	<100	95-125	110-130	110-140	>130	
submerged	<60	55-65	60-70	65-85	>75	

FIGURE 5—RELATIVE DENSITY AND ANGLE OF INTERNAL FRICTION FOR COARSE-GRAINED SOILS (After Teng¹⁰)

Where practical, undisturbed samples of fine-grained soil should be obtained and unconfined compression tests made in the laboratory. The shear strength is equal to one-half the unconfined compression strength, therefore the shear strength may be approximated by the standard penetration test as shown in Figure 6.¹⁰ The relationship between the unconfined compression strength of cohesive soils and the standard penetration test is rather unreliable and should be used with great caution.

Where friction piles are to be driven in fine-grained material and pile settlement is of concern, undisturbed samples should be obtained for consolidation tests.

Consistency	Very Soft	Soft	Medium	Stiff	Very Stiff	Hard
q_u = unconfined compression strength, pounds per square ft.	0	500	1000	2000	4000	8000
Standard penetration resistance, N = no. of blows per ft.	0	2	4	8	16	32
Unit weight, pcf (saturated)		100-120	110-130	120-140		130+

Shear strength (s) = Cohesion (c) = $\frac{1}{2}$ unconfined compression strength (q_u)

FIGURE 6—SHEAR STRENGTH OF FINE-GRAINED SOILS (After Teng¹⁰)

* See References

E. Conclusions Relative to Pile Construction

Dense coarse-grained soils and sound bedrock provide excellent support for piling. Settlement of piles in such soils occurs concurrently with load application. Fine-grained soils with cohesive strengths above one kip per square foot will generally provide satisfactory side friction support and lateral restraint of piling although pile displacements generally tend to occur over a prolonged period. Settlement of friction piles may be minimized by maintaining shear stresses below about one-half the shear strength of soil. Fine-grained soils with cohesive strengths below one kip per square foot may require evaluation for frictional support or lateral restraint of bearing piles. The presence of such soil strata causes settlements in foundations terminated at lesser depths, and may cause drag or negative skin friction on piles end-bearing in underlying firm soil or rock bearing strata.

In design, attention must be given to problems that may arise in pile installation. The high strength of steel piling permits hard driving with minimum damage, although it may be necessary to precore where large or nested boulders are encountered or to jet where closed-end pipe piles are required to penetrate dense coarse-grained soils. An advantage of opened-end piles is that boulders can be penetrated by drilling within the pipe. The nature and condition of adjoining construction should be given careful consideration in the selection of the pile type and the mode of installation.

IV. Steel Bearing Pile Design

A. General Requirements

The following information is essential to bearing pile design:

1. Structural Requirements—location, magnitude and direction of all applied loadings, details of the pile-structure connection, structural requirements for piles extending above the ground line, and possibly permissible vertical and horizontal displacements of the piles.

2. Test borings and geological profiles showing the boundaries and complete descriptions of all strata including bedrock and ground water level. The exploration program should determine the properties of the pile bearing stratum and of the overburden soils through which the piles extend to reach bearing. As a minimum, results of the standard penetration test (number of blows of a 140 pound hammer, falling 30 inches, to drive a standard 2-inch diameter split-spoon sampler a depth of one foot) are required. On major projects, laboratory tests should provide the pertinent soil properties needed for design.

3. Miscellaneous—nature and condition of site and adjoining construction; contemplated future construction on the site or adjoining property, particularly any construction that would change the ground line or ground water level; and corrosivity of soil and water.

B. Design of a Pile Foundation

Design examples of pile foundations are given in the United States Steel Corporation's "Highway Structures Design Manual," Volume II, Chapter 11.

The design examples in Volume II use procedures and formulas discussed in the following paragraphs.

C. Methods for Establishing Pile Capacity

Three methods commonly are employed to establish the capacity of single piles and pile groups:

1. Theoretical analysis, based upon results of the test boring program.

The axial load applied to any pile is carried partially by skin friction along the length of the pile and partially by the resistance of the soil directly beneath the point, as shown in Figure 7. Expressed in terms of ultimate capacity:

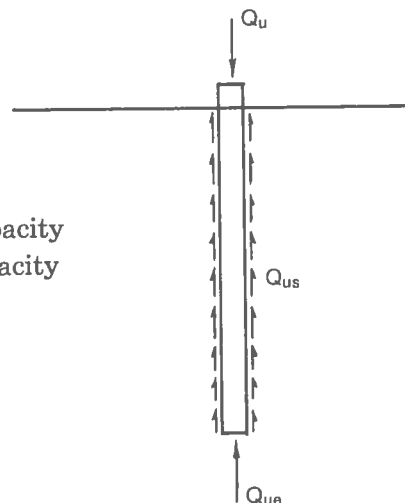
$$Q_u = Q_{us} + Q_{up} \quad (\text{Eq. 1})$$

where Q_u = ultimate load capacity

Q_{us} = skin friction component of ultimate capacity

Q_{ue} = end bearing component of ultimate capacity

FIGURE 7—PILE LOAD IS CARRIED PARTIALLY BY SKIN FRICTION AND PARTIALLY BY END BEARING



Theoretical analyses will be presented to establish the skin friction and end bearing components of ultimate capacity of piles driven in coarse-grained material and fine-grained material.

For piles driven to end bearing on rock, it is normally assumed that all load is carried in end bearing, although some and sometimes most of the load is carried by skin friction, depending on the type and depth of the soil penetrated.

2. Dynamic evaluation of driving performance of single piles employing dynamic driving formulas.

Such formulas yield only crude estimates of pile capacity but are useful in ensuring adequate penetration during construction where their predictions have been correlated with the results of field tests or prior experience in similar conditions.

3. Full-scale field tests—such tests establish reliable capacity values provided the installation procedure and imposed loadings correspond closely to those anticipated in the final construction. Common practice is to load test single piles and attempt to correlate their behavior to anticipated group action by theoretical analysis.

D. Capacity of Single Embedded Piles Under Axial Loading

1. *Rock Bearing*: The allowable loads of H-piles of ASTM A36 grade steel and concrete filled pipe piles of ASTM A252 grade steel driven to end bearing in sound rock are given in Section 1.4.4 (c) of the AASHTO Standard Specifications for Highway Bridges as follows:

- For H-piles, 9000 psi multiplied by the cross sectional area of the pile tip, not including the area of any pile tip reinforcement.
- For concrete filled pipe piles, 0.4 times the 28-day cylinder strength of the concrete (but not more than $0.4 \times 4500 = 1800$ psi) multiplied by the total actual area of the concrete and steel.

Higher allowable loadings may be used if substantiated by fields load tests and may justify consideration of higher strength grades resulting in more economical loadings.

2. *Coarse-Grained Soils—Theoretical Evaluation of Capacity*: The end bearing component of ultimate capacity for piles that penetrate into the coarse-grained material at least 10 times the section depth d of H-piles, or 10 times the diameter d of pipe piles, is given by:¹

$$Q_{ue} = (\frac{1}{2} \gamma_e d N_\gamma + K_b \gamma_e D N_q) A \quad (\text{Eq. 2})$$

where Q_{ue} = end bearing component of ultimate capacity

γ_e = average effective unit weight of soil over total pile length (pcf). (Use the submerged unit weight γ' where below the water table.)

N_γ , N_q = bearing capacity factors, determined from Figure 8.

d = section depth of H-pile or diameter of pipe pile (ft.)

K_b = ratio of horizontal to vertical soil pressure on side of pile within distance of $10b$ from base, determined from Figure 8.^{1,19}

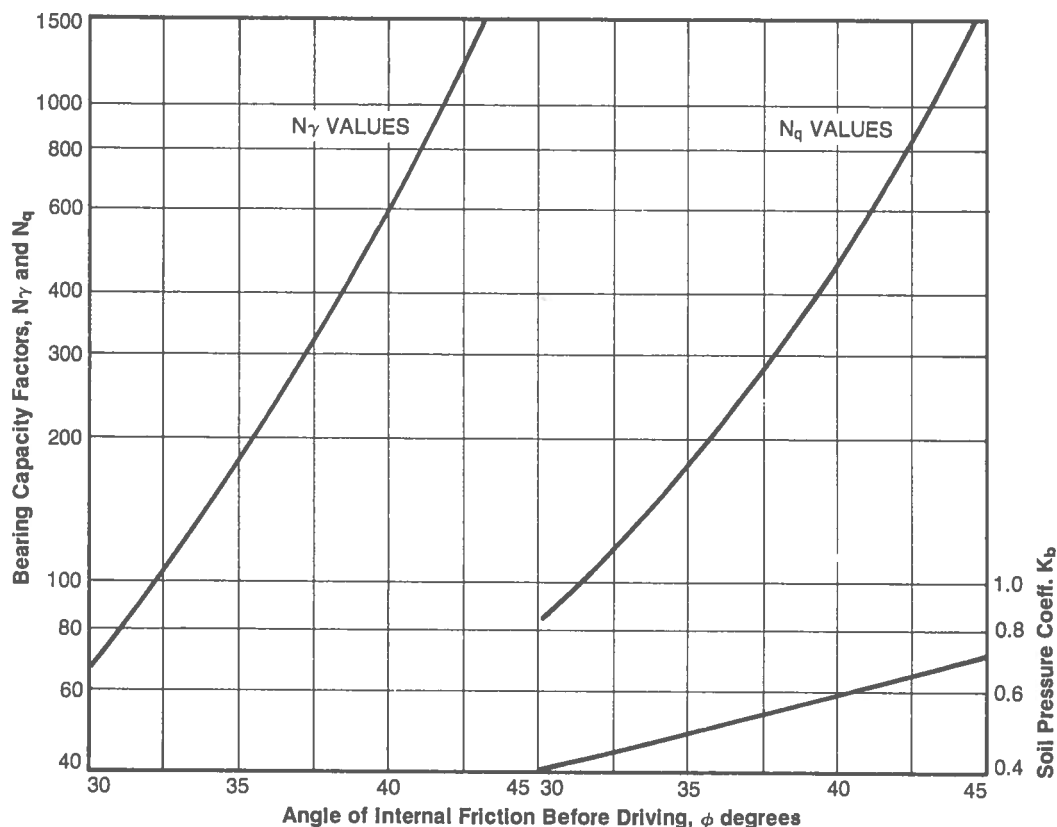


FIGURE 8—BEARING CAPACITY FACTORS FOR COARSE-GRAINED SOIL (after Navdocks⁹)

D = embedment of pile in coarse-grained material

A = product of section depth and width of H-pile, or area of pipe pile (ft.²).

For lesser penetration within coarse-grained material:¹

$$Q_{ue} = \frac{D}{10d} (\frac{1}{2} \gamma_d N_\gamma + K_b \gamma_e D N_q) A \quad (\text{Eq. 3})$$

The skin friction component of ultimate capacity is given by:¹

$$Q_{us} = \frac{1}{2} p K_b \gamma_e D^2 (\tan \delta) \quad (\text{Eq. 4})$$

where Q_{us} = skin friction component of ultimate capacity

p = perimeter; take twice sum of section depth and width for H-pile (ft.)

K_b = ratio of horizontal to vertical earth pressures on side of pile above distance of $10b$ from pile tip. The values of K_b are given on Figure 8.

$\tan \delta$ = coefficient of pile-soil friction. Values may vary from 0.25 to 0.4 depending upon nature of steel surface and soil; assume 0.3 in absence of specific information.

The ultimate capacity $Q_u = Q_{ue} + Q_{us}$

The ultimate pull-out capacity of a single pile is approximately equal to the skin friction component of ultimate capacity as given by Equation 4. However, the allowable uplift per pile should not exceed 0.4 times the allowable compressive load per pile in accordance with Section 1.4.4F of the AASHTO Specifications.

3. *Coarse-Grained Soils—Dynamic Evaluation of Capacity*—the compressive load capacity of driven single piles may be approximately determined from the driving resistance encountered at final penetration employing so-called dynamic formulas.

Many such formulas have been evolved, each of which has proven satisfactory for particular conditions, but none of which may be employed indiscriminately with any assurance of validity.

The basic approach to a dynamic pile-driving formula is to equate the energy applied to the pile by the hammer to the work done on the pile. This may be expressed by the following formula:

$$Q_u \left(\frac{s}{12} \right) = \frac{W_r v^2}{2g} \quad (\text{Eq. 5})$$

where W_r = weight of the hammer ram, in pounds

g = acceleration due to gravity (32.2 ft. per second, per second)

v = velocity of impact in ft. per second

Q_u = ultimate load or bearing capacity of pile, in pounds

s = penetration per blow, in inches

Energy may also be expressed as Wh where W is the weight of the free-falling hammer and h the height of fall. This simple relationship does not hold in practice because it is necessary to apply corrections for amount of energy actually applied to the pile; for energy losses in the pile; and for variations arising from different soil types and conditions. Generally, these variable factors cannot be accurately compensated for to comprehend all pile driving conditions. No pile driving formula gives safe and reliable results when used indiscriminately under all conditions.

The so-called complete driving formula equates the effective energy applied to the pile to five different terms representing useful work and loss for one hammer blow as follows:

$$\begin{array}{ccccccccc} \text{applied} & = & \text{useful} & + & \text{loss in} & + & \text{loss in} & + & \text{loss in} & + & \text{loss in} \\ \text{energy} & & \text{work} & & \text{impact} & & \text{cap} & & \text{pile} & & \text{soil} \end{array}$$

Obviously, this formula is too cumbersome for convenient field use. All other dynamic pile driving formulas may be derived from this complete formula by making various simplifications. Several of the more popular formulas are listed below; however, because of the various imponderables involved they should be used with some caution.

(a) Engineering News formula drop hammers:

$$Q_u = \frac{12W_r h}{s+1} \quad (\text{Eq. 6})$$

(b) Engineering News formula for steam hammers:

$$Q_u = \frac{12W_r h}{s+0.1} \quad (\text{Eq. 6A})$$

(c) Modified Engineering News formula:

$$Q_u = \frac{12W_r}{s+0.1(W_p/W_r)} \quad (\text{Eq. 7})$$

Recent studies (2) have shown the following formula to be perhaps the most generally appropriate at this time. This also represents a modification of the Engineering News formula.

$$(d) \quad Q_u = \frac{12E_n}{s+0.1} \left(\frac{W_r + e^2 W_p}{W_r + W_p} \right) \quad (\text{Eq. 8})$$

The expression in parenthesis should not exceed 0.8.

Good agreement between the computed pile capacities and the actual test failure loads have been found (24) (25) for the Jambu formula.

(e) Jambu formula:

$$Q_u = \frac{12e_f E_n}{K_u s} \quad \text{where} \quad (\text{Eq. 9})$$

$$K_u = C_d \left(1 + \sqrt{1 + \frac{\lambda e}{C_d}} \right) \quad (\text{Eq. 9A})$$

$$C_d = 0.75 + 0.15 \frac{W_p}{W_r} \quad (\text{Eq. 9B})$$

$$\lambda e = \frac{e_f E_n L}{A E s^2} \quad (\text{Eq. 9C})$$

In these formulas:

Q_u = ultimate compressive load capacity.

s = average penetration per blow for the last 6 inches of driving, or for the final inch or less if sudden refusal conditions are encountered (inch).

W_r = weight of hammer ram (pounds).

W_p = weight of pile and driving appurtenances (pounds).

h = hammer fall (feet).

E_n = manufacturer's maximum rated energy (foot-pounds).

e = coefficient of restitution of cushioning material.

Laminated Micarta .65

Steel on Steel .55

Hardwood Blocks .50

Softwood .25

e_f = hammer efficiency.

L = length of pile (inches).

A = cross-sectional area of pile (square inches).

E = modulus of elasticity of pile (pounds per square inch).

A safety factor of six is ordinarily applied to the ultimate load in all the above formulas (except the Jambu formula) to determine the allowable design load. A safety factor of three is applied to the ultimate load for the Jambu formula.

In general, pile driving formulas are useful for comparison purposes. If a group of piles in a specific location drives to the same resistance as the test piles of approximately the same length, it may be assumed that sub-surface soils and conditions are essentially the same, and that design loads obtained from the test loads may be justified for all single piles in the group if point bearing. This assumption, of course, must be substantiated by adequate soil borings. In cases where test loadings are not available, the correlation of driving records with those available from adjacent pile foundations can be extremely valuable to the foundation engineer. Where homogeneous soils prevail in extensive deposits over large areas, previous experience with pile foundations in the same soils will often be enough to satisfy the foundation engineer of the reliability of load values determined from comparative driving records. In such cases, the choice of pile driving formula is determined by the formula used in previous driving records.

4. *Fine-Grained Soils—Theoretical Evaluation of Capacity:* In fine-grained soils the disturbance created by pile driving causes an immediate loss of shear strength although generally this is recovered to some degree with the passage of time.

The capacities to be considered in design are those corresponding to the final equilibrium condition, which may be assumed to prevail about one month after installation. The ultimate capacity is primarily dependent upon restraint along the length of the pile arising from adhesion between pile and soil, or cohesion between soil trapped within the flanges of H-piles and the adjoining soil. Adhesion generally is

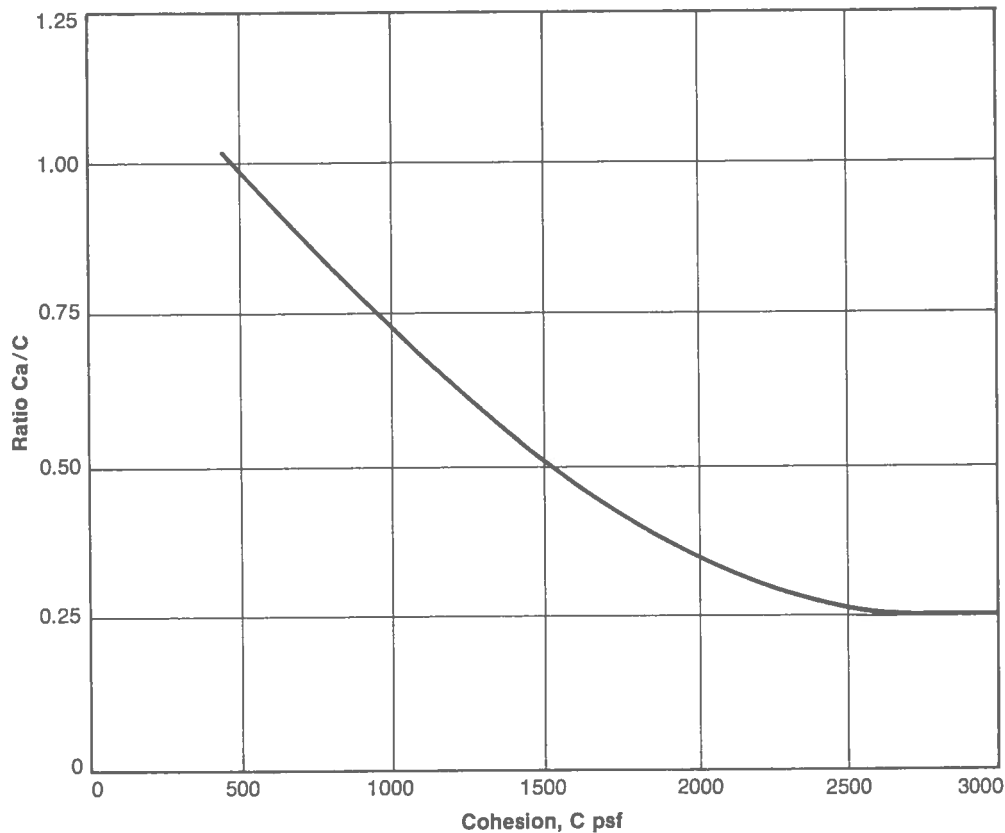


FIGURE 9—ADHESION IN FINE-GRAINED SOILS (after Navdocks⁹)

taken as a decreasing proportion of the cohesion as the cohesion increases, as illustrated by Figure 9.^{3,9} If laboratory test results are not available, cohesion can be approximated from Figure 6. The cohesion is equal to one-half the unconfined compressive strength, q_u . Projections of a bottom plate beyond the pile surface, transient lateral loads, vibrations, or any other factors which may produce an opening between pile and soil and tend to reduce adhesion. Since the factors in this relationship are poorly defined, it is utilized primarily for estimates to be confirmed by field load tests.

