# SECTION G
# FOUNDATION DESIGN

The recommended criteria of Section G.1 through Section G.11 are devoted to pile foundations, and more specifically to steel cylindrical (pipe) pile foundations. An introduction to Design of Shallow Foundations is given in Section G.12 and Section G.13, and specifics in preliminary form are presented in Comm. G.13 while recommendations are under development.

## G.1 GENERAL

The foundation should be designed to carry static, cyclic, and transient loads without excessive deformations or vibrations in the platform. Special attention should be given to the effects of cyclic and transient loading on the strength of the supporting soils as well as on the structural response of piles. Guidance provided in Sections G.3, G.4, and G.5 is based upon static, monotonic loadings. Furthermore, this guidance does not necessarily apply to so called problem soils such as carbonate material or volcanic sands or highly sensitive clays. The possibility of movement of the seafloor against the foundation members should be investigated and the forces caused by such movements, if anticipated, should be considered in the design.

## G.2 PILE FOUNDATIONS

Types of pile foundations used to support offshore structures are as follows:

**G.2.1 Driven Piles.** Open ended piles are commonly used in foundations for offshore platforms. These piles are usually driven into the seafloor with impact hammers which use steam, diesel fuel, or hydraulic power as the source of energy. The pipe wall thickness should be adequate to resist axial and lateral loads as well as the stresses during pile driving. It is possible to predict approximately the stresses during pile driving using the principles of one-dimensional elastic stress wave transmission by carefully selecting the parameters that govern the behavior of the soil, pile, cushions, capblock, and hammer. For a more detailed study of these principles refer to Reference G1. The above approach may also be used to optimize the pile-hammer-cushion and capblock with the aid of computer analyses (commonly known as the Wave Equation Analyses).

The design penetration of driven piles should be determined in accordance with the principles outlined in Sections G.3 through G.6 and Section G.9 rather than upon any correlation of pile capacity with the number of blows required to drive the pile a certain distance into the seafloor.

When a pile refuses before it reaches design penetration, one or more of the following actions can be taken:

(a) **Review of hammer performance.** A review of all aspects of hammer performance, possibly with the aid of hammer and pile head instrumentation, may identify problems which can be solved by improved hammer operation and maintenance, or by the use of a more powerful hammer.

(b) **Reevaluation of design penetration.** Reconsideration of loads, deformations and required capacities, of both individual piles and other foundation elements, and the foundation as a whole, may identify reserve capacity available. An interpretation of driving records in conjunction with instrumentation mentioned above may allow design soil parameters or stratification to be revised and pile capacity to be increased.

(c) **Modifications to piling procedures.** Modifying procedures, usually the last course action, may include one of the following:

1. **Plug Removal.** The soil plug inside the pile is removed by jetting and air lifting or by drilling to reduce pile driving resistance. If plug removal results in inadequate pile capacities, the removed soil plug should be replaced by a grout or concrete plug having sufficient load-carrying capacity to replace that of the removed soil plug. Attention should be paid to plug/pile load transfer characteristics. Plug removal may not be effective in some circumstances particularly in cohesive soils.

2. **Soil Removal Below Pile Tip.** Soil below the pile tip is removed either by drilling an undersized hole or by jetting and possibly air lifting. The drilling or jetting equipment is lowered through the pile which acts as the casing pipe for the operation. The effect on pile capacity of drilling an undersized hole is unpredictable unless there has been previous experience under similar conditions. Jetting below the pile tip should in general be avoided because of the unpredictability of the results.

3. **Two-Stage Driven Piles.** A first stage or outer pile is driven to a predetermined depth, the soil plug is removed, and a second stage or inner pile is driven inside the first stage pile. The annulus between the two piles is grouted to permit load transfer and develop composite action.

4. **Drilled and Grouted Insert Piles.** Refer to Comm. G.2.2(2).

**G.2.2 Drilled and Grouted Piles.** Drilled and grouted piles can be used in soils which will hold an open hole with or without drilling mud. Load transfer between grout and pile should be designed in accordance with Sections H.4.2, H.4.3, and H.4.4.