The skin friction component of ultimate capacity is given by:

For H-piles, the least of:

$$Q_{us} = 2(d + b_f)cD \text{ (Cohesion around the net perimeter.)} \quad (\text{Eq. 10})$$

$$Q_{us} = 2(d + sb_f)c_a D \text{ (Adhesion around the entire perimeter.)} \quad (\text{Eq. 11})$$

$$Q_{us} = 2(dc + b_fc_a)D \text{ (Adhesion to the flanges and cohesion across the web openings.)} \quad (\text{Eq. 12})$$

For pipe piles, the lesser of:

$$Q_{us} = \pi d D c \text{ (Cohesion around the perimeter.)} \quad (\text{Eq. 13})$$

$$Q_{us} = \pi d D c_a \text{ (Adhesion around the perimeter.)} \quad (\text{Eq. 14})$$

where Q_{us} = skin friction component of ultimate capacity (pounds).

b_f = flange width of H-pile (feet).

d = section depth of H-pile or diameter of pipe pile (feet).

D = embedment of pile in fine-grained material (ft).

c = cohesion of fine-grained soil (psf).

c_a = adhesion between fine-grained soil and pile (psf).

The end bearing component of ultimate capacity of single piles in fine-grained soil is given by:³

$$Q_{ue} = 9cA \quad (\text{Eq. 15})$$

where Q_{ue} = end bearing component of ultimate capacity (lbs).

c = cohesion of fine-grained soil (psf).

A = base area (square feet).

For pipe piles, the ultimate capacity $Q_u = Q_{us} + Q_{ue}$

For H-piles, the end bearing component of ultimate capacity should be neglected, and the ultimate capacity considered equal to the skin friction: $Q_u = Q_{us}$ (Eq. 1A)

Piles in fine-grained soils tend to undergo creep movements when subjected to prolonged tension loads. Therefore, the ultimate pullout capacity for long-term tension loads should be taken no higher than 0.7 of the value obtained from Equations 10 to 14. Section 1.4.4F of the AASHTO Specifications restricts the allowable uplift per pile to 0.4 of the allowable compressive load per pile.

Where slender piles pass through soft clay overlying the bearing stratum, it is necessary to ensure that the ultimate capacity determined in the above specified manner does not exceed the capacity of the pile acting as a column. This latter capacity may be determined from the following expression:⁹

$$Q_{\text{buckling}} = j(cEI_1)^{1/2} \quad (\text{Eq. 16})$$

where j = dimensionless parameter = 8 for $c < 250$ psf
 = 10 for $250 < c < 500$ psf

I_1 = lowest moment of inertia of pile cross section (feet⁴).

E = modulus of elasticity of steel (psf).

5. *Fine-Grained Soils—Dynamic Evaluation of Capacity*: The phenomena that create dynamic resistance to the penetration of piles during driving in fine-grained soil are completely different from those that provide resistance to static loading. During driving the dynamic resistance to penetration of the pile point is large, as is characteristic in all plastic materials, and skin friction is usually very small with the sides of the pile lubricated by a thin film of water squeezed out of the soil. Under static loading the major portion of resistance to pile loading comes from skin friction which develops with time, and point resistance usually contributes a small portion of the total. Hence, any correspondence between the dynamic and static resistances is coincidental and dynamic formulas are not applicable. Dynamic resistance is, however, a useful measure of the consistency or hardness of the fine-grained soil and hence is a useful control in pile driving after load capacity has been determined by load tests or estimated from the static formulas previously presented.

6. *Multi-Strata Soils*: When a pile penetrates different strata of soil, the ultimate load capacity of the pile is approximately equal to the sum of the resistances due to skin friction in each strata plus the end bearing resistance at the tip of the pile.

The ultimate resistance due to skin friction in each layer of coarse-grained material is given by:

$$Q_{us} = [\Sigma \gamma_e D_i + \frac{1}{2} \gamma_e D] D p K_b (\tan \delta) \quad (\text{Eq. 4A})$$

where $\Sigma \gamma_e D_i$ = total weight of soil above top of stratum being considered.

γ_e = average effective unit weight of soil in stratum being considered (pcf).
 (Use the submerged unit weight γ' where below water table.)

D = embedment of pile in stratum being considered.

The ultimate resistance due to skin friction in each layer of fine-grained material is given by Equations 10 to 14.

The ultimate resistance due to end bearing in coarse-grained material where the pile penetrates the coarse-grained stratum by at least 10 times the section depth d of H-piles or 10 times the diameter d of pipe piles is given by:

$$Q_{ue} = [\frac{1}{2} \gamma_e d N_\gamma + K_b N_q \sum \gamma_e D_i] A \quad (\text{Eq. 2A})$$

where $\sum \gamma_e D_i$ = total weight of soil above tip of pile.

The ultimate resistance due to end bearing in coarse-grained material where the pile tip is less than 10 times the section depth d of H-piles or 10 times the diameter d of pipe piles below the ground surface is given by:

$$Q_{ue} = \frac{D}{10d} (\frac{1}{2} \gamma_e d N_\gamma + K_b N_q \sum \gamma_e D_i) A \quad (\text{Eq. 3A})$$

where D = embedment of pile in coarse-grained stratum.

The ultimate resistance due to end bearing in fine-grained material should be neglected for H-piles and is given by Equation 15 for pipe piles.

E. Groups of Embedded Piles

1. *Introduction*: The design of pile groups normally is based upon an analysis of the behavior of a single pile. This approach is essential when pile capacity is established by single pile load tests or dynamic driving formulas. Normally, all piles in a group are of the same type and are driven to the same depth, despite evidence that the distribution of loads within a pile group is not uniform.

When piles are driven in a group, stresses in the soil beneath the tip of one pile are affected by the driving of adjacent piles, and stresses along the length of a friction pile are affected by the driving of adjacent piles. In some types of soil, the capacity of the pile group will be considerably less than the capacity of a single pile times the number of piles.

2. *Rock*: The capacity of a group of piles driven to refusal in end bearing on sound rock may be assumed equal to the capacity of a single pile times the number of piles. Distributed bearing pressures in the rock beneath the pile tips should not exceed the allowable bearing capacity of the rock as determined, if possible, by load testing of the applicable pile. Theoretical bearing capacities of various rocks as applicable to spread footings are seldom pertinent to piles.

3. *Coarse-Grained Soils*: The driving of a group of piles in loose coarse-grained material causes greater densification of the material than the driving of a single pile, thereby yielding an average ultimate capacity per pile higher than for a single pile driven in the same soil. Normally, this increase in capacity is neglected, and each pile in the group is designed for the capacity of a single driven pile.

The driving of a group of piles in very dense coarse-grained material causes greater dilation of the material than the driving of a single pile, thereby yielding an average ultimate capacity per pile lower than for a single pile in the same soil.

4. *Fine-Grained Soil*: In fine-grained soils, pile groups impose increased shearing stresses within the intervening soil, but without contributing any strength increase corresponding to that developed by densification in loose coarse-grained soils. Thus the average pile capacity in a group in fine-grained soils often is less than the capacity of a single pile.

The average pile capacity is determined either by empirical efficiency equations or by a theoretical method considering all soil bounded within the periphery of the pile group to act as a deep block foundation. The use of the theoretical method is recommended.

(a) *Empirical Efficiency Equations*: Several efficiency equations have been developed to account for the behavior of pile groups. These formulas attempt to relate the behavior of one pile to the behavior of a group, and do not consider the characteristics of the soil or the length of the piles.

The Converse-Labarre formula is recommended in Section 1.4.4(G) of the AASHTO Specifications. This formula is based upon the assumption that the area of the pile

available for developing shear is reduced by the influence of adjacent piles and takes the form:

$$E = 1 - \phi \frac{(n-1)m + (m-1)n}{90mn} \quad (\text{Eq. 16})$$

where E = the efficiency or the decimal fraction of the single pile capacity to be used for each pile in the group.

n = number of piles in each row of a pile group.

m = number of rows in a pile group.

d = diameter of pipe pile or section depth of H-pile.

s = center to center spacing of piles.

$\tan \phi = d/s$

ϕ is numerically equal to the angle expressed in degrees.

(b) *Theoretical Method*: The piles and the fine-grained material enclosed within the pile group are treated as a deep block footing founded at a depth corresponding to the pile tips. As shown in Figure 10, the ultimate capacity of the pile group is composed of the skin friction around the periphery of the pile group and the end bearing resistance of the group.

The skin friction component of ultimate capacity of the pile group is given by:

$$Q_{us} = 2D(B+L)c \quad (\text{Eq. 17})$$

where Q_{us} = skin friction component of ultimate capacity (pounds).

D = embedment of pile in fine-grained material (feet).

B = width of pile group (feet).

L = length of pile group (feet).

c = average cohesion of fine-grained soil (psf).

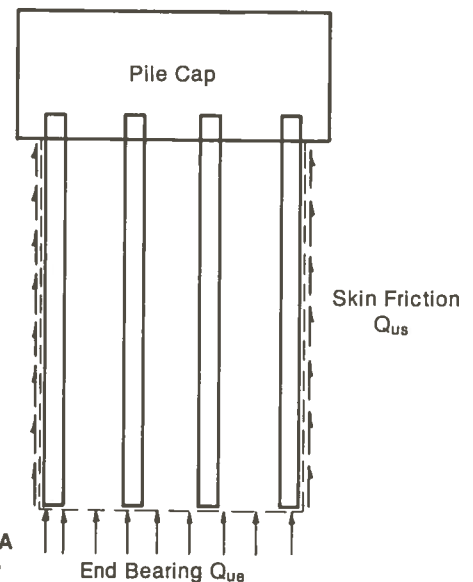


FIGURE 10—CAPACITY OF PILE GROUPS IN FINE GRAINED SOIL BASED ON END BEARING RESISTANCE OF GROSS AREA OF THE GROUP AND SKIN FRICTION ALONG ITS SIDES

The end bearing component of ultimate capacity of the pile group is given by:

$$Q_{ue} = 9c(BL) \quad (\text{Eq. 18})$$

where Q_{ue} = end bearing component of ultimate capacity (pounds).

The ultimate capacity of the pile group is $Q_u = Q_{ue} + Q_{us}$.

This method assumes that load is distributed uniformly to the piles at the top of the pile group. Piles under highway foundations normally are positioned to be loaded equally under constantly applied loads such as dead load and earth pressure, and possibly under some live load conditions. Wind, centrifugal force, longitudinal force and temperature change cause pile loads to be unequal with the maximum pile load occurring on one or a few piles in the group. Under these conditions, the ultimate group capacity probably should be compared to the total vertical load on the pile group, not to the maximum pile load multiplied by the number of piles in the group.

F. Settlement

1. *Coarse-Grained Soils*: Computation of settlement of piles in coarse-grained soils is a highly inexact procedure and the following method is intended as an aid to judgment only. Where a reliable load settlement relationship is required a pile load test should be performed.

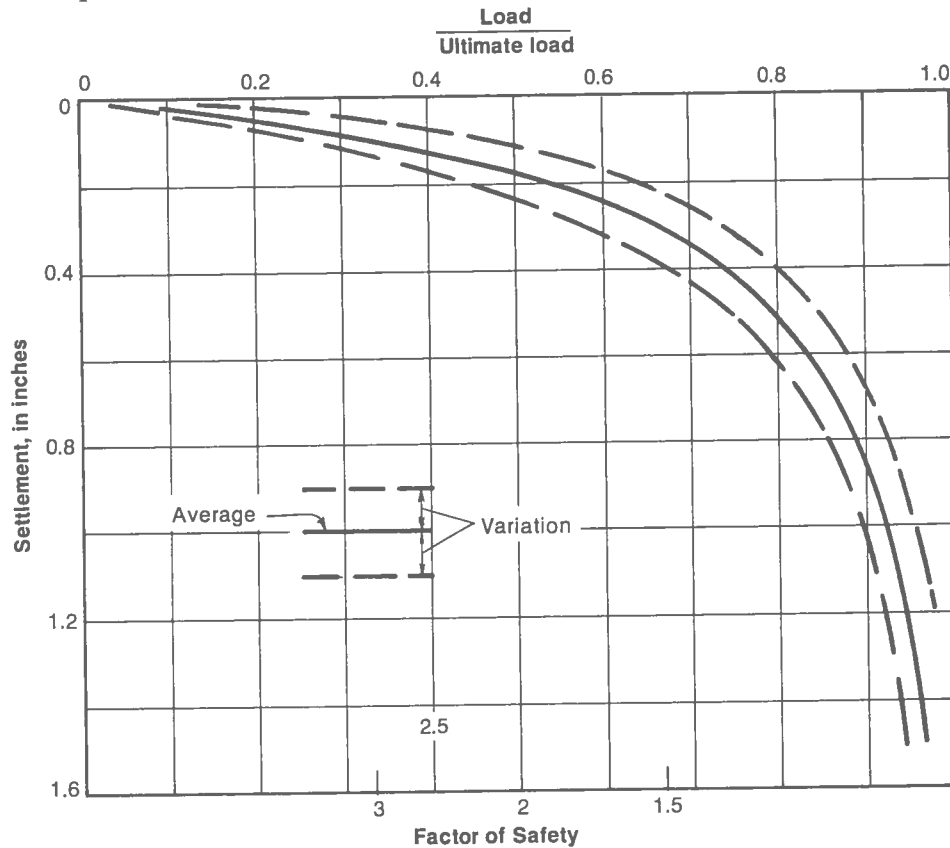


FIGURE 11—LOAD SETTLEMENT CURVES FOR PILES DRIVEN IN COARSE GRAINED SOIL (after Skempton¹⁸)

(a) *Single Pile*: For a single pile, the settlement under load can be estimated empirically from design curves proposed by Skempton.^{18 11} Figure 11 shows a relationship between the settlement and the factor of safety of the pile. Since the factor of safety is usually in the order of 2.5, the settlement of a single pile will not be excessive provided the layer is deep and fairly homogeneous. To the above value the elastic shortening in the pile must be added, assuming a reasonable distribution of applied load through the pile to determine settlement at the top of pile.

(b) *Pile Group*: The settlement of a pile group is usually greater than the settlement of an individual pile for the same unit pile loading. This relationship is shown in Figure 12. In this figure Skempton relates the width of foundation to the ratio between the settlement of the foundation and the settlement of a single pile.

If settlement of a pile group is critical, the allowable load on a pile group may be controlled by considerations of settlement rather than ultimate bearing capacity.

Settlement due to elastic shortening of the pile must be added to the above settlement.

(c) *Multistrata Conditions*: Where the layer supporting the pile tips is underlain by a layer or layers of weaker soil, additional settlement due to these layers can be expected. The stress in these layers may be estimated by assuming the total sustained load on the pile group to be applied to the soil at the level of the pile tips over the area enclosed by the group perimeter and to be distributed to the underlying soil with a 30° spread from the vertical similar to Figure 14.

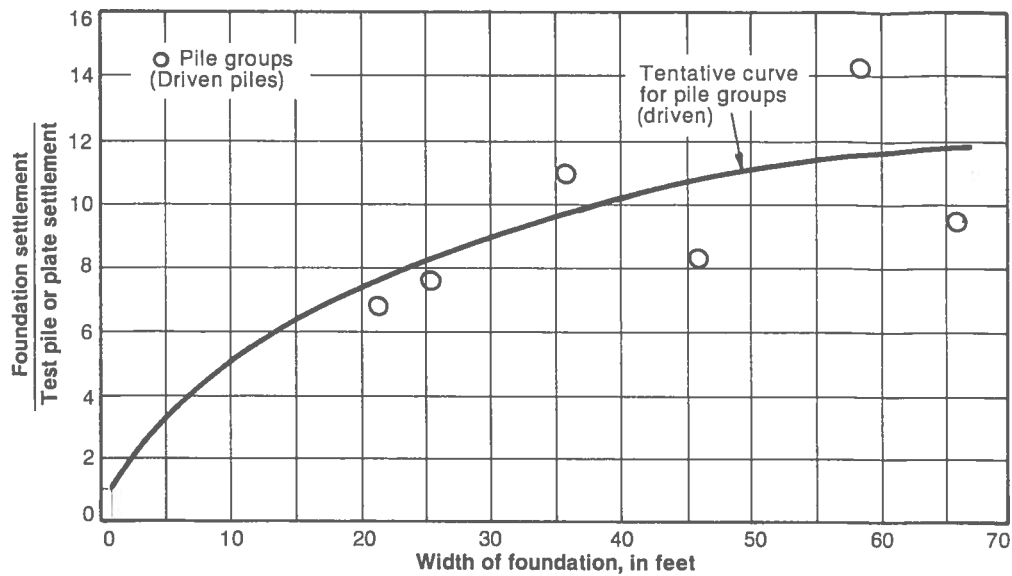


FIGURE 12—DESIGN CURVE FOR SETTLEMENT OF FOUNDATIONS IN COARSE GRAINED SOIL (after Skempton¹⁸)

2. Fine-Grained Soils:

(a) *Single Pile*: Settlement of a single pile under sustained loading may be estimated by assuming the applied load to be uniformly distributed over a circular area at the lower third point of the pile penetration in the bearing layer as shown in Figure 13. Settlement is computed by a consolidation analysis for a soil layer starting at the tip of the pile and extending upward a distance of $D/3$. The soil beneath the pile tip should be checked for a more compressible layer which would result in additional settlement.

(b) *Pile Group*: In the case of pile groups both the intensity of stress and the depth of the stressed zone will increase in the soil below the pile tip. Thus the settlement of a pile group is greater than the settlement of a single pile. Settlement of a

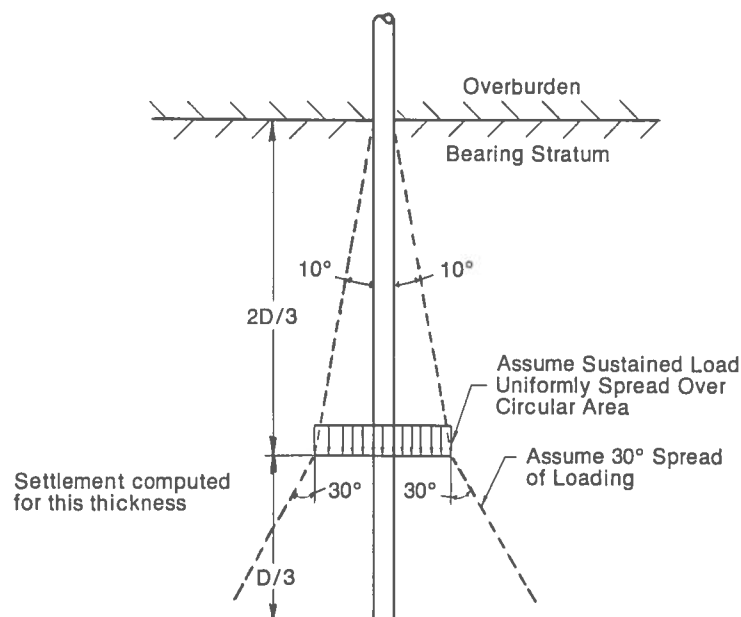


FIGURE 13—ASSUMED LOAD DISTRIBUTION FOR COMPUTATION OF SETTLEMENTS OF SINGLE PILES IN FINE GRAINED SOILS

pile group in a deep homogeneous soil may be approximated by assuming that the total sustained loading on the group is applied as a uniform pressure within the perimeter of the group, at a level $\frac{2}{3}$ down that length of the pile within the bearing layer, and is distributed to the underlying soil with a 30° spread, as shown in Figure 14. Settlement is computed by a consolidation analysis for a soil layer starting from the lower third point of the pile penetration to a depth below the pile tip equal to about twice the width of the group.