**G.2.3 Belled Piles.** Bells may be constructed at the tip of piles to give increased bearing and uplift capacity through direct bearing on the soil. Drilling of the bell is carried out through the pile by underreaming with an expander tool. A pilot hole may be drilled below the bell to act as a sump for unrecoverable cuttings. The bell and pile are filled with concrete to a height sufficient to develop necessary load transfer between the bell and the pile. Bells are connected to the pile to transfer full uplift and bearing loads using steel reinforcing such as structural members with adequate shear lugs, deformed reinforcement bars or pre-stressed tendons. Load transfer into the concrete should be designed in accordance with ACI 318. The concrete and reinforcing steel requirements should be determined by using the ACI 318 nominal strength equations and resistance factors in conjunction with the load conditions and corresponding load factors from these RP2A guidelines. The steel reinforcing should be enclosed for their full length below the pile with spiral reinforcement meeting the requirements of ACI 318. Load transfer between the concrete and the pile should be designed in accordance with Sections H.4.2, H.4.3, and H.4.4.

## G.3 PILE DESIGN

**G.3.1 Foundation Size.** When sizing a pile foundation, the following items should be considered: diameter, penetration, wall thickness, type of tip, spacing, number of piles, geometry, location, mudline restraint, material strength, installation method, and other parameters as may be considered appropriate.

**G.3.2 Foundation Response.** A number of different analysis procedures may be utilized to determine the requirements of a foundation. At a minimum, the procedure used should properly simulate the nonlinear response behavior of the soil and assure load-deflection compatibility between the structure and the pile-soil system.

**G.3.3 Deflections and Rotations.** Deflections and rotations of individual piles and the total foundation system should be checked at all critical locations which may include pile tops, points of contraflecture, mudline, etc. Deflections and rotations should not exceed serviceability limits which would render the structure inadequate for its intended function.

**G.3.4 Foundation Capacity.**

1. Pile Strength: The pile strength should be verified using the steel tubular strength checking equations given in Section D.3. for conditions of combined axial load and bending. Internal pile loads at the location being checked should be those caused by the factored loads using a coupled structure/soil nonlinear foundation model. When lateral restraint normally provided by the soil is inadequate or nonexistent, column buckling effects on the pile should also be checked as defined in Section G.10.2.

2. Pile Axial Resistance: The axial pile capacity should satisfy the following conditions.

$$P_{DE} \leq \Phi_{PE}\, Q_D \qquad \text{(G.3-1)}$$

$$P_{DO} \leq \Phi_{PO}\, Q_D \qquad \text{(G.3-2)}$$

where:

$Q_D$ = ultimate axial pile capacity as determined in Sections G.4 and G.5.

$P_{DE}$ (or $P_{DO}$) = axial pile load for extreme (or operating) environmental conditions determined from a coupled linear structure and nonlinear foundation model using factored loads.

$\Phi_{PE}$ = pile resistance factor for extreme environmental conditions (= 0.8)

$\Phi_{PO}$ = pile resistance factor for operating environmental conditions (= 0.7)

**G.3.5 Scour.** Seabed scour affects both lateral and axial pile performance and capacity. Scour prediction remains an uncertain art. Sediment transport studies may assist in defining scour design criteria but local experience is the best guide. The uncertainty on design criteria should be handled by robust design or by an operating strategy of monitoring and remediation as needed. Typical remediation experience is documented in References G94 and G95. Scour design criteria will usually be a combination of local and global scour.

## G.4 PILE CAPACITY FOR AXIAL BEARING LOADS

**G.4.1 Ultimate Bearing Capacity.** The ultimate bearing capacity of piles, $Q_D$, including belled piles, should be determined by the equation:

$$Q_D = Q_f + Q_p = f\,A_s + q\,A_p \qquad \text{(G.4-1)}$$

where:

$Q_f$ = skin friction resistance, in force units

$Q_p$ = total end bearing, in force units

$f$ = unit skin friction capacity, in stress units

$A_s$ = side surface area of pile

$q$ = unit end bearing capacity, in stress units

$A_p$ = gross end area of pile

Total end bearing, $Q_p$, should not exceed the capacity of the internal plug. In computing pile loading and capacity, the weight of the pile-soil plug system and hydrostatic uplift should be considered.