The above methods are very approximate and serve only as a guide. The final settlement will be influenced by such factors as long term consolidation, soil disturbance due to pile driving, surcharge loading at the ground surface and negative skin friction.

The settlement computed by the above methods should include the elastic shortening of the pile under the axial loading.

(c) *Multistrata Conditions*: Settlement under working load for a pile group may be estimated by extending the 30° spread of loading as shown in Figures 13 and 14 down through all compressible layers. The average pressure in the various layers is then determined and used in a consolidation analysis to compute the settlement in each layer. The settlement in each layer is added to obtain the total settlement.

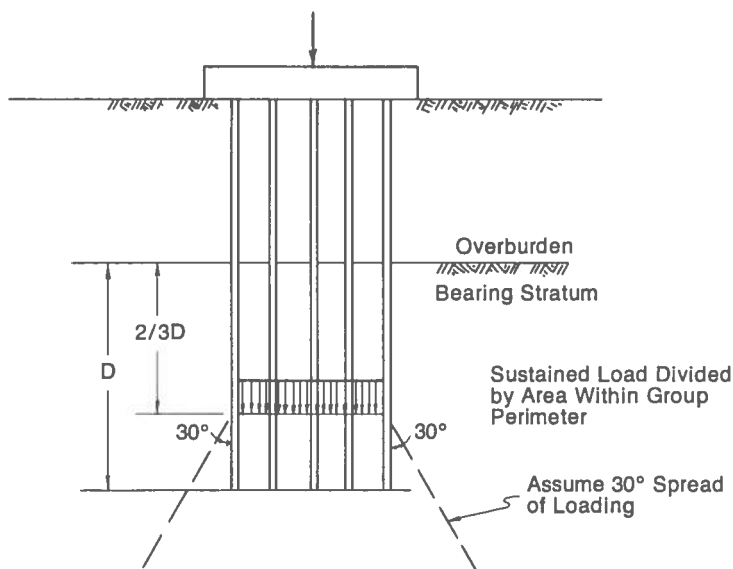


FIGURE 14—ASSUMED LOAD DISTRIBUTION FOR COMPUTATION OF SETTLEMENTS OF GROUPS IN FINE-GRAINED SOILS

G. Negative Skin Friction

Additional load on piles due to negative skin friction may occur under any of the following conditions:

1. Piles driven through a fine-grained soil of high sensitivity. (Generally greater than 3 where sensitivity is defined as the ratio of undisturbed to fully remoulded shear strength.)
2. Additional fill is placed above a fine-grained compressible stratum shortly before or after driving piles.
3. The ground water level is lowered.

Under any of these conditions, the fine-grained compressible soil and any overlying material will gradually settle downward. If the pile tips are located in the fine-grained compressible stratum, pile settlement will be excessive. If piles penetrate below the compressible stratum and pile capacity can be developed below the compressible stratum, the downward movement of the compressible stratum and overlying material can exert considerable additional load on the piles.

The additional load caused by negative skin friction may be considered in two parts. The first part consists of the weight of all soil above the compressible stratum

and within limits of the pile group. After placement of fill or after driving piles in sensitive fine-grained soil, the compressible stratum tends to settle. However, the material located above the compressible stratum cannot settle freely because its downward movement is resisted by skin friction on the piles. A very small downward movement is sufficient to transfer all the weight of the material above the compressible layer onto the piles, thus placing an additional load onto the piles as given by:¹²

$$Q' = \frac{BL}{r} \gamma_e D_c \quad (\text{Eq. 19})$$

where Q' = Additional load per pile due to weight of fill or coarse-grained material above compressible stratum (pounds).

B = width of pile group (feet).

L = length of pile group (feet).

r = number of piles in pile group.

γ_e = average effective unit weight of soil (pounds per cubic foot).

D_c = length of piles in coarse-grained stratum or fill overlying sensitive fine-grained strata (feet).

The second part of the additional load is caused by settlement of the compressible fine-grained stratum outside the periphery of the pile group. Within the pile group, the piles prohibit appreciable settlement of the compressible layer. Outside the periphery of the pile group, greater settlement will occur and exert a downward drag on the pile group. The magnitude of this drag depends upon the amount of settlement with a maximum value as given by:¹²

$$Q''_{\max} = \frac{2(B+L)D_f c}{r} \quad (\text{Eq. 20})$$

where Q''_{\max} = maximum additional load per pile due to drag of compressible fine-grained material (pounds).

D_f = length of piles in sensitive fine-grained stratum (feet).

c = cohesion of fine-grained stratum (psf).

When designing piles that extend through a fine-grained soil of high sensitivity or when new fill is placed above a compressible fine-grained stratum shortly before or after driving piles, the total load per pile should be taken equal to the sum of the applied structural loading and $Q' + Q''_{\max}$. Settlements should be calculated on the assumption that the total load (including $Q' + Q''_{\max}$) is transferred to the supporting stratum as a uniform pressure within the boundary of the group at the level of the pile tips. No gain in pull-out capacity should be considered.

H. Lateral Loads

1. *Introduction:* The ability of piles to resist lateral loads depends upon the development of horizontal soil resistance along the pile length as a result of lateral deflections. The type of soil, pile size and type, the embedment of pile head, depth of pile penetration and type of loading are important factors in evaluation of lateral load capacity. Generally, the allowable lateral load is governed by the amount of lateral movement considered acceptable.

The following procedures to design piles for lateral loads represent a simplified method, based on studies by Broms.^{6,7,8} Lateral loads and displacements calculated by these procedures are approximate and should be used only as estimates. Field tests should be employed where accurate determinations of these values are required.

Piles subjected to lateral loads may be classified according to the following conditions:

- (a) Free-headed or restrained.
- (b) Short, intermediate or long.

A free-headed pile is free to rotate at its top. A restrained pile is fixed against rotation at its top by sufficient embedment of the pile head into the concrete cap to develop a fixed end moment at the top of pile.

The ultimate lateral resistance of a pile is governed by either the yield strength of the pile section or by the ultimate passive resistance of the supporting soil. For a short pile, the ultimate lateral load is governed by the passive resistance of the soil; whereas, the plastic strength of the pile section governs for a long pile. A pile of intermediate length should be checked for both modes of failure.

Failure modes are shown in Figure 15.

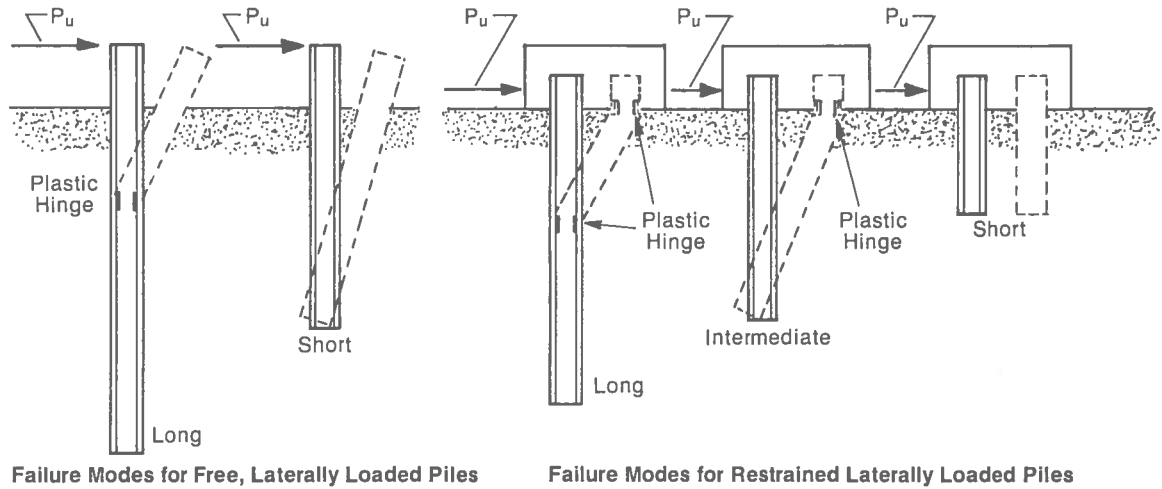


FIGURE 15—FAILURE CRITERIA (after Broms⁶)

2. *Coarse-Grained Soils—Capacity*: A pile is considered a short pile when the value of ηD is smaller than 2.0 and a long pile when larger than 4.0, and intermediate for values between 2.0 and 4.0.

where D = length of pile embedment (feet).

$$\eta = \sqrt[5]{\frac{n_h}{EI}} \quad (\text{Eq. 21})$$

and EI = section stiffness about axis perpendicular to loading plan (kip-ft²).

n_h = coefficient of horizontal subgrade reaction for a long strip with a width of unity at a depth of unity below the ground surface (kips/c.f.).

This procedure assumes the coefficient of subgrade reaction to increase linearly with depth. Values of n_h are given in Figure 16 and may be used in the absence of field plate load tests.

Relative density of soil	Coefficient n_h (Kips per cubic foot)		
	Loose	Medium	Dense
Above ground-water table	14	42	112
Below ground-water table	8	28	68

FIGURE 16

The ultimate capacity of short piles may be determined from Figure 17 by substitution of values of the following dimensionless quantities:

$$\frac{P_u}{K_p b^3 \gamma_o}; \quad \frac{D}{b}; \quad \frac{e}{D} \quad (\text{Eq. 22})$$

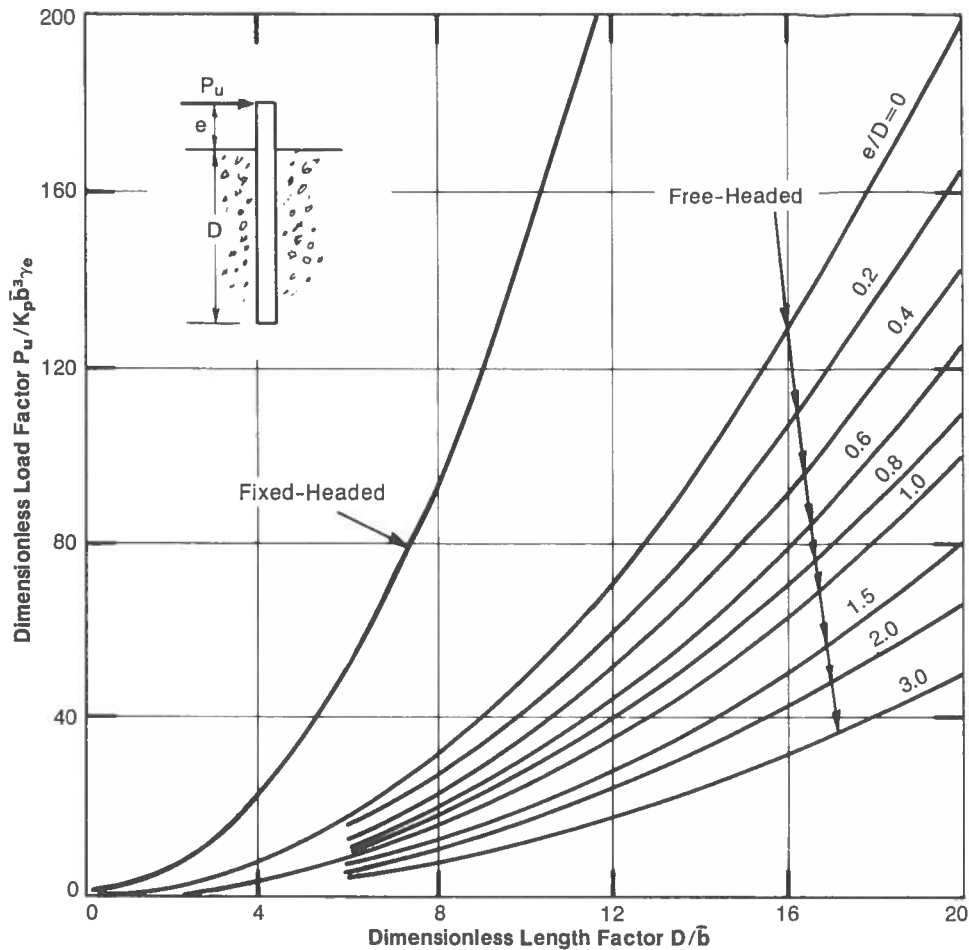


FIGURE 17—ULTIMATE LATERAL LOAD CAPACITY OF SHORT PILES IN COARSE-GRAINED SOIL (after Broms⁷)

where P_u = ultimate lateral load capacity (pounds).

K_p = Rankine passive pressure coefficient = $\tan^2 (45 + \phi/2)$

\bar{b} = section width of H-pile perpendicular to loading, replace by diameter d for pipe pile (feet).

γ_e = average effective density over embedded length of pile (pcf).

D = embedment below ground surface (feet).

e = height of line of action of lateral load above ground surface (feet).

The ultimate lateral load capacity of long piles may be estimated from Figure 18 by substituting values of the following dimensionless quantities:

$$\frac{P_u}{K_p \bar{b}^3 \gamma_e}; \quad \frac{M_{\text{plastic}}}{\bar{b}^4 \gamma_e K_p}; \quad \frac{e}{\bar{b}} \quad (\text{Eq. 23})$$

The plastic moment may be estimated from the following expression where the applied axial load is small.

$$M_{\text{plastic}} = n_s f_v S \quad (\text{Eq. 24})$$

where n_s = dimensionless shape factor = 1.3 for pipe section.

= 1.1 for H-section where applied lateral loading is in direction of maximum moment resistance of pile (normal to pile flanges).

= 1.5 for H-section where applied lateral loading is in direction of minimum moment resistance of pile (parallel to pile flanges).

f_v = yield stress of pile material (psf).

S = section modulus about axis perpendicular to load plane (feet³).

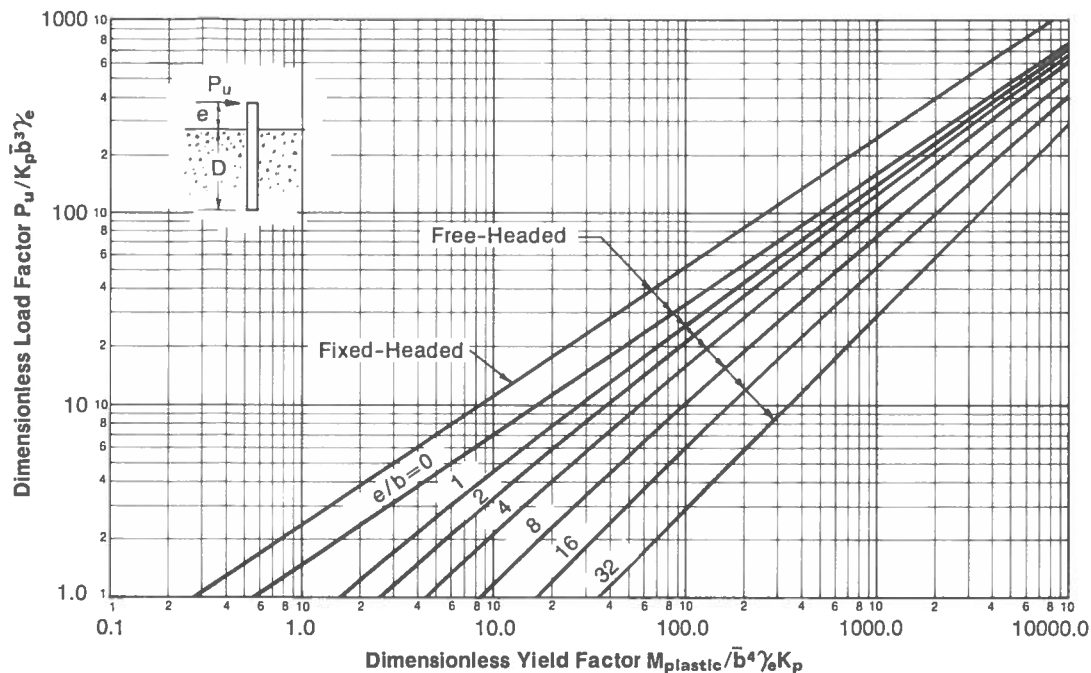


FIGURE 18—ULTIMATE LATERAL LOAD CAPACITY OF LONG PILES IN COARSE-GRAINED SOIL (after Broms⁷)

3. *Coarse-Grained Soils—Displacements*: The lateral capacity generally is limited by the permissible lateral displacement under working load. The lateral displacement at the ground surface may be estimated from Figure 19 by substitution of values of the following dimensionless quantities:

$$\frac{\Delta_a (EI)^{3/5} n_h^{2/5}}{P_a D}; \quad \eta D; \quad \frac{e}{D} \quad (\text{Eq. 25})$$

where P_a = working lateral load (pounds).

Δ_a = displacement at ground surfaces (feet).

For piles with sufficient embedment to be classified as long piles, the lateral displacement is independent of the pile embedment.

The largest part of the total displacement of a laterally loaded pile in coarse-grained soil occurs at the time of loading. However, repetitive loading or vibration may cause a substantial increase in displacement. The maximum displacement that could occur under repeated loads may be estimated by assuming that the coefficient of horizontal subgrade reaction n_h (see Figure 16) is reduced to $\frac{1}{4}$, $\frac{1}{3}$ and $\frac{1}{2}$ of its initial value for loose, medium and dense soils respectively.

Displacement values calculated by the above procedure usually will over-estimate the displacement at working loads except for piles which have been placed by jetting.