In determining the load capacity of a pile, consideration should be given to the relative deformations between the soil and the pile as well as the compressibility of the soil-pile system. Equation G.4-1 assumes that the maximum skin friction along the pile and the maximum end bearing are mobilized simultaneously. However, the ultimate skin friction increments along the pile are not necessarily directly additive, nor is the ultimate end bearing necessarily additive to the ultimate skin friction. In some circumstances this effect may result in the capacity being less than that given by equation

G.4-1. In such cases a more explicit consideration of axial pile performance effects on pile capacity may be warranted. For additional discussion of these effects refer to Section G.6 and References G2, G88 and G89.

The foundation configurations should be based on those that experience has shown can be installed consistently, practically and economically under similar conditions with the pile size and installation equipment being used. Alternatives for possible remedial action in the event design objectives cannot be obtained during installation should also be investigated and defined prior to construction.

For the pile-bell system, the capacity check should be according to that given in Section G.3.4. The ultimate skin friction values on the pile section should be those given in this section and in Section G.5. Skin friction on the upper bell surface and possibly above the bell on the pile should be discounted in computing skin friction resistance, $Q_f$. The end bearing area of a pilot hole, if drilled, should be discounted in computing total bearing area of the bell.

**G.4.2 Skin Friction and End Bearing in Cohesive Soils.** For pipe piles in cohesive soils, the shaft friction, f in stress units, at any point along the pile may be calculated by the equation:

$$f = \alpha c \qquad \text{(G.4-2)}$$

where:

$\alpha$ = a dimensionless factor

c = undrained shear strength of the soil (in stress units) at the point in question

The factor, $\alpha$, can be computed by the equations:

$$\alpha = 0.5\Psi^{-0.5} \quad \Psi \leq 1.0 \qquad \text{(G.4-3)}$$

$$\alpha = 0.5\Psi^{-0.25} \quad \Psi > 1.0.$$

with the constraint that $\alpha \leq 1.0$.

where:

$\Psi$ = $c/p_o'$ for the point in question

$p_o'$ = effective overburden pressure at the point in question

A discussion of appropriate methods for determining the undrained shear strength, c, and effective overburden pressure, $p_o'$, including the effects of various sampling and testing procedures is included in the commentary. For underconsolidated clays (clays with excess pore pressures undergoing active consolidation), $\alpha$ can usually be taken as 1.0. Due to the lack of pile load tests in soils having $c/p_o'$ ratios greater than three, Equation G.4-3 should be applied with considerable care for high $c/p_o'$ values. Similar judgment should be applied for deep penetrating piles in soils with high undrained shear strength, c, where the computed shaft frictions, f, using Equation G.4-2 above, are generally higher than previously specified in the RP2A.

For very long piles some reduction in capacity may be warranted, particularly where the shaft friction may degrade to some lesser residual value on continued displacement. This effect is discussed in more detail in the commentary.

Alternative means of determining pile capacity that are based on sound engineering principles and are consistent with industry experience are permissible. A more detailed discussion of alternative prediction methods is included in the commentary.

For piles end bearing in cohesive soils, the unit end bearing, q, in stress units, may be computed by the equation

$$q = 9c \qquad \text{(G.4-4)}$$

The shaft friction, f, acts on both the inside and outside of the pile. The total resistance is the sum of the external shaft friction, the end bearing on the pile wall annulus, and the total internal shaft friction or the end bearing of the plug, whichever is less. For piles considered to be plugged, the bearing pressure may be assumed to act over the entire cross section of the pile. For unplugged piles, the bearing pressure acts on the pile wall annulus only. Whether a pile is considered plugged or unplugged may be based on static calculations. For example, a pile could be driven in an unplugged condition but act plugged under static loading.

For piles driven in undersized drilled holes, piles jetted in place, or piles drilled and grouted in place, the selection of shaft friction values should take into account the soil disturbance resulting from installation. In general, f, should not exceed values for driven piles, however, in some cases for drilled and grouted piles in overconsolidated clay, f may exceed these values. In determining f for drilled and grouted piles, the strength of the soil-grouted interface, including potential effects of drilling mud, should be considered. A further check should be made of the allowable bond stress between the pile steel and the grout as recommended in Section H.4-2. For further discussion refer to Reference G3.