4. *Fine-Grained Soils—Capacity*: Piles embedded in fine-grained soils may be classified similarly to piles embedded in coarse-grained soils. A pile is considered a short pile when the value βD is smaller than 2.25 and a long pile when larger than 2.25:

$$\text{where } \beta = \sqrt[4]{\frac{k\bar{b}}{4EI}} \quad (\text{Eq. 26})$$

and k = coefficient of horizontal subgrade

$$\text{reaction} = \frac{160\bar{m}c}{\bar{b}} \quad (\text{Eq. 27})$$

in which $\bar{m} = 0.32$ for $c < 1$ ksf

$= 0.36$ for $1 < c < 4$ ksf

$= 0.40$ for $c > 4$ ksf

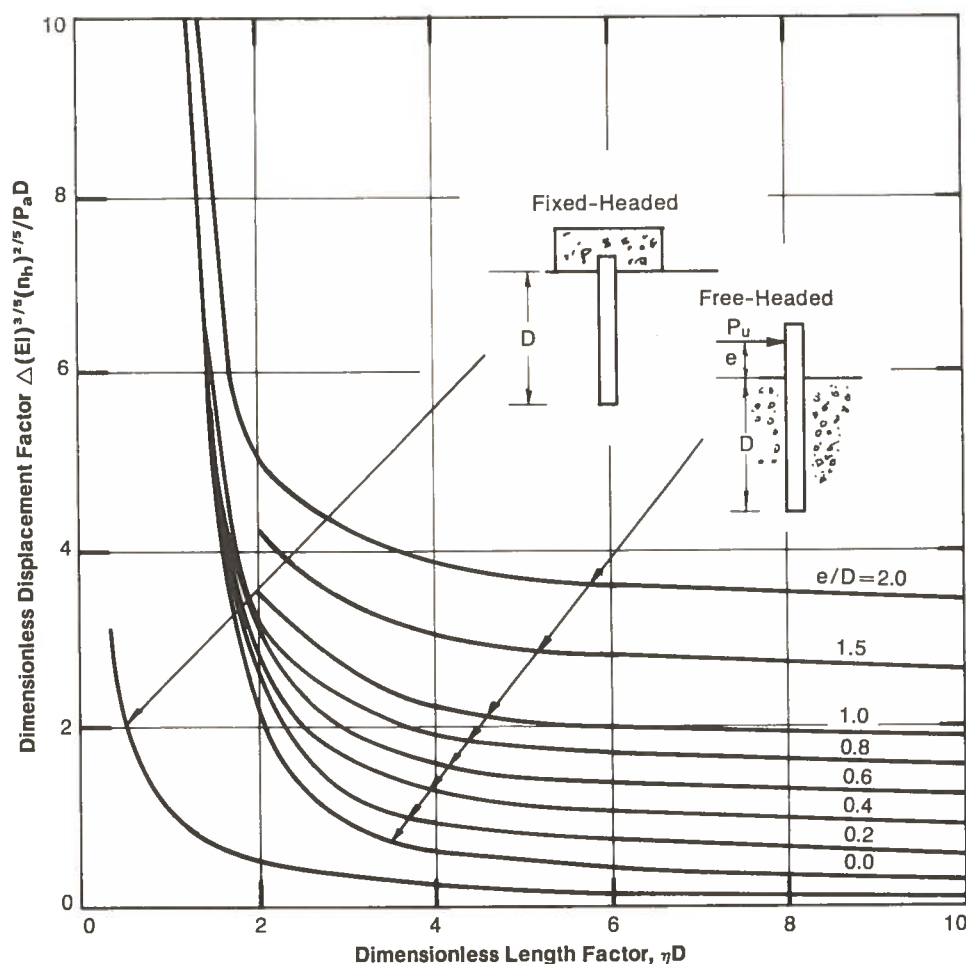


FIGURE 19—LATERAL DISPLACEMENTS OF PILES IN COARSE-GRAINED SOILS (after Broms⁷)

For fine-grained soil, the coefficient of subgrade reaction is assumed constant with depth.

The ultimate capacity of short piles may be estimated from Figure 20 by substitution of values of the following dimensionless quantities:

$$\frac{P_u}{cb^2}, \frac{D}{b}, \frac{e}{b} \quad (\text{Eq. 28})$$

The ultimate capacity of long piles may be estimated from Figure 21 by the substitution of values of the following dimensionless quantities:

$$\frac{P_u}{cb^2}, \frac{M_{\text{plastic}}}{cb^3}, \frac{e}{b} \quad (\text{Eq. 29})$$

5. *Fine-Grained Soils—Displacements*: The displacement of long piles in fine-grained soil at the ground surface, at working lateral loads less than one-half the ultimate values is independent of the pile embedment and may be approximated from Figure 22 by substitution of values of the following dimensionless terms:

$$\frac{\Delta_a k b D}{P_a}, \beta D, \frac{e}{D} \quad (\text{Eq. 30})$$

The displacement of short piles at the ground surface may be estimated as follows: The applied loading on the pile is resolved into a concentrated horizontal force P_a and moment M_e acting at the mid-height of the embedded length of the pile as shown

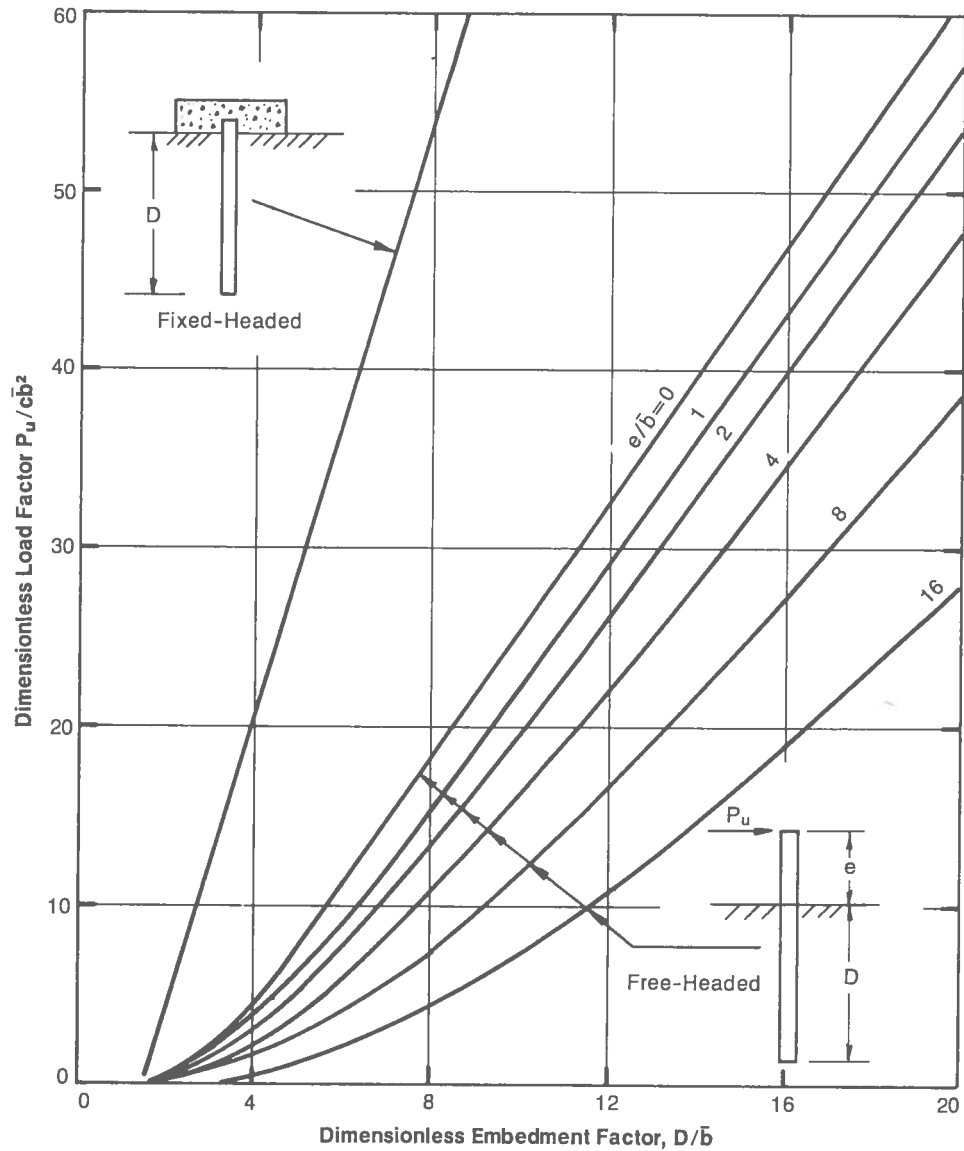


FIGURE 20—ULTIMATE LATERAL LOAD CAPACITY OF SHORT PILES IN FINE-GRAINED SOILS (after Broms⁶)

in Figure 23. The corresponding coefficients of subgrade reaction k_q and k_m are obtained from Figure 24 by substitution of the terms involved:

$$\frac{(k_q \text{ or } k_m) \bar{b}}{100c}; \quad \frac{D'}{\bar{b}} \quad (\text{Eq. 31})$$

where D' = embedded length in the determination of k_q
 $= 1/10D$ in the determination of k_m

The displacement at the ground surface attributable to the force P_a is then determined from:

$$\Delta_{lq} = \frac{P_a}{\bar{b} D k_q} \quad (\text{Eq. 32})$$

and the displacement attributable to the moment M_e determined from:

$$\Delta_{lm} = \frac{12.3 M_e}{\bar{b} D^2 k_m} \quad (\text{Eq. 33})$$

The sum of these component displacements yields an approximation of the total displacement at the ground surface at the time of initial load application.

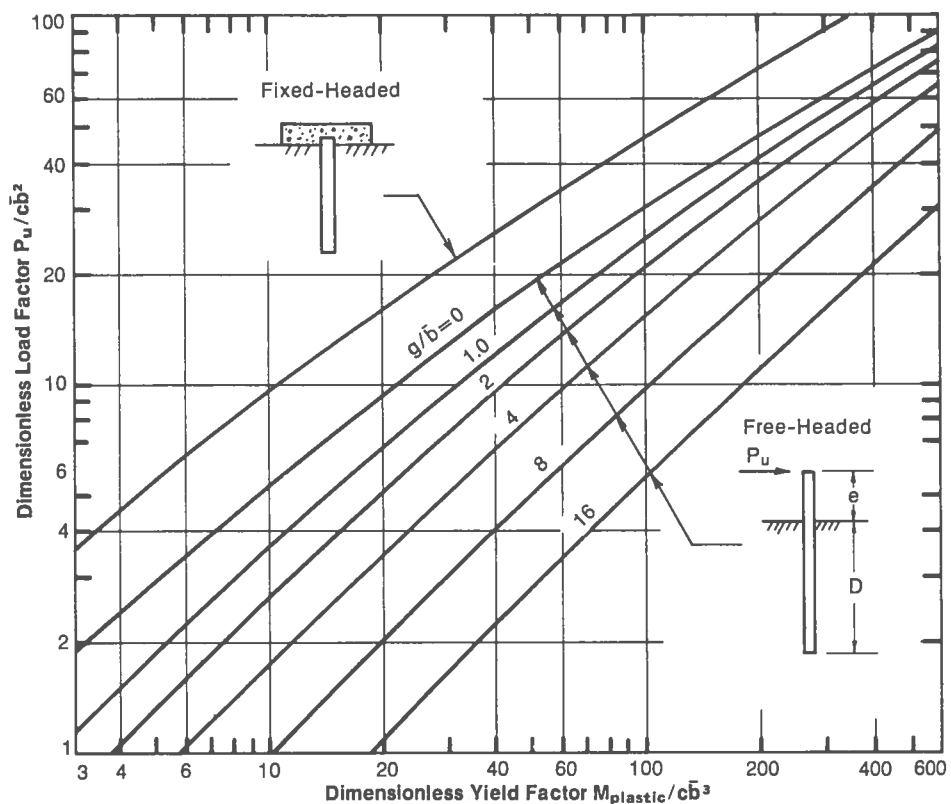


FIGURE 21—ULTIMATE LATERAL LOAD CAPACITY OF LONG PILES IN FINE-GRAINED SOILS (after Broms⁶)

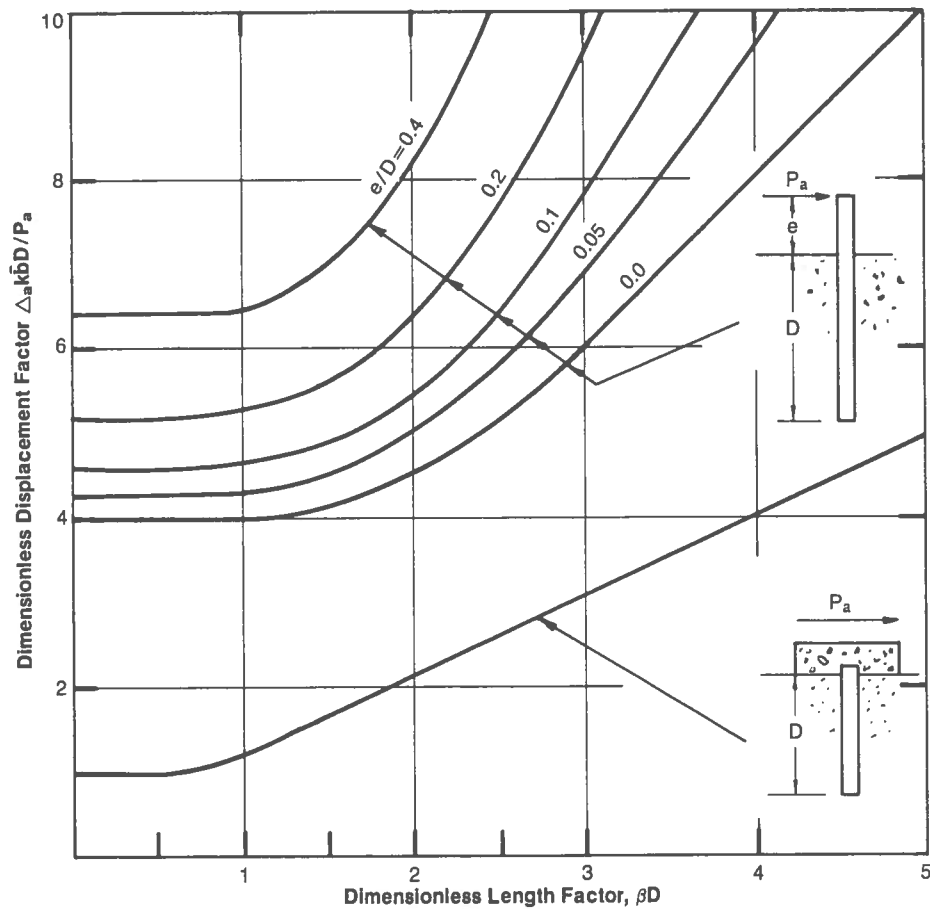
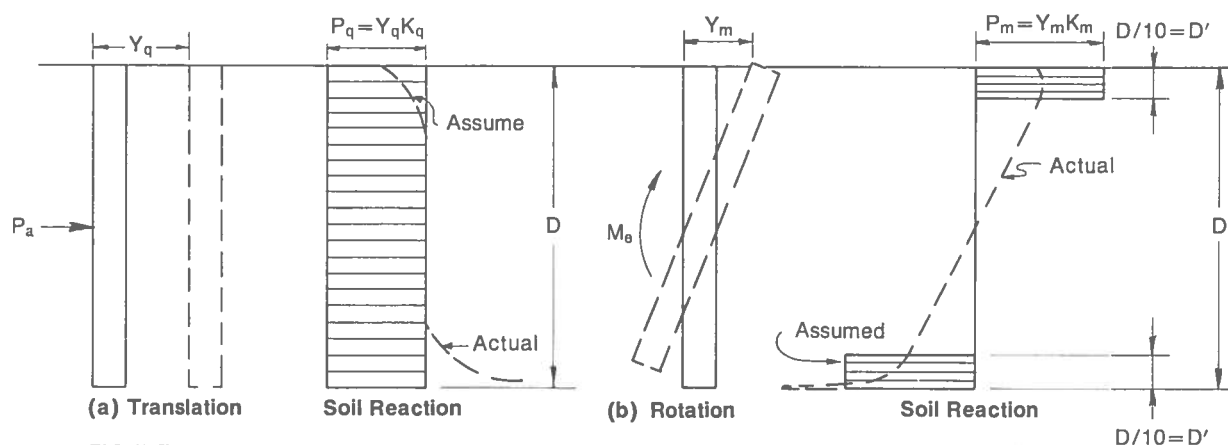


FIGURE 22—LATERAL DISPLACEMENTS OF LONG PILES IN FINE-GRAINED SOILS (after Broms⁶)



Lateral displacements in fine-grained soils increase with time due to consolidation and creep of the soil. To obtain approximate values of the maximum long term displacements, reduced values of the coefficient of subgrade reaction are used in Equations 26, and 30 to 33. The reduced values of coefficient of subgrade reaction are determined by multiplying the values given in Equation 27 by the following factors:

Reduction factor for coefficients of subgrade reaction k , k_q , k_m	Cohesive strength ksf
$\frac{1}{5}$	< 0.5
$\frac{1}{4}$	$0.5 \text{ to } 1.5$
$\frac{1}{3}$	> 1.5

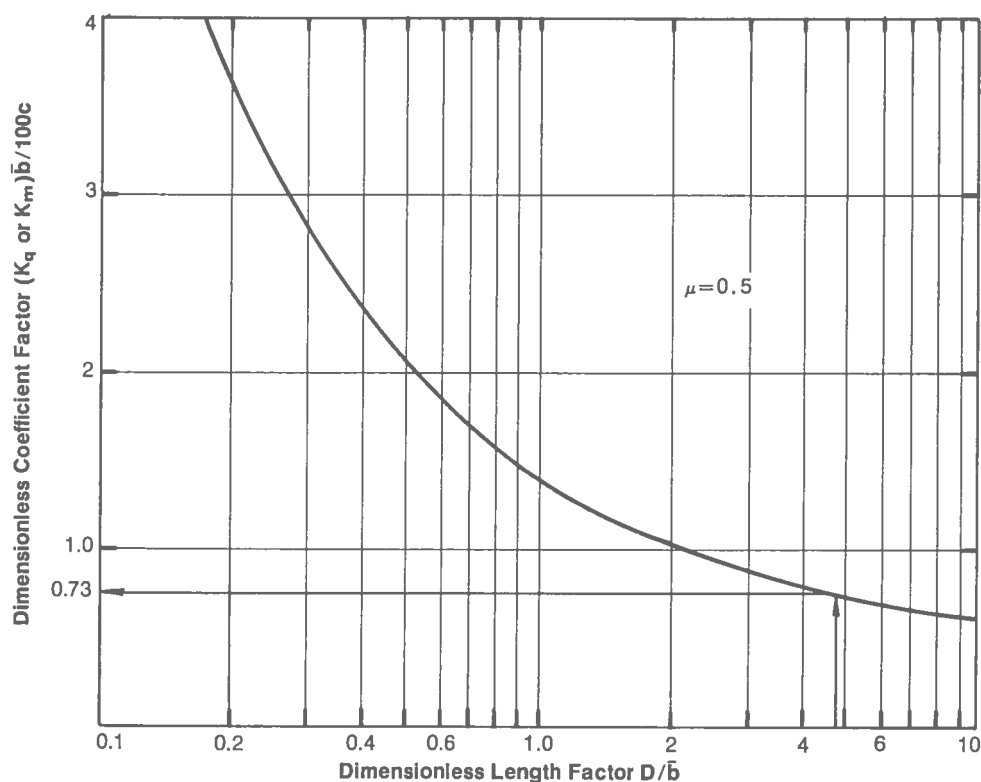


FIGURE 24—EVALUATION OF COEFFICIENTS OF SUBGRADE REACTION FOR DETERMINATION OF LATERAL DISPLACEMENTS OF SHORT PILES IN FINE-GRAINED SOILS (after Broms⁶)

Coarse Grained Material Above Water Table					
Pile	Type of Soil	Minimum Embedment for "Long" Pile Ft.	Lateral Load to Cause ¼" Lateral Movement Kips	Ultimate Lateral Load Capacity Kips	Ultimate Lateral Load Capacity ÷ 2.5 Kips
HP 10 x 42	Loose	20	3	25	10
	Medium Dense	16	6	27	11
	Dense	13	10	30	12
HP 10 x 57	Loose	21	3	30	12
	Medium Dense	17	7	32	13
	Dense	14	12	35	14
HP 12 x 53	Loose	22	4	35	14
	Medium Dense	18	7	37	15
	Dense	15	13	40	16
HP 12 x 74	Loose	24	4	45	18
	Medium Dense	19	9	47	19
	Dense	16	15	50	20
HP 14 x 73	Loose	25	5	50	20
	Medium Dense	20	9	55	22
	Dense	17	17	60	24
HP 14 x 89	Loose	27	5	58	23
	Medium Dense	21	10	63	25
	Dense	18	19	70	28

Note: Calculated lateral loads are based upon the following soil properties:

Loose Soil: $\gamma_e = 95$ Lbs./Cu. Ft., $\phi = 28^\circ$, $n_h = 14,000$ Lbs./Cu. Ft.
Medium Dense Soil: $\gamma_e = 110$ Lbs./Cu. Ft., $\phi = 30^\circ$, $n_h = 42,000$ Lbs./Cu. Ft.
Dense Soil: $\gamma_e = 110$ Lbs./Cu. Ft., $\phi = 36^\circ$, $n_h = 112,000$ Lbs./Cu. Ft.

FIGURE 25—LATERAL LOAD CAPACITY OF SINGLE "FREE-HEADED" PILES LOADED AT GROUND SURFACE NORMAL TO FLANGE ($\rightarrow|$)

Coarse Grained Material Below Water Table					
Pile	Type of Soil	Minimum Embedment for "Long" Pile Ft.	Lateral Load to Cause ¼" Lateral Movement Kips	Ultimate Lateral Load Capacity Kips	Ultimate Lateral Load Capacity ÷ 2.5 Kips
HP 10 x 42	Loose	22	2	20	8
	Medium Dense	17	5	23	9
	Dense	14	8	23	9
HP 10 x 57	Loose	23	2	25	10
	Medium Dense	18	5	25	10
	Dense	15	9	30	12
HP 12 x 53	Loose	25	3	30	12
	Medium Dense	20	6	30	12
	Dense	16	10	35	14
HP 12 x 74	Loose	27	3	35	14
	Medium Dense	21	7	37	15
	Dense	18	11	40	16
HP 14 x 73	Loose	28	4	45	18
	Medium Dense	22	7	45	18
	Dense	19	12	50	20
HP 14 x 89	Loose	30	4	50	20
	Medium Dense	23	8	52	21
	Dense	19	13	57	23

Note: Calculated lateral loads are based upon the following soil properties:

Loose Soil: $\gamma' = 55$ Lbs./Cu. Ft., $\phi = 28^\circ$, $n_h = 8,000$ Lbs./Cu. Ft.
Medium Dense Soil: $\gamma' = 60$ Lbs./Cu. Ft., $\phi = 30^\circ$, $n_h = 28,000$ Lbs./Cu. Ft.
Dense Soil: $\gamma' = 65$ Lbs./Cu. Ft., $\phi = 36^\circ$, $n_h = 68,000$ Lbs./Cu. Ft.

FIGURE 26—LATERAL LOAD CAPACITY OF SINGLE "FREE-HEADED" PILES LOADED AT GROUND SURFACE NORMAL TO FLANGE ($\rightarrow|$)

6. *Combined Axial and Lateral Loading*: Piles required to withstand both axial and lateral loading usually fall under the designation of long piles, because of the length generally required to sustain the axial load. For such piles the ultimate lateral load capacity is dependent upon the reduced plastic resistance of the pile section.

The reduced plastic resistance can be found from the following equations:¹⁴

$$M_o = M_{\text{plastic}} \text{ when } Q_m/Q_y \leq 0.15 \quad (\text{Eq. 34})$$

$$\text{or } M_o = 1.18M_{\text{plastic}} [1 - Q_m/Q_y] \text{ when } Q_m/Q_y > 0.15 \quad (\text{Eq. 35})$$

where Q_m = axial load (kips).

$Q_y = f_y A_s$ (kips).

f_y = yield stress of pile material (kips/square inch).

A_s = area of steel section (square inch).

Yield stress values for the various bearing pile grades are presented in Figure 1. In Equations 23 and 29 and in Figures 18 and 27, M_{plastic} should be replaced by M_o as found from the above equations.

7. *Lateral Loading—Multistrata Condition*: Very little work has been performed for laterally loaded piles in a multistrata condition. Davison and Gill¹⁷ found that the surface layer governs the pile response to lateral loads and present a method of analysis for multistrata conditions.

The following approximations are suggested:

- (a) Surface layer presents lower lateral resistance than deeper layers.
 - (1) Determine the lateral load capacity using soil properties of the surface layer full depth of the pile.
 - (2) Determine the lateral load capacity assuming the surface layer offers no lateral resistance. In this calculation, increase the arm " e " by the depth of the surface layer and use soil properties of the deeper layer.
 - (3) Base design upon the higher of the lateral load capacities determined in (1) and (2).
- (b) Surface layer presents higher lateral resistance than deeper layers.
 - (1) Determine the lateral load capacity using soil properties of the surface layer full depth of the pile.
 - (2) Determine the lateral load capacity using soil properties of the deeper layer full depth of the pile.
 - (3) Base design upon a value between the lateral load capacities determined in (1) and (2).

8. *Approximate Values of Allowable Lateral Load Per Pile*: Calculation of the allowable lateral load per pile is time consuming and often is complicated by poor descriptions of the soil, by variable soil, and by group action. As an aid to the designer, the values tabulated in Figures 25, 26 and 27 have been calculated using Brom's method. The tabulated values apply for a single "free-headed" pile loaded at the ground surface with a load normal to the flange. It is assumed that all piles are "long" by Brom's definition and that the pile stress due to axial load is $\frac{1}{4}$ of the yield stress. The minimum embedment for a "long" pile, the lateral load required to cause $\frac{1}{4}$ " lateral movement, the ultimate lateral load capacity, and the ultimate lateral load capacity divided by a factor of safety of 2.5 are tabulated. The ultimate lateral load capacity never governs design unless very large lateral deflections are permissible.

Within the range of lateral deflections normally considered, lateral deflections vary linearly with lateral load. For example, if $\frac{1}{2}$ " lateral movement is permissible, the allowable lateral load is double that tabulated; if only $\frac{1}{8}$ " lateral movement is permissible, the allowable lateral load is $\frac{1}{2}$ that tabulated.

If lateral loads are applied parallel to the pile flanges, allowable lateral loads are less than those tabulated. For loads applied parallel to the pile flanges, the lateral load required to cause $\frac{1}{4}$ " deflection in coarse-grained material is $\frac{2}{3}$ of the value

Fine-Grained Material							
Pile	Type of Soil	Minimum Embedment for "Long" Pile Ft.		Lateral Load to Cause ¼" Lateral Movement Kips		Ultimate Lateral Load Capacity Kips	Ultimate Lateral Load Capacity ÷2.5 Kips
		Short Time Load	Long Time Load	Short Time Load	Long Time Load		
HP 10 x 42	Medium Stiff	20	29	2	1	25	10
	Stiff	17	24	4	2	35	14
	Very Stiff	13	18	8	4	48	19
HP 10 x 57	Medium Stiff	22	31	3	1	30	12
	Stiff	18	26	5	2	42	17
	Very Stiff	15	19	9	4	55	22
HP 12 x 53	Medium Stiff	24	34	3	1	35	14
	Stiff	19	28	5	2	48	19
	Very Stiff	16	21	9	4	60	24
HP 12 x 74	Medium Stiff	26	37	3	1	43	17
	Stiff	21	30	6	2	58	23
	Very Stiff	17	23	10	5	73	29
HP 14 x 73	Medium Stiff	28	40	3	1	50	20
	Stiff	23	32	6	2	63	25
	Very Stiff	18	24	11	5	82	33
HP 14 x 89	Medium Stiff	29	41	4	1	55	22
	Stiff	24	34	6	2	73	29
	Very Stiff	19	26	12	5	95	38

Note: Calculated lateral loads are based upon the following soil properties:

Medium Stiff Soil: $C = 500$ Lbs./Sq. Ft.

Stiff Soil: $C = 1000$ Lbs./Sq. Ft.

Very Stiff Soil: $C = 2000$ Lbs./Sq. Ft.

FIGURE 27—LATERAL LOAD CAPACITY OF SINGLE "FREE-HEADED" PILES LOADED AT GROUND SURFACE NORMAL TO FLANGE (→|←)

tabulated; and in fine-grained material is $\frac{3}{4}$ of the value tabulated. The ultimate lateral load capacity when load is applied parallel to the pile flanges is $\frac{1}{2}$ to $\frac{3}{4}$ of the value tabulated and seldom governs design.

9. *Lateral Loads on Pile Groups:* The action of a group of piles under lateral loads differs from that of a single free-headed pile for the following reasons:

- The zone of influence of one pile interferes with the zone of influence of other piles in the group. The behavior of one pile in a group is quite different from that of other piles in the group. This effect tends to reduce the ultimate lateral capacity per pile and to increase lateral deflections.
- A large footing restrains rotation of the top of piles, causing the pile heads to be partially fixed. This effect tends to increase the ultimate lateral capacity per pile and to decrease lateral deflections.

Lateral deflections are more important than ultimate lateral resistance in the design of pile groups for highway foundations. If only the overlapping zones of influence are considered, the effect of group action is to decrease the effective value of the coefficient of horizontal subgrade reaction.

For common pile spacings of 3 or 4 pile diameters in coarse-grained material, this effect would increase the lateral deflection under a given lateral load per pile by a factor of about 2.²² However, the partial fixity of the pile head tends to counter-balance this—a fixed-headed pile in coarse-grained material deflects $1/2.67$ times that of a free-headed pile; a fixed-headed pile in fine-grained material deflects $\frac{1}{2}$ that of a free-headed pile.²¹

Limited full scale tests by Prakash²¹ indicate that the lateral load per pile required to cause a given lateral deflection is greater for pile groups than for a single free-headed pile.

Much additional research is needed before a simple design method can be established to determine the lateral load capacity of a pile group. However, since the allowable lateral load on a pile group under a highway foundation normally is governed by an arbitrarily chosen permissible lateral movement, an approximate design procedure is appropriate. The limited data available indicates that using a lateral load capacity per pile equal to the lateral load capacity of a single free-headed pile is a practical approach.

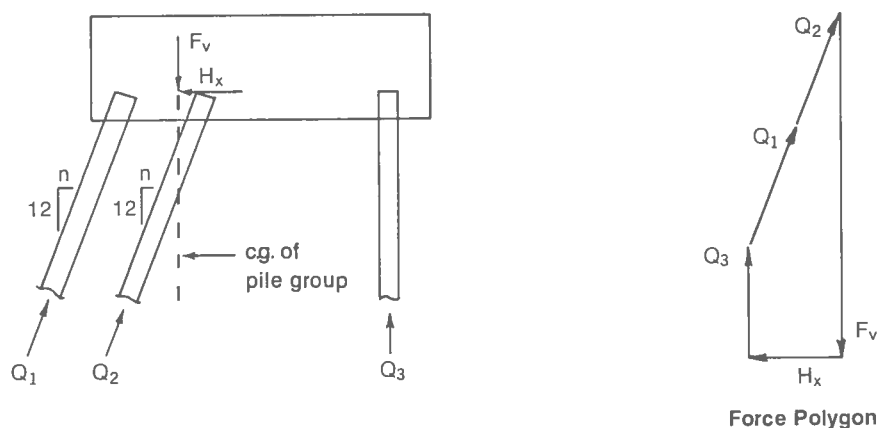


FIGURE 28—PILE BATTER SHOULD BE SET SO THAT FORCE POLYGON OF APPLIED LOADS AND PILE LOADS CLOSES

I. Batter Piles

When piles are subjected to horizontal loads exceeding allowable lateral load capacity, battered piles are provided to resist a portion of the applied horizontal load. Some designers prefer to batter enough piles to resist all horizontal loads, neglecting the lateral resistance of the piles. Common batter varies between 1 horizontal:12 vertical and 5 horizontal:12 vertical although special driving equipment is required when the batter exceeds 3 horizontal:12 vertical. It is normally assumed that a battered pile is capable of resisting the same axial load as a vertical pile of the same type and size and driven to the same stratum.

For a group of battered piles to be in equilibrium with the applied loads, it is necessary that the force polygon of the applied loads and the pile loads close as shown in Figure 28.

The following method is recommended for the design of battered piles:

- (1) Place the vertical force F_v and the horizontal force H_x on the pile group at the top of piles as shown in Figure 28.
- (2) Calculate the vertical load per pile as if all piles were vertical.
- (3) Choose a trial pile batter $n/12$.
- (4) Calculate the horizontal component of load in pile = vertical load times $n/12$.
- (5) Sum up the horizontal components in all battered piles and compare to the applied horizontal load. If the applied horizontal load less the horizontal components in all battered piles is not approximately equal to the allowable lateral load capacity of the pile group, adjust batter and/or number of piles battered.
- (6) Calculate the axial load in the battered pile = vertical load times:

$$\frac{\sqrt{(12)^2 + n^2}}{12}$$

If horizontal loads are acting on a pile group in various directions (as would be the case for a bridge pier), the horizontal loads are broken into components H_x and H_y parallel and perpendicular to the pile rows and selected piles are battered to resist each component.

J. Free-Standing Piles

Steel bearing piles are frequently extended above ground level to form frame support structures such as piers, bents and wharfs. The design of these structures is performed on the basis of standard structural design concepts. The principal problem in the design of these structures is the bending and buckling of the partially embedded piles.

In evaluating possible buckling of a partially embedded pile, it is necessary to estimate the lower condition of fixity. The term fixity as employed herein indicates restraint against rotation only. The depth of the point of fixity can be computed by the following expressions:¹⁵

(1) Coarse-grained soils:

$$\bar{D} = 1.8 \sqrt[5]{\frac{EI}{n_h}} \quad (\text{Eq. 36})$$

(2) Fine-grained soils:

$$\bar{D} = 1.4 \sqrt[4]{\frac{EI}{k}} \quad (\text{Eq. 37})$$

where \bar{D} = depth to fixity below ground surface (feet).

EI = modulus of elasticity times moment of inertia of the pile (kips feet²).

n_h = coefficient of horizontal subgrade reaction for coarse-grained soil (kips per cubic foot).

k = coefficient of horizontal subgrade reaction for fine-grained soil (kips per square foot).

The factors are illustrated and values of the coefficients given in Figure 29. If total pile embedment does not equal or exceed $3\bar{D}$, simple support must be assumed for the portion of the pile below ground.

Soil that may be removed, by natural forces or otherwise, within the lifetime of the planned structure should not be counted on for restraint. Particular care should be taken to investigate the possible scouring or erosion that may result from the presence of the planned construction, as for instance, around a bridge pier that reduces the width of a river channel. Protective measures should be employed to insure the permanence of soil vulnerable to scouring action. These may consist of dumped stone protection, woven mats of branches or some other similar materials.

The effective length to be used in the structural design of the unsupported portion of the pile should be taken equal to the distance between the structural connection and the point of fixity in the ground, multiplied by a factor depending upon support conditions and found in Section 1.8 of the Commentary on the AISC Specifications.¹⁴

Additional loads on piling are involved where piling extends through open water. In exposed locations with substantial water depths, wave, current and ice forces may assume significant values. For an estimate of wave forces refer to Quinn.¹³

An estimate of the pressures exerted on piles by flowing water may be determined from the following relation, which is a modification of that presented in AASHTO Standard Specifications for Highway Bridges, Section 1.2.17.

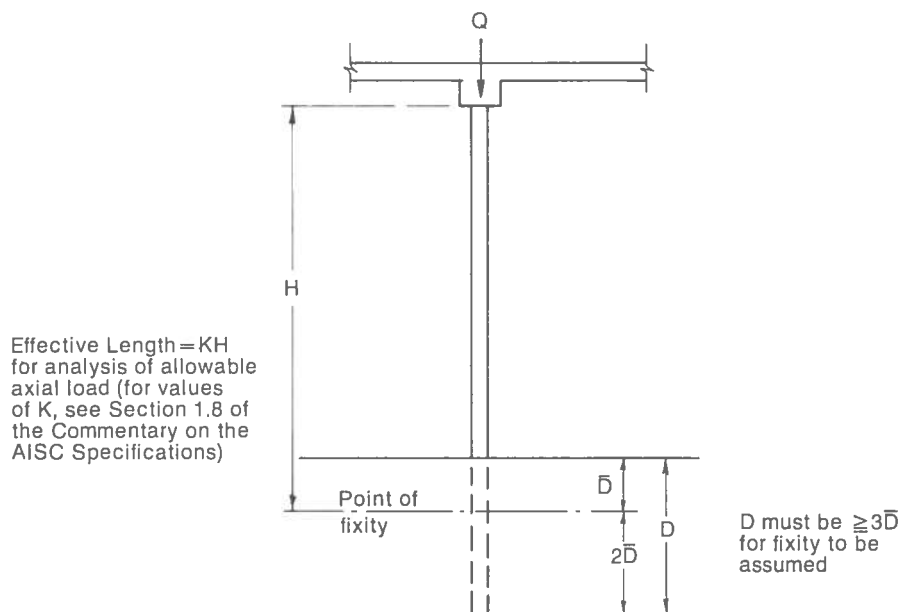
$$\sigma_w = K v_w^2 \quad (\text{Eq. 38})$$

where σ_w = pressure exerted by flowing water (psf).

K = constant depending on geometry of pile or pile groups, ordinarily taken equal to $\frac{1}{3}$.

v_w = velocity of flow (ft./sec.).

The pressure of ice on piles should be calculated at 400 psi. The thickness of ice and the height at which it applies should be determined by field investigation.



$$\bar{D} = 1.8 \sqrt[5]{\frac{EI}{n_h}} \text{ for coarse grained soils}$$

$$\bar{D} = 1.4 \sqrt[4]{\frac{EI}{k}} \text{ for fine grained soils}$$

Where E = Modulus of Elasticity } Kip
 I = Moment of Inertia of Piles } ft.
 n_h = Coefficient of Horizontal Subgrade } Units
 Reaction for Coarse Grained Soil (K/ft^3)

Coefficient n_h (Kips per cubic feet)			
Relative Density	Loose	Medium	Dense
Above Ground Water	14	42	112
Below Ground Water	8	28	68

k = Coefficient of Horizontal Subgrade
 Reaction for Fine Grained Soil $-\frac{160\bar{m}c}{\bar{b}} \left(\frac{K}{\text{ft}^2} \right)$

in which $\bar{m} = 0.32$ for $c < 1$ Ksf
 $= 0.36$ for $1 < c < 4$ Ksf
 $= 0.40$ for $c > 4$ Ksf
 \bar{b} = width of pile (ft)

FIGURE 29—POINT OF FIXITY FOR FREE STANDING PILE

V. Steel Bearing Pile Installation

A. General

Steel bearing piles are most commonly installed by driving with a hammer operated by steam, compressed air, and, more recently, hydraulic pressure. Diesel and vibratory hammers are becoming increasingly popular. Where headroom is limited, necessitating installation of piles in short lengths, or where the vibrations associated with driving cannot be tolerated, then jacks are employed to advance the piles.

It is essential that the engineer supervising pile installation have considerable experience in such work, and that the specifications for a particular project provide for means of overcoming difficulties likely to be encountered. Considerable assistance may be derived from the performance of full-scale field tests. Complete and accurate records should be compiled for the driving of each pile.

B. Handling

Steel H-piles and pipe piles do not require special care in handling unless over about 60 feet in length, or covered with a protective coating; in which case they should be supported at several points and, for H-piles, with the web vertical. Some form of head is advisable to prevent damage to the pile, particularly where hard or prolonged driving is anticipated.