In layered soils, shaft friction values, f, in the cohesive layers should be as given in Equation G.4-2. End bearing values for piles tipped in cohesive layers with adjacent weaker layers may be as given in Equation G.4-4 assuming that (1) the pile achieves penetration of two to three diameters or more into the layer in question, and (2) the tip is approximately three diameters above the bottom of the layer to preclude punch through. Where these distances are not achieved, some modification in the end bearing resistance may be necessary. Where adjacent layers are of comparable strength to the layer of interest, the proximity of the pile tip to the interface is not a concern.

**G.4.3 Shaft Friction and End Bearing in Cohesionless Soils.** For pipe piles in cohesionless soils, the shaft friction, in stress units, may be calculated by the equation

$$f = Kp'_o \tan \delta \quad \text{(G.4-5)}$$

where:

K = dimensionless coefficient of lateral earth pressure (ratio of horizontal to vertical normal effective stress)

$p'_o$ = effective overburden pressure at the point in question,

$\delta$ = friction angle between the soil and pile wall

For open-ended pipe piles driven unplugged, it is usually appropriate to assume K as 0.8 for both tension and compression loadings. Values of K for full displacement piles (plugged or closed end) may be assumed to be 1.0. Table G.4.3-1 may be used for selection of $\delta$ if other data are not available. For long piles, f may not indefinitely increase linearly with the overburden pressure as implied by Equation G.4-5. In such cases, it may be appropriate to limit f to the values given in Table G.4.3-1.

For end bearing of piles in cohesionless soils, the unit end bearing, q in stress units, may be computed by the equation:

$$q = p'_o N_q \quad \text{(G.4-6)}$$

where:

$p'_o$ = effective overburden pressure at the pile tip

$N_q$ = dimensionless bearing capacity factor

Recommended values of $N_q$ are presented in Table G.4.3-1. The shaft friction, f, acts on both the inside and outside of the piles. However, the total resistance in excess of the external shaft friction plus annular end bearing is the total internal shaft friction or the end bearing of the plug, whichever is less. For piles considered to be plugged the bearing pressure may be assumed to act over the entire cross section of the pile. For unplugged piles the bearing pressure acts on the pile annulus only. Whether a pile is considered to be plugged or unplugged may be based on static calculations. For example, a pile could be driven in an unplugged condition but act plugged under static loading.

Load test data for piles in sand (Reference G69) indicate that variability in capacity predictions may exceed those for piles in clay. Other data (Reference G70) suggest that for piles in loose sands and long piles >50 m (>150 ft.) in tension, the method may be less conservative than for compression piles in medium dense to dense sands. Therefore, in unfamiliar situations, the designer may want to account for this uncertainty through a selection of conservative design parameters and/or resistance factors. This may be especially important where load shedding subsequent to peak load development leading to an abrupt (brittle) failure may occur such as the case for short piles under tension loading.

For soils that do not fall within the ranges of soil density and description given in Table G.4.3-1 or for materials with unusually weak grains or compressible structures, Table G.4.3-1 may not be appropriate for selection of design parameters. For example, very loose silts or

TABLE G.4.3-1
DESIGN PARAMETERS FOR COHESIONLESS SILICEOUS SOIL*

| Density | Soil Description | Soil-Pile Friction Angle, $\delta$ Degrees | Limiting Skin Friction Values kPa (kips/ft²) | $N_q$ | Limiting Unit End Bearing Values MPa (kips/ft²) |
|---|---|---|---|---|---|
| Very Loose<br>Loose<br>Medium | Sand<br>Sand-Silt**<br>Silt | 15 | 47.8(1.0) | 8 | 1.9( 40) |
| Loose<br>Medium<br>Dense | Sand<br>Sand-Silt**<br>Silt | 20 | 67.0(1.4) | 12 | 2.9( 60) |
| Medium<br>Dense | Sand<br>Sand-Silt** | 25 | 81.3(1.7) | 20 | 4.8(100) |
| Dense<br>Very Dense | Sand<br>Sand-Silt** | 30 | 95.7(2.0) | 40 | 9.6(200) |
| Dense<br>Very Dense | Gravel<br>Sand | 35 | 114.8(2.4) | 50 | 12.0(250) |