C. Driving Equipment

The more common of the many types of rigs employed in pile driving are as follows: Crane with fixed leads, crane with swinging leads, and the barge mounted A-frame. Pile driver leads should be so constructed as to permit freedom of movement of the hammer, and should be held in position by guys or stiff braces to insure support to the pile during driving. Except where piles are driven through water, the leads should be of such a length as to eliminate the need for a follower. The leads should be inclined in driving batter piles.

A considerable variety of drivers is available. In the gravity or drop hammer the striking weight is raised by some mechanical means and then released for free fall. In the single-acting hammer, steam or compressed air raises the striking weight which then drops by gravity alone. The characteristics of the blow are a low striking velocity, due to the low fall and heavy striking weight. In the double-acting hammer, steam or compressed air raises the striking weight and also imparts additional energy during the down stroke. The double-acting hammer generally operates at greater speeds than a single-acting hammer of equal energy per blow and may be preferred where a considerable length of pile must be driven and where the penetration per blow is small.

In a differential acting hammer, steam or compressed air raises the striking weight and imparts additional energy during the down stroke. The speed of operation is intermediate between that of the slow single-acting hammer and the double-acting hammer. The striking weight is exceptionally heavy for the energy developed, usually comprising about one-half the total hammer weight, and strikes at low velocity. The blow frequency is affected by the resistance encountered by the pile, the resiliency of the cushioning device, and the duration of the impact period.

The diesel hammer is receiving increasingly wide usage. The hammer is a self-contained unit in which fuel is injected in controlled amounts to obtain the desired blow energy. This hammer will frequently stall under easy driving conditions and the rating of the energy per blow is difficult. However, it is convenient to handle and eliminates the necessity for steam boilers or air compressors.

Vibratory drivers are a more recent development. They comprise high amplitude vibrators and are particularly appropriate for driving through moderately compact overburden to hard strata or to reach a predetermined tip elevation. The frequency of vibration in the pile must be at least twice the natural frequency of the soil so as to prevent damage to adjacent structures.

The hydraulic hammer represents the most recent development. Hydraulic pressure is used to raise the striking weight and impart additional energy during the down stroke.

Hydraulic or mechanical screw jacks are also used to advance the pile which is usually built up in short convenient lengths. They are used where access is limited and difficult and where vibrations must be eliminated.

D. Installation

Pile locations should be carefully ascertained with stakes driven, where possible, to mark the location for each pile. The pile driving rig should have sufficiently rigid support that the leads remain accurately aligned. Where a high degree of accuracy is required, it may be necessary to erect templates or guide frames, at or close, to the ground or water surface.

The heads of steel piling should be cut squarely and a driving cap provided to hold the axis of the pile in line with the axis of the hammer. The base of the cap should

conform approximately to the shape of the pile and be loosely attached to the hammer so that it will at all times rest squarely over the entire surface of the pile head. Impact eccentricities may lead to damage of the pile head and reduces the efficiency of the driving operation.

The use of followers should be avoided where possible. Followers are employed to permit the driving of piles below the reach of the hammer or below water. They are preferably of the same material as the pile and consideration must be given to the energy loss arising from their temporary compression under the hammer blow. When followers are used, one pile from each group of 10 should be a long pile driven without a follower, and should be employed as a test pile to determine the average bearing power of the group.

All piles should be driven to the specified driving resistance within the designated bearing stratum. Driving should be continuous except when splicing is required or when attaching a follower, and must in any event be continuous during penetration within the bearing stratum.

Where the required penetration is not obtained, a more powerful hammer is generally brought in. Jetting, while generally not desirable, is often necessary as a last resort. In performing the latter, the number of jets and the volume and pressure of water at the jet nozzles should be sufficient to freely erode the material adjacent to the pile. Jetting should always be discontinued prior to final penetration. Other techniques that may be employed to reduce driving resistance in order to facilitate penetration to the designated bearing stratum are presented in Figure 30.

The soil around piles will heave occasionally when piles are driven in a group. The elevation of each pile should be repeatedly checked and the pile redriven where neces-

Method	Equipment and Procedure Utilized	Application
Temporary casing	Open-end pipe casing driven and cleaned out. May be pulled later.	a. To drive through minor obstructions. b. To minimize displacement. c. Prevent caving or squeezing of holes. d. Permit concreting of pile prior to excavation to subgrade of foundation.
Precoring	By continuous flight auger or churn drill, a hole is formed into which the pile is lowered. Pile is then driven to bearing below the cored hole.	a. For driving through thick stratum of stiff to hard clay. b. To avoid displacement and heave of surrounding soil. c. To avoid injury to timber and thin shell pipes. d. To eliminate driving resistance in strata unsuitable for bearing.
Spudding	Heavy structural sections or closed-end pipes are alternately raised and dropped to form a hole into which pile is lowered. Pile is then driven to bearing below the spudded hole.	a. For driving past individual obstruction. b. To drive through strata of fill with large boulders or rock fragments.
Jetting	Water, air or mixture of both forced through pipe at high pressures and velocity, jets are sometimes built into piles.	a. Used in practically all soils to reduce friction in strata unsuitable for bearing during driving of pile.

FIGURE 30—MEANS OF REDUCING DRIVING RESISTANCE ABOVE BEARING STRATUM

sary. In the case of spliced piles care should be taken to ascertain that heaving has not pulled apart the joints. In soils that contain concentration of boulders of such size or numbers that the driving to a specified resistance of successive piles in a group will reduce the supporting capacity of the initially driven piles, the re-establishment of the desired supporting capacity of the piles will require redriving of all piles in the group to the specified resistance. In order to ascertain whether or not any such action is required certain piles in a group should be subjected to test redriving. Open-end pipe piles are normally installed by driving, cleaning out the pipe casing, and redriving the empty shell to the specified resistance.

The redriving of a pile in fine saturated sand initially encounters greater resistance than experienced in the original driving. However, the blow count will drop off as driving continues and approach that at which driving originally ceased.

Pile heaving may be reduced by the use of wider pile spacings and by driving pile groups from the center of the group outward, as well as by the use of the several techniques for reducing driving resistance presented in Figure 30. Heave generally will be minimal for H-piles and open-end pipe piles.

Piles should be driven with a variation of not more than one-quarter inch per foot from the vertical or from the batter shown on the plans, except that piles for pile bents should be so driven that the cap may be correctly located without inducing excessive stresses in the piles. Foundation piles should not be more than 6 inches from their intended plan position.

VI. Load Tests on Steel Bearing Piles

A. General

Full-scale load tests permit accurate determination of pile behavior for use in design provided that the procedures employed in installation and loading closely simulate those anticipated for prototype construction. The principal deficiency of load testing lies in the difficulty of accounting for the time effects inherent in the behavior of fine-grained soils. For example, the skin friction in soils near the top of pile generally acts upward in a short duration load test and may carry a large part or all of the test load. However, in long-term loading, the upper strata of soils may not continue to provide support and may even apply a drag downward on the pile. Some account may be taken of the strength regain with time following pile installation in fine-grained soils by delaying field tests for at least two weeks, but it is clearly impractical to attempt to apply loads in the tests at rates simulating construction loading.

Load tests preferably should be performed on all projects of significant size, but particularly where theoretical analyses cannot clearly establish the suitability of a given design, where large friction forces are involved, where piles are to be installed by procedures other than driving, thereby precluding use of dynamic formulas, or where penetrations are controlled by strata considerations rather than some stipulated driving resistance.

Results of load tests should be interpreted with care, and particular attention must be given to any features of test behavior that might possibly restrict the suitability or capability of the piling. Extrapolation of test results on single piles to establish the behavior of groups, or even from small to large groups, is full of uncertainties, and should only be attempted in association with a detailed theoretical analysis of group action.

B. Test Pile Installation

Test piles preferably should be installed beside a completed test boring from which sufficient data has been obtained to establish the depth and characteristics of the bearing stratum and to permit a theoretical evaluation of the ultimate bearing capacity. Every effort should be made to ensure satisfaction simulation of anticipated prototype conditions. Thus, both the test piles and driving equipment should be of the type intended for use in the actual construction. If the contract piles are to be driven

from the base of an excavation, then the test piles should be installed within casing driven to the excavation level. If the piling will extend through sensitive fine-grained soil likely to exert negative skin friction or drag forces, then the test piles should be installed within the casing driven to the soil or rock underlying the sensitive soil to permit determination of pile capability within the intended bearing stratum. Full details of the pile installation process should be recorded by competent personnel.

C. Single Pile Testing

A wide variety of arrangements are available to apply vertical and lateral loads to single piles. The most common techniques for vertical compressive load application are described in ASTM Designation D-1143T and comprise the placement of dead load within a box or on a platform seated directly on the pile, or the use of calibrated jacks seated on the pile and reacting against either a heavily-loaded box or platform supported at the ground surface or a beam restrained at its end by further piling. Tension loads are best applied by jacking upwards on a beam passing through a straddle attached to the pile or by jacking the straddle directly. Lateral loads are generally applied by jacking between the test pile and either other piling or some special reaction system founded at or near the ground surface. The use of a load cell is recommended in order to verify the loads determined from the gage readings of the jack.

It is necessary to ensure that the test arrangement does not influence the test pile behavior, and that the loading is concentric, accurate, and readily adjustable. Movements of the test pile under load are generally determined by dial gages reading to 0.001 inch.

The load generally is applied in increments equal to about 25 % of the anticipated working load. Readings of each settlement measuring device should be taken immediately before the addition of each increment and afterward at $\frac{1}{2}$, 1, 2, 4, 8, 15, 30 minutes, and thereafter at 30-minute intervals. Additional loads should not be applied until the rate of settlement under the previous increment is less than 0.01 inch in one hour or until two hours have elapsed, whichever occurs first. The maximum test loading required depends on the criterion adopted for selection of allowable load capacity. AASHTO Standard Specifications for Highway Bridges, Section 2.3.6(A) stipulates that the safe compressive load be taken as 50 % of that load which, after a continuous application of 48 hours, produces a permanent settlement not greater than $\frac{1}{4}$ inch measured at the pile top. Further, this maximum settlement should not be increased by a continuous application of the test load for a period of 60 hours or longer. A more thorough evaluation of pile capability is attained by continuing loading until rapid progressive displacements occur, or, in the case of laterally loaded long piles, until the pile yields. The pile should be unloaded at several intermediate load stages, and after final loading, in order to ascertain the permanent displacements.

D. Group Testing

Economic considerations invariably necessitate that groups be loaded either vertically by the placement of dead load on the pile cap and/or laterally by jacking between the pile cap and adjoining construction. Otherwise, the test should be performed in accordance with the recommendations presented for single pile testing. Groups of piles subjected to testing ordinarily represent an integral part of the planned construction and are rarely tested to failure. Determination of the load distribution among piles in a group may be accomplished through the attachment of gages to the piles or by employing a load cell on top of each pile.

E. Test Data Interpretation

Data obtained from a test employing step-wise loading and several intermediate rebounds will yield plots of total and permanent settlement versus load similar to those shown in Figure 31. That load under which the pile undergoes rapid continuous (permanent) settlement may be considered as the ultimate capacity. The yield load,

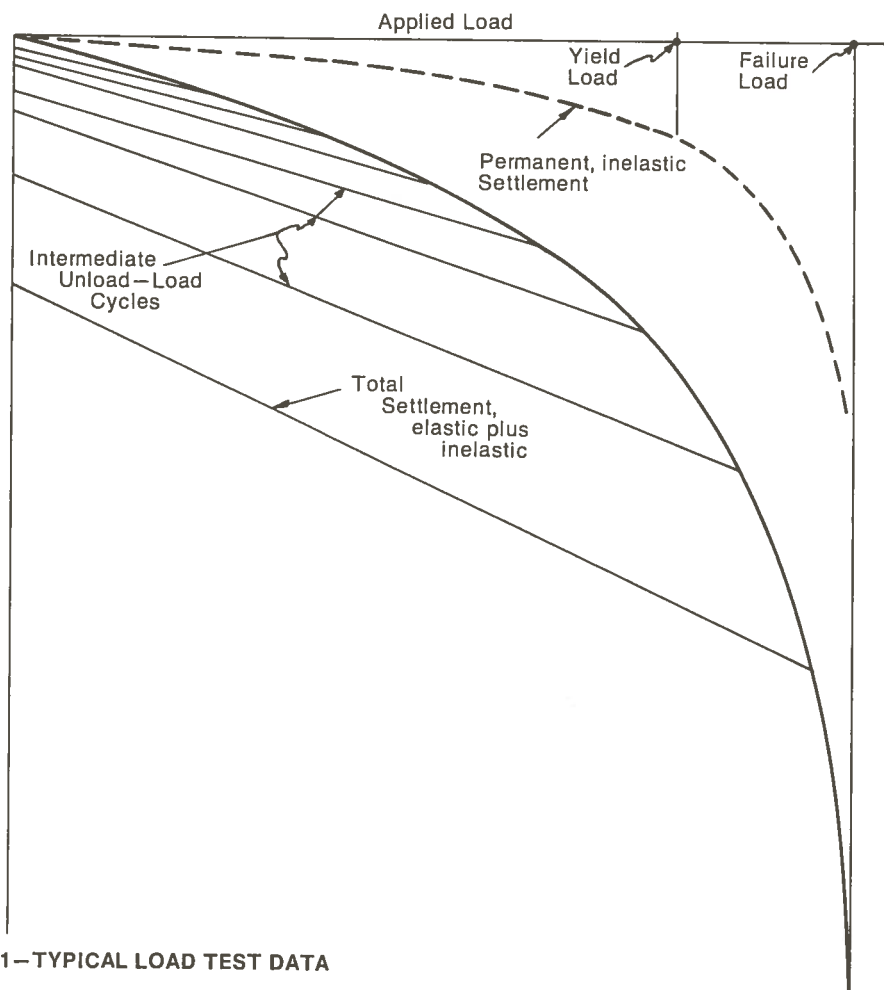


FIGURE 31—TYPICAL LOAD TEST DATA

under which the pile commences to exhibit significant continuing settlement over a long period of time may be taken as the load corresponding to the point of maximum curvature on the curve of permanent settlement versus load.

Recoverable settlements represent the combined elastic straining of the pile and adjoining soil under the prevailing load distribution. Permanent settlements are primarily attributable to inelastic straining of the soil, plus any pile buckling or distortion.

Selection of an allowable load from pile load test data is subject to many considerations. The AASHTO stipulation that the safe compressive load be taken as 50% of that load which, after a continuous application of 48 hours, produces a permanent settlement not greater than $\frac{1}{4}$ inch measured at the pile top, establishes a reasonable, if somewhat conservative, allowable compressive load capacity. In general, it is necessary that the allowable load neither approach too closely to the yield load nor incur settlements in excess of permissible values for the supported structure. The allowable load should be taken no greater than one-half of the yield load and usually not greater than one-third of the ultimate capacity. For laterally loaded long piles, where the ultimate capacity is dependent upon yielding of the pile section, the allowable lateral load should be taken no greater than one-half of the ultimate capacity, unless horizontal movements of the pile restrict allowable lateral loads to smaller values.

Allowable loads on piles in fine-grained soils are generally limited by permissible structural displacements, and it is important to note that load tests in such soils may develop only some fraction of the final settlement values within the brief duration of such testing. In coarse-grained soils, displacement generally will control the allowable capacity of axially loaded groups and laterally loaded single piles and groups. In these soils, field tests of usual duration may be assumed to yield realistic displacements.

VII. Corrosion of Steel Piles and Methods of Protection

A. General

The primary purpose of this chapter is to review the variety of corrosive conditions that steel piling might encounter in various structural applications and to describe the most widely accepted means of protection used today.

The problem of corrosion is a highly complex one largely because of the diversity of the environments which can be encountered in service. All commercial irons and steels, except the stainless steel grades, tend to corrode in the presence of air and moisture. To better understand the problem, it is necessary to briefly discuss how corrosion develops, and how best to prevent it. The corrosion of metals in aqueous environments is caused by an electrochemical reaction. Simply stated, electricity flows between metallic areas through a solution. Corrosion occurs where the electricity or current leaves the metal and enters the solution.

In the case of steel piling in marine environments, the conducting solution (the electrolyte) is sea water. One method of protecting the steel piling is to create an effective barrier to the flow of current and thus prevent deterioration. Another method is to send electric current from an external source through the electrolyte to exposed areas of the base steel. The applied protective current opposes the flow of corrosion current and when applied in adequate amounts will essentially prevent any metal loss. Protection in this manner by current flow from any source is termed cathodic protection, since the metal being protected is made the cathode of an electrolytic cell. Cathodic protection will be discussed in more detail later in the chapter.

The corrosion rate of steel is accelerated when water is thoroughly aerated. The reason for this stimulation is the presence of the electrolyte with a plentiful supply of oxygen available for the chemical reaction to take place. Corrosion, therefore, will occur more rapidly in an environment where the steel is constantly wet in the presence of excess oxygen.

Predictions of the possible life of unprotected steel piling installations are quite difficult. The active agents (oxygen, water, acids, pollutants, and saline solutions) which bring about corrosion may carry on their work under many different conditions in the same installation, and the rate at which the corrosion proceeds is determined by these conditions. Because steel piling may be partly exposed to air, partly submerged in water, and partly buried in soil, it is desirable to treat each of these conditions individually to obtain a complete understanding of the corrosion behavior of steel piling and means for preventing corrosion.

B. Atmospheric Corrosion

Steel piling installations in atmospheric environments do not usually present serious corrosion problems. The rate of atmospheric corrosion of steel is dependent on 1) the length of time that moisture is in contact with the surface, 2) the extent of contamination in the atmosphere, and 3) the chemical composition of the steel.

The results of atmospheric-corrosion tests conducted by various American Society for Testing and Materials (ASTM) technical committees^{26,27*} and other societies and associations over the years have shown the importance of the environmental factors. It is recognized from this extensive work that atmospheric sulfur dioxide, chlorides, and factors relating to the time of wetness of a corroding surface exert critical effects on the corrosivity of an atmosphere. Copson²⁸ found from his work that the corrosion rate of steel depends on the quality and quantity of water reaching the steel surface. The most corrosive condition exists when the water contains certain contaminants, particularly chlorides. Sulfur oxides are also corrosive toward steel but not nearly as aggressive as chlorides, typified by sea salt. When both chlorides and sulfur oxides are almost absent, the atmospheric corrosion of steel is not significant.

Data from tests in marine atmospheres reveal that large differences in corrosivity can exist at locations only a few hundred feet apart. The results of tests conducted both 80 and 800 feet from the ocean are given in Table I.²⁹ They show that for iron

*See References

Table I
Comparison of Corrosion Rates
80 and 800 feet from the Ocean*

Metal	Corrosion Rate, mils per year		Ratio of Rates
	80 ft from Shore	800 ft from Shore	
Iron	29.0	3.0	10
AISI Type 302 stainless steel	0.006	0.002	3

*All specimens exposed for two years at International Nickel Company's test site at Kure Beach North Carolina.

the corrosion rate 80 feet from the ocean was 10 times that obtained 800 feet from the ocean. Substantial differences in corrosivity were also observed for stainless steel.

Further evidence of the wide variations in atmospheric corrosivity that can exist along the seashore can be found in tests conducted by the ASTM at Cape Kennedy, Florida.³⁰ The atmospheric corrosivity of five locations at Cape Kennedy was determined by exposing test specimens of carbon steel 60 yards from the ocean at ground level and at elevations of 60 feet and 30 feet, as well as on the beach and one-half mile from the ocean. The results of these tests are given in Table II. It is evident that the beach site is the most corrosive location, followed by the ground elevation 60 yards from the ocean. The corrosion rates decrease with increasing elevations. The corrosivity one-half mile from the ocean appears to be slightly less than that obtained at the 60-foot elevation. It is evident that corrosion and maintenance costs for outdoor marine structures will be appreciably affected by location of the structure with regard to distance from the shore.

Table II
Corrosion Rate of Carbon Steel at Various
Locations at Cape Kennedy, Florida

Corrosion Rate, mils per year*				
60 Yards from Ocean			$\frac{1}{2}$ Mile from Ocean	Beach
Ground Level	30-ft Elevation	60-ft Elevation		
17.4	6.5	5.3	3.5	21.0

*Based on a two-year exposure except for the beach test, which was only a one-year exposure.

The third factor, the chemical composition of the steel, is as important in determining the corrosion rate of steel as are the environmental factors. Since the pioneering work of Buck³¹ and the early publications and interpretation of the results of ASTM tests³² it has been generally acknowledged that carbon steel containing 0.2 percent or more copper has twice the atmospheric-corrosion resistance of a similar steel containing only 0.01 to 0.02 percent residual copper. Tests conducted by U. S. Steel's Applied Research Laboratory³³ confirmed these findings and also showed that nickel, chromium, silicon, and phosphorus, singly, are beneficial in improving the corrosion resistance of steel. The tests further showed that the greatest improvements in corrosion resistance are obtained by the addition of specific combinations of these alloying elements. One such combination can be found in USS COR-TEN A steel and another in COR-TEN B steel. Section thicknesses used for steel piling would suggest the use of COR-TEN B steel.

The atmospheric-corrosion performance of COR-TEN B steel has been determined over the years in urban industrial, rural, and marine atmospheres. Atmospheric-corrosion tests were conducted by exposing 4- by 6-inch specimens on test racks at an angle of 30 degrees to the horizontal and facing south (the standard ASTM testing method). The specimens were exposed for 7.5 years. Plots of the average penetration in thickness (based on loss in weight) versus time for COR-TEN B steel and structural carbon steel exposed in urban industrial, semirural, and moderate marine atmospheres are shown in Figures 32, 33, and 34. From the slopes of these time-corrosion curves, it can be calculated that COR-TEN B steel is four times more corrosion-resistant than carbon steel with a low residual copper content. COR-TEN steel owes its excellent atmospheric corrosion resistance to the formation of a tightly adherent protective oxide film. The results of long-term corrosion tests have shown that the formation of the protective oxide film on COR-TEN steel is accompanied by a very low metal loss and as a result this steel can be used in the unpainted condition in most atmospheres. Thus, the use of bare COR-TEN B steel eliminates the need for original and maintenance painting. As a result, designers are now using bare USS COR-TEN B steel to economic advantage in a wide range of structures, with bridges being the most notable application at this time.

In a moderate marine atmosphere, at a distance of 800 feet from the ocean, the degree of superiority of COR-TEN B steel to carbon steel is about the same as it is in industrial atmospheres, see Figures 32 and 34. If COR-TEN B steel is exposed near the ocean where it can receive recurring wetting by salt spray, its superiority over carbon steel is maintained; however, the corrosion losses for both steels are of several orders of magnitude higher than that obtained in industrial atmospheres. As a result, these steels generally should not be used in the unpainted condition in severely corrosive marine atmospheres. There are exceptions where bare COR-TEN B steel may be suitable, however, and we suggest that for guidance on the use of bare COR-TEN steel, the U. S. Steel Corporation should be contacted.

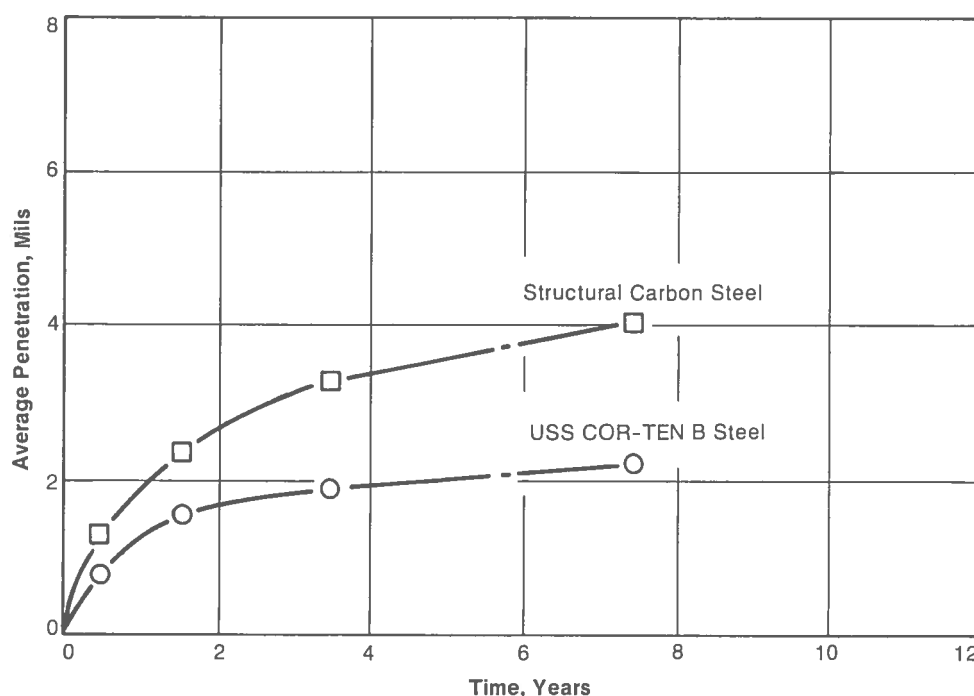


FIGURE 32—TIME-CORROSION CURVES OF USS COR-TEN B STEEL AND STRUCTURAL CARBON STEEL EXPOSED IN AN URBAN INDUSTRIAL ATMOSPHERE

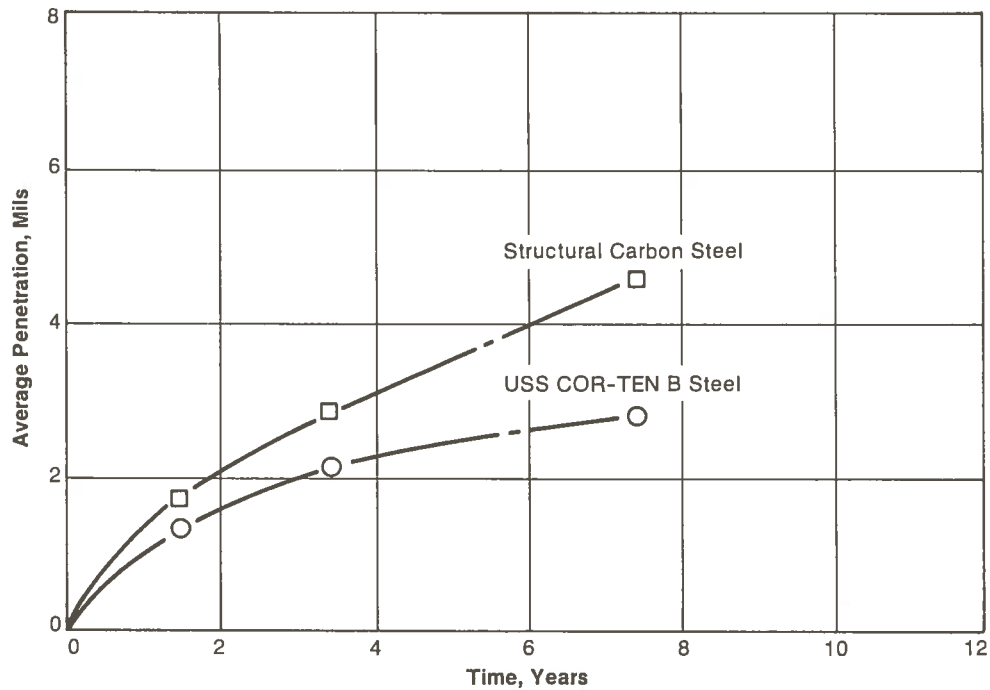


FIGURE 33—TIME-CORROSION CURVES OF USS COR-TEN B STEEL AND STRUCTURAL CARBON STEEL EXPOSED IN A SEMIRURAL ATMOSPHERE

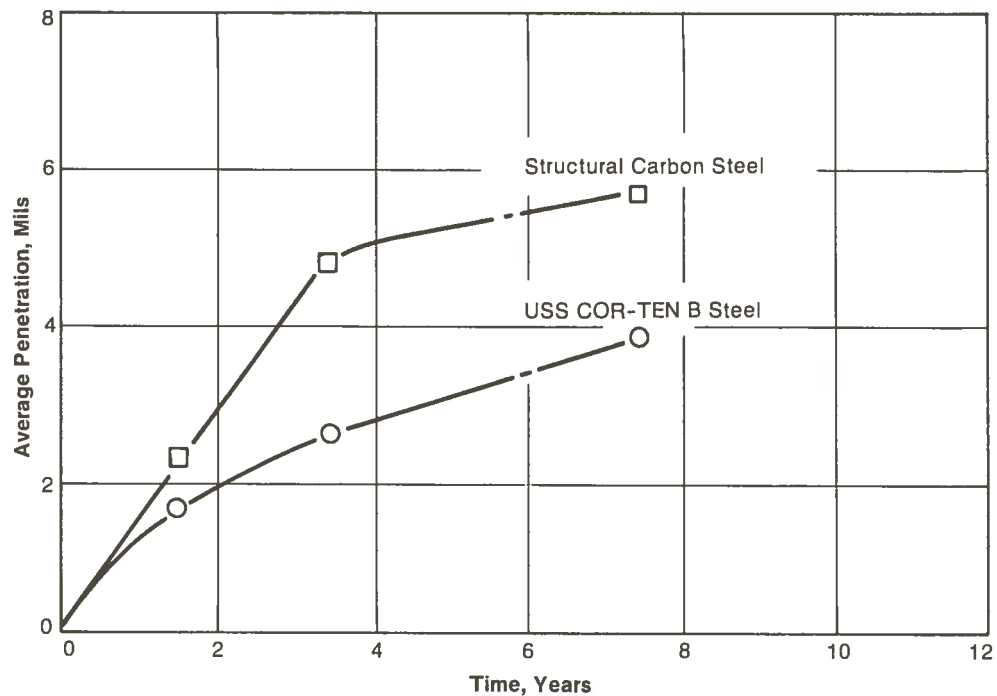


FIGURE 34—TIME-CORROSION CURVES OF USS COR-TEN B STEEL AND STRUCTURAL CARBON STEEL EXPOSED IN A MODERATE MARINE ATMOSPHERE

As will be discussed later, COR-TEN B steel offers little or no advantage from a corrosion standpoint over carbon steel when totally immersed in water or buried in the soil. Therefore, a designer may want to think in terms of a combination H-pile. For example, the section exposed to the atmosphere could be of COR-TEN B steel welded to a structural carbon-steel section that will be buried in soil or immersed in

water. Of course, the carbon-steel section should be adequately protected, depending on the corrosivity of the soil or water to which it will be exposed, see section on Methods for Preventing Corrosion. Obviously, the decision to use a composite pile section will be determined by an economic analysis wherein the ratio of the length of carbon steel to the length of COR-TEN B steel is considered. For example, if the section to be installed in the water or soil is the larger segment, then it may be economically sound to use a composite H-pile with COR-TEN B steel welded to carbon steel. However, if the section to be immersed in the water or buried in the soil is small compared to the section exposed to the atmosphere, then the use of an all COR-TEN B steel pile may be justified.

With respect to controlling corrosion of carbon-steel piling exposed to the atmosphere, there are many relatively inexpensive protective coatings that will provide practically indefinite life with little maintenance required. Both field tests and service experiences have shown that paint coatings are more durable on COR-TEN steel than on carbon steel. Any rust that forms at breaks or holidays or underneath the paint film is less voluminous on COR-TEN steel. Thus, because of the smaller volume of rust, there is less rupturing of the paint film and less moisture reaches the steel to promote further corrosion. It follows, therefore, that the extra durability of the paint is due to better atmospheric-corrosion behavior of the COR-TEN steel. For guidelines on painting steel see section on Methods for Preventing Corrosion.

C. Soil Corrosion

Data obtained by the National Bureau of Standards (NBS) on the corrosion performance of steel piles driven into the ground in a wide variety of undisturbed soil environments show the strength and useful life of steel piles are not significantly affected by corrosion.³⁴ Studies also conducted by the NBS have shown that corrosion of steel can occur in disturbed soils, such as excavations which were dry and back-filled after installation. Corrosion of steel exposed to disturbed soil takes place in the form of localized corrosion or pitting and varies from negligible to severe depending upon the type of soil environment. In piling structures the depth of scattered pitting is not particularly significant, whereas a uniform reduction in thickness from corrosion over a considerable area of structural surface is much more serious.

Corrosion in disturbed soils is attributed to a nonuniform distribution of oxygen and water along the surfaces of the buried pile section which results in the formation of oxygen-concentration cells that initiate electrochemical corrosion. Basically, the corrosion of steel is in direct proportion to the amount of dissolved or free oxygen coming in contact with the steel surface. Thus steel piling exposed to fill soil along or in the water-table zone is most vulnerable to corrosion. It is also well known that the rate of corrosion slows up materially as soon as the steel takes on a film of corrosion products, which tend to protect the steel from further corrosion. Where steel H-piles are driven in sand, conditions are particularly favorable to the formation of an impervious, insoluble coating of ferrosilicate as soon as the steel corrodes slightly. This crust or film forms an effective retardant to further corrosion or deterioration of the steel surfaces.

Generally, steel piles driven into undisturbed soil are not affected by corrosion and therefore no special protection is required for steel piles. Localized corrosion in the form of pitting may occur at the top of the pile that is in contact with disturbed soil; however, these areas can be protected with suitable paint coatings such as the coal-tar epoxys and/or cathodic protection.

In a vertical position, carbon-steel H-piles generally are spaced far enough apart so that they do not offer convenient paths for the conduction of stray electric surface currents. Hence, there is little likelihood of steel H-piles being damaged by electrolysis due to such currents. However, if it can be shown that there is a stray current situation then cathodic protection may be justified.

For guidelines on paint coatings and cathodic protection see section on Methods for Preventing Corrosion.

D. Fresh Water Corrosion

Fresh water can be defined as any river, lake, pond, or well water having a chloride content less than 1000 parts per million. Fresh waters usually have pH values close to neutral and corrosion by these waters normally proceeds at a rather slow rate. The results of tests conducted with structural carbon steel exposed to various fresh waters for a period of four years are given in Table III. For the fresh waters tested the corrosion rate of structural carbon steel ranges from about 1 to 3 mils per year (mpy). Corrosion tests conducted over a 16-year period in fresh water by Southwell and Alexander³⁵ showed a decrease of corrosion rate with time: 7.5 mpy after a 1-year exposure and about 0.7 mpy after 16 years' exposure, see Figure 35.

Table III
Corrosion Performance of Structural Carbon
Steel in Fresh Waters
4-Year Exposure

Type of Water	Corrosion Rate, mils per year	Depth of Pitting, mils	
		Max	Avg
Mississippi River	1.1	31	12
Monongahela River	3.5	21	14
Allegheny River	2.8	39	31
Spring Creek	1.8	22	15

The corrosion of steel immersed in water is a function of the availability of the dissolved oxygen at the metal-water interface. Thus, differences in oxygen concentration at different areas on the steel surfaces can lead to localized corrosion in the form of pitting. This is evident from the data in Table III where the maximum depth of pitting for structural carbon steel exposed for 4 years in fresh water ranged from 21 to 39 mils depending on type of water. However, the pitting rate of steel exposed in fresh water also decreases with time as in the case with the general corrosion rate of steel, which was discussed above. This can be seen in Figure 35, 16-year studies of Southwell and Alexander, where pitting penetration of the carbon steel was high initially but eventually decreased to a final rate approximately equal to the general corrosion rate.

In most applications where steel piles are in contact with fresh water, natural-forming protective coatings form on the steel surface and corrosion protection such as protective coatings or cathodic protection are not usually required. However, there may be cases where the water is contaminated, for example, by sewage or leachings from coal (see discussion below on polluted water) and serious corrosion of the steel may occur. Under these conditions, protective coatings and/or cathodic protection can be employed to prevent corrosion. Another accelerating factor is water velocity. Under conditions of high water velocity, it may be necessary to encase the steel to the mudline to minimize erosion of the steel. This is particularly true if sand or abrasive silt particles become entrained in fast-moving waters.

The corrosion-resistant low-alloy steels, such as COR-TEN B steel, were developed primarily for atmospheric corrosion resistance. However, the superior corrosion resistance shown by these steels in various atmospheres is not always observed in aqueous environments. Therefore, the low-alloy steels should not be depended upon to have improved corrosion resistance over that of carbon steel when exposed to water. To obtain a significant improvement in corrosion resistance to water over that of carbon steel, it is necessary to add a minimum of 12 percent chromium to steel.

For guidelines on paint coatings, cathodic protection and concrete encasement see section on Methods for Preventing Corrosion.

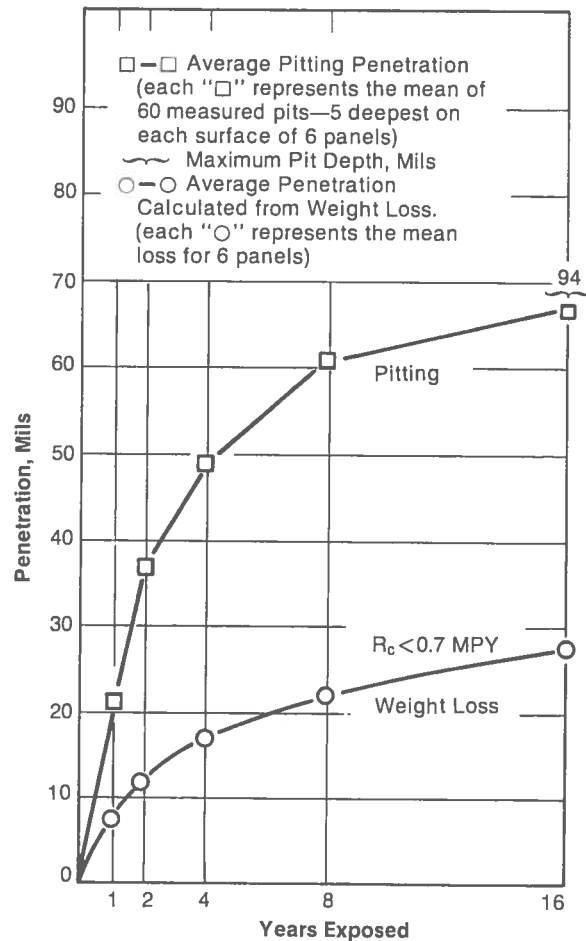


FIGURE 35—CORROSION OF CARBON STEEL CONTINUOUSLY IMMERSSED IN FRESH WATER. (COURTESY OF U.S. NAVAL RESEARCH LABORATORY)

E. Polluted Water Corrosion

The nature of the suspended solid material found in fresh water polluted with domestic sewage or organic wastes is such that the deposits which collect on steel are not always protective and thus may lead to localized corrosion or pitting. As discussed above, the corrosion of steel in water is a function of the availability of the dissolved oxygen at the metal-water interface. Therefore, because the organic matter in sewage can consume oxygen, corrosion of steel exposed to these waters usually is not a problem. However, if the pollution is of sufficient intensity to result in anaerobic conditions, the activity of the bacteria may result in the production of hydrogen sulfide and to the formation of sulfates. As a result, these waters can become highly aggressive toward carbon steel. Under these conditions, steps should be taken to protect steel H-piles with suitable coatings such as the coal-tar epoxys and/or cathodic protection.

If pollution of the fresh water takes the form of leachings from ore and coal then these waters will be appreciably more corrosive toward carbon steel than neutral fresh water as a result of the acidic nature of the leachings. These waters are usually referred to as "acid waters" as they contain appreciable quantities of free sulfuric acid because of the leaching action of the water on the sulfides contained in coal or ore. Steel piling exposed to these waters will be corroded rapidly and should be properly protected through the use of paint coatings and/or cathodic protection.

For guidelines on paint coatings or cathodic protection see section on Methods for Preventing Corrosion.

F. Sea Water Corrosion

Corrosion of carbon-steel piling installed in sea water is most severe in the splash zone, the area from mean tide to the upper limit of wave action. The attack on steels immersed in sea water is a function of the availability of the dissolved oxygen at the metal-water interface. Inasmuch as the splash zone of piling is alternately wet and dry, there is a plentiful supply of oxygen available for the corrosion process. Between mean tide and low tide, the steel is generally subject to less severe corrosion than in the splash zone. This is due to the decreased oxygen supply resulting from periodic submergence of the steel, and apparently to the fact that the submerged steel provides some cathodic protection to the steel in the tidal zone.³⁶

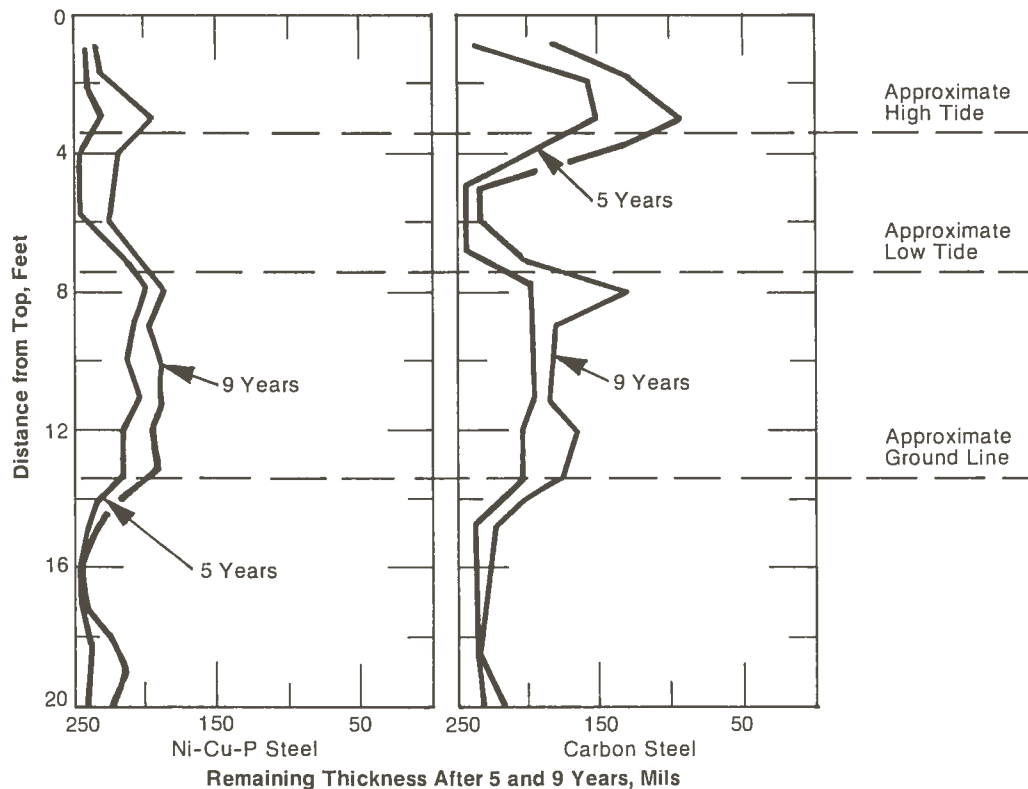


FIGURE 36—COMPARATIVE CORROSION OF USS MARINER STEEL AND CARBON STEEL IN MARINE ENVIRONMENT. TEST STRIPS EXPOSED FOR 5 AND 9 YEARS AT WRIGHTSVILLE BEACH, NORTH CAROLINA.

Cathodic protection, which is an effective method of controlling corrosion of steel piling immersed in sea water, will not extend to the splash zone since this area does not continually stay wet. In the past, the means available for obtaining longer life from steel in the splash zone were the use of thicker steel sections, reinforced concrete jacketing, metal jacketing, or coating systems. Recognizing the need for a piling steel with improved splash-zone corrosion resistance, U. S. Steel Corporation conducted a series of corrosion tests with certain experimental steels. For this study $\frac{1}{4}$ -inch by 6-inch wide by 20-foot long specimens of carbon steel and experimental steels were exposed on a dock at Wrightsville Beach, North Carolina, so that their tops extended above the "splash zone" and their bottoms were in the mud. The results obtained after 5 and 8 years' exposures³⁷ showed that a steel containing 0.5 percent Ni, 0.5 percent Cu, and 0.12 percent P (presently marketed under the tradename USS MARINER Steel) was appreciably more resistant than carbon steel to corrosion in the splash and tidal zones, see Figure 36.

To confirm the earlier findings and to obtain longer-term data on USS MARINER Steel, tests were conducted at 14 different coastal sites in the United States. The corrosion data obtained from these studies, which ranged from 1 to 10 years' duration, confirm the results of the previous tests that showed USS MARINER Steel to be at

least twice as corrosion resistant as carbon steel in the splash zone. Because MARINER steel offers little or no advantage from a corrosion standpoint over carbon steel when totally immersed in water or buried in the mud, a designer may want to think in terms of composite H-pile. For example, the section exposed to the atmosphere and salt spray could be of MARINER steel welded to a structural carbon-steel section that would be exposed to the sea water and buried in the mud. For a compatible system, the structural carbon steel should be adequately protected in the immersed zone in order to have a life expectancy equal to the MARINER steel section. As discussed earlier, the decision to use a composite H-pile must be based on an economic analysis wherein the length of pile exposed to the atmosphere and splash zone is compared with the length exposed to water and buried in the soil.

For guidelines on paint coatings, cathodic protection, and concrete encasement see section on Methods for Preventing Corrosion.

The corrosion rate most commonly used for carbon steel in quiescent sea water is 5 mils per year (mpy), which is generally considered to be linear with time.³⁸ However, these earlier data were based on relatively short-term tests, and longer-term data are now available which show that the corrosion rate of steel actually decreases to values below 5 mpy with time. Carefully controlled tests conducted over a 16-year period by Southwell and Alexander,³⁹ showed a decrease of corrosion rate with time: 5.8 mpy after a one-year exposure and 2.7 mpy after 16 years' exposure, see Figure 37. Larra-bee,⁴⁰ in conducting an investigation of carbon-steel piles immersed in sea water for 23.6 years, concluded that steel has a low corrosion rate during long sea water exposure. From a study of some 20 piles, he reported that the corrosion rate for carbon steel in sea water averages 2 mpy for the first 20 years, then drops to 1 mpy.

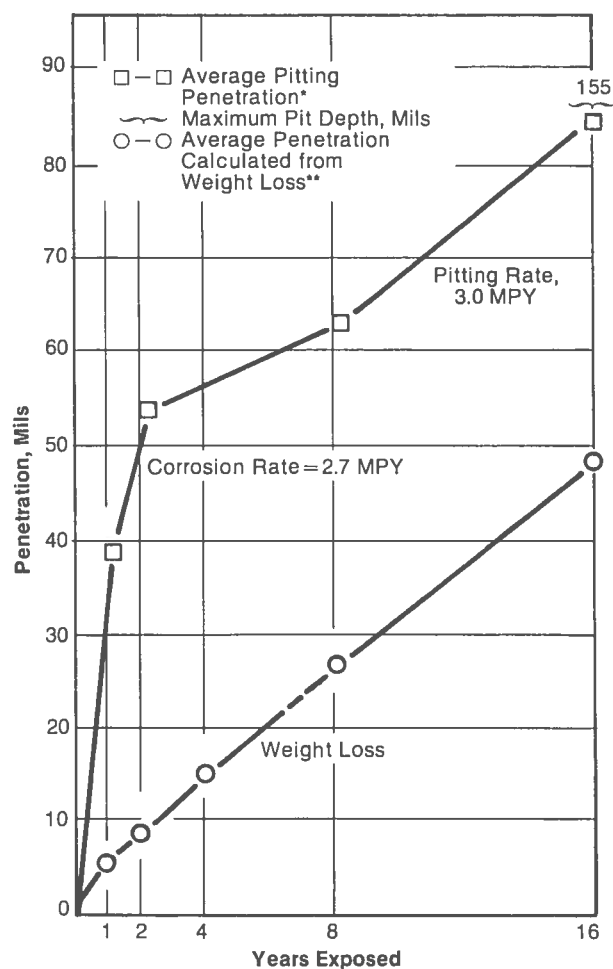
As mentioned earlier, the corrosion of steels immersed in sea water is a function of the availability of the dissolved oxygen at the metal-water interface. Therefore, as might be anticipated, increasing flow velocity raises the corrosion rate because it increases the amount of oxygen diffusing to the steel surface. As an example of the effect of flow rate on corrosion rate, Kirk, et al.⁴¹ reported corrosion rates for mild steel after a 1-year exposure of 4 mpy under quiescent conditions and 9 mpy at a flow rate of 2 feet per second. Another important factor is that differences in oxygen concentration at different parts of the steel surface can lead to localized corrosion. For example, if there are small areas where the soluble oxygen is deterred from reaching the metal as a result of slow diffusion through rust and large areas exist where the oxygen remains in ready contact with the steel, pitting attack will result at the areas where there is a deficiency of oxygen. The initial rate of pitting of steel in sea water is usually found to be several times higher than the average rate of penetration of steel. However, long-term studies of 16 years' duration⁴² show that after 2 to 4 years' exposure there is an appreciable decrease in the pitting rate of steel, see Figure 37. In fact, after about 8 years, the pitting curve in Figure 37 parallels the weight-loss curve and the average pitting penetration over long periods is about 3 mpy.

To obtain reasonably long life of a structure immersed in sea water, it is desirable to protect the steel H-piles either through the use of good paint coatings and/or cathodic protection or encasement in concrete. Further, as discussed above, high water velocity can increase the corrosion of steel piling by removing protective films that form on the steel surface and thus result in serious corrosion to the structure. This problem is further aggravated when sand or abrasive particles such as sea shells become entrained in the water. Under these conditions the steel H-piles should be encased in concrete below the mudline.

For guidelines on paint coatings, cathodic protection, and concrete encasement see section on Methods for Preventing Corrosion.

G. Methods For Preventing Corrosion

1. *General:* To obtain reasonable longevity with steel H-pile installations, particularly in marine installations, protection should ordinarily be provided for USS Steel H-piles. Failure to do so may lead to deterioration of the structure and unnecessary



*Each "□" represents the mean of 60 measured pits—5 deepest on each surface of 6 panels

**Each "○" represents the mean loss for 6 panels

FIGURE 37—CORROSION OF CARBON STEEL CONTINUOUSLY IMMERSSED IN SEA WATER (COURTESY OF U.S. NAVAL RESEARCH LABORATORY)

expenditures for repairs. Choice of protection is ultimately based on economics. In general, four types of protection are available; namely, cathodic protection, paint coatings, metal sheathing, and concrete encasement. Usually, more than one type of protection will be used on a structure for most economical, adequate protection. Although it is not classified as a method of protection, the use of heavier steel pile sections for a given application should also be considered as a means of increasing the service life of a structure. Depending upon the application and rate of corrosion, it may be less expensive to use additional steel thickness rather than to employ expensive protection methods.

Cathodic protection is usually low in initial cost and low in maintenance. It can be of value only where the piles are continually wet, as in the submerged zone.

Paint protection is usually low in initial cost but may require relatively frequent maintenance. Also, in the case of marine structures, it is extremely difficult to renew in the tidal zone between mean tide and low tide.

Concrete encasement or metal jacketing are relatively expensive in initial cost but require no maintenance if properly constructed. They may be readily applied to the full length of steel H-piles. A brief description of each type of protection follows.

2. *Organic and Metallic Coatings:* Organic coatings are probably the most widely used means of protecting steel against corrosion in sea water. The tremendous growth

of the chemical industry, along with the need for improved coatings, has led to some outstanding developments in the organic coating field. The various types of coatings are too numerous to discuss in a chapter of this type. The generic classes of coatings that have performed well in protecting steel include vinyls, epoxies, urethanes, and coal-tar epoxies.

The Steel Structures Painting Council, 4400 Fifth Avenue, Pittsburgh, Pennsylvania, 15213, published Volume 2, "Systems and Specifications," to fulfill the need for specific recommendations for the painting of various types of steel structures. The specific recommendations in Volume 2 are a supplement to the General Recommendations of Good Practice which is found in Volume 1 published by the Steel Structures Painting Council.

Volume 2 consists, mainly, of specifications for surface preparation, pre-treatment, paint application, paints and complete paint systems, including the Steel Structures Painting Council's specification numbers. The 9 basic paint system specifications included in this volume include approximately 40 primer, 9 surface preparations, 4 pre-treatments, and many intermediate and finish coats. These paint systems are flexible enough to cover almost every painting problem which will normally be encountered. Six of the 10 classifications listed in the "General Painting Guide for Steel Structures" (see Table I of Vol. 2) will apply to the conditions under which any proposed steel structure or structures will be subjected. Each classification outlines surfaces to be painted, recommendations, and comments. Suggested paint system specifications and specification numbers are recommended.

By far, the most important item in obtaining good protection of metal by a paint is the proper preparation of the metal surface before the coating is applied. All loose scale, preferably all scale, should be removed by suitable mechanical means, such as sandblasting to a degree adequate for the type coating to be applied. For best results, painting should be done in a dry atmosphere on metal which is preferably warm. It should never be done when the metal is cold or wet; such conditions will materially lessen the protective life of any paint or other nonmetallic coating.

Regardless of type of coating and system finally adopted, it is important that materials be purchased from a properly equipped manufacturer who has had prior experience in manufacturing the selected coating and who can cite structures, other than sample panels, painted with a coating of this character, on which satisfactory service has been rendered over a period of years.

Metallized aluminum coatings to protect steel in sea water have been in use for over a decade and have established an excellent performance record. About 6 to 12 mils of aluminum are sprayed on the steel surface which is first blastcleaned to white metal. The American Welding Society reports⁴³ that steel panels metallized with 6 mils or more of aluminum and sealed with a clear vinyl coating are in excellent condition after 12 years' exposure to sea water.

3. *Cathodic Protection*: Cathodic protection is an effective method of controlling the corrosion of steel piling immersed in water, particularly sea water. Depending on the type and location of the structure, cathodic protection may be achieved either by a galvanic system using zinc or magnesium anodes or by an impressed current system employing a rectifier and perhaps graphite, lead-silver, or platinum anodes. The need for cathodic protection should be based on the actual performance of the structure, which can be determined by periodic inspection. It is always desirable in the construction of steel structures to tie the unit together electrically so that if cathodic protection is needed sometime during the life of the structure it can be applied quickly and at low cost. In some highly aggressive warm water or polluted sea water, it may be necessary to apply cathodic protection at the time the steel structure is installed in order to obtain long-term service.

Either type of cathodic protection (sacrificial anodes or compressed current) can be employed; the only essential is that a sufficient cathodic current density be maintained at all areas of the protected metal which contacts the corrosive solution. Determining the magnitude of the current density just sufficient to prevent corrosion

under various types of service conditions is a difficult problem. Fortunately, for most ferrous structures, it is not necessary to know the minimum current density with great accuracy. Any cathodic current density which is applied to a given area will reduce corrosion. Further, if the minimum current density is exceeded, no harm is done to the structure. This means, simply, that protection is costing more than necessary because some of the current is being wasted. Usually, current consumption can be sharply reduced after the structure has been installed for a relatively short time.

On coated structures where the coating remains tightly adherent, the amount of current applied to protect the structure will be less than that required for an uncoated structure. This will result in a savings in power cost and anode-replacement costs. However, on structures where the coating deteriorates relatively rapidly the initial current requirements will increase.

Cathodic protection is a very effective method of preventing corrosion by most types of electrically conductive media. Also, it is relatively safe, since as long as all the current flows to the structure, corrosion will be reduced even if protection is not complete. It is relatively easy to apply cathodic protection to small, geometrically-simple structures. However, skilled corrosion engineers are required to develop efficient cathodic protection for large or geometrically-complicated structures.

Other protective means are required in conjunction with cathodic protection, since the latter can only be considered effective in areas where the electrolytic solution is continuously present. Normally, cathodic protection is considered effective below mean tide level, and should be applied to all surfaces of the piles if need for protection is indicated by contamination of water or by periodic examination of the completed structure. No cathodic protection system should be designed or installed without competent engineering advice.

4. *Metal Sheathing*: As mentioned earlier, corrosion of steel is most severe in the splash zone, and cathodic protection is ineffective in controlling corrosion in this area. Metal sheathing to protect new and existing structures is an effective method of combating splash-zone corrosion. The U. S. Coast Guard has used heavy iron plates to protect the legs of fixed offshore towers along the Atlantic seaboard. One popular protection method is the use of Monel sheathing because of its excellent resistance to sea-water corrosion. Stainless steel also has received attention as sheathing. When either Monel or stainless is used as sheathing, a sheet of insulating material should be placed between the metallic sheath and the carbon steel to prevent galvanic corrosion.

5. *Reinforced Concrete Protection*: Another important way in which protection can be achieved is to encase the structure with reinforced concrete effectively bonded to the steel. Concrete encasement will extend the service life of the structure by giving excellent protection from corrosion. However, to protect the piles adequately, the proper type of concrete must be used. If the concrete is porous, deterioration of the piles will occur. Corrosion will take place, causing the concrete to crack and spall. The primary remedy, of course, is to provide a dense impermeable concrete with adequate thickness.

In the case where steel H-piles or open-end pipe piles will be exposed to sea water, it is recommended that the piles be entirely encased from the atmospheric area to below the mudline. This is particularly true if the installation will be exposed to fast-moving water that may contain entrained sand or abrasive particles.

6. *Conclusions*: In view of the foregoing, the United States Steel Corporation advised all persons contemplating installation of steel pile structures in fresh and sea water to give thorough consideration to the following:

- (a) If fresh water is contaminated with sewage or corrosive pollutants so as to be harmful to steel piles, provide protection to the immersed areas through the use of organic coatings and/or cathodic protection.
- (b) If no contamination exists in fresh water, provide for future cathodic protection in permanently wet areas while structure is being built by tying the units together electrically.

- (c) If exposed to sea water provide complete protection to the piles. This can be accomplished through the use of organic and metallic coatings, cathodic protection, encasement in concrete or combinations of these systems.
- (d) It is good practice to provide for periodic inspection of all steel piles to determine their condition and to establish whether additional protection is required.

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