*The parameters listed in this table are intended as guidelines only. Where detailed information such as in situ cone tests, strength tests on high quality samples, model tests, or pile driving performance is available, other values may be justified.
**Sand-Silt includes those soils with significant fractions of both sand and silt. Strength values generally increase with increasing sand fractions and decrease with increasing silt fractions.

soils containing large amounts of mica or volcanic grains may require special laboratory or field tests for selection of design parameters. Of particular importance are sands containing calcium carbonate which are found extensively in many areas of the oceans. Available data suggest that driven piles in these soils may have substantially lower design strength parameters than given in Table G.4.3-1. Drilled and grouted piles in carbonate sands, however, may have significantly higher capacities than driven piles and have been used successfully in many carbonate areas. The characteristics of carbonate sands are highly variable and local experience should dictate the design parameters selected. For example, available qualitative data suggest that capacity is improved in carbonate soils of high densities and higher quartz contents. Cementation may increase end bearing capacity, but result in a loss of lateral pressure and a corresponding decrease in frictional capacity. These materials are discussed further in the Commentary.

For piles driven in undersized drilled or jetted holes in cohesionless soils the values of f and q should be determined by some reliable method that accounts for the amount of soil disturbance due to installation, but they should not exceed values for driven piles. Except in unusual soil types such as described above, the values of f and q in Table G.4.3-1 may be used for drilled and grouted piles, with consideration given to the strength of the soil grout interface.

In layered soils shaft friction values, f, in the cohesionless layers should be as outlined in Table G.4.3-1. End bearing values for piles tipped in cohesionless layers with adjacent soft layers may also be taken from Table G.4.3-1, assuming that the pile achieves penetration of two or three diameters or more into the cohesionless layer, and the tip is approximately three diameters above the bottom of the layer to preclude punch through. Where these distances are not achieved, some modification in the tabulated values may be necessary. Where adjacent layers are of comparable strength to the layer of interest, the proximity of the pile tip to the interface is not a concern.

**G.4.4 Skin Friction and End Bearing of Grouted Piles in Rock.** The unit skin friction of grouted piles in jetted or drilled holes in rock should not exceed the triaxial shear strength of the rock or grout, but in general should be much less than this value based on the amount of reduced shear strength from installation. For example, the strength of dry compacted shale may be greatly reduced when exposed to water from jetting or drilling. The sidewall of the hole may develop a layer of slaked mud or clay which will never regain the strength of the rock. The limiting value for this type pile may be the ultimate bond stress between the pile steel and the grout as recommended in Section H.4.3.

The end bearing capacity of the rock should be determined from the triaxial shear strength of the rock and an appropriate bearing capacity factor based on sound engineering practice for the rock materials but should not exceed 9.6 MPa (100 tons per square foot).

## G.5 PILE CAPACITY FOR AXIAL PULLOUT LOADS

The ultimate pile pullout capacity may be equal to or less than but should not exceed $Q_f$, the total skin friction resistance. The effective weight of the pile including hydrostatic uplift and the soil plug should be considered in the analysis to determine the ultimate pullout capacity. For clay, f should be the same as stated in Section G.4.2. For sand and silt, f should be computed according to Section G.4.3. For rock, f should be the same as stated in Section G.4.4.

## G.6 AXIAL PILE PERFORMANCE

**G.6.1 Static Axial Response of Piles.** Piling axial deflections should be within acceptable serviceability limits and these deflections should be compatible with the structural forces and movements. Pile response is affected by load directions, load types, load rates, loading sequence, installation technique, soil type, axial pile stiffness, and other parameters. See Commentary.

**G.6.2 Cyclic Axial Response of Piles.** Unusual pile loading conditions or limitations on design pile penetrations may warrant detailed consideration of cylic loading effects.

Cyclic loadings (including inertial loadings) developed by environmental conditions such as storm waves and earthquakes can have two potentially counteractive effects on the static axial capacity. Repetitive loadings can cause a temporary or permanent decrease in loading carrying resistance, and/or an accumulation of deformation. Rapidly applied loadings can cause an increase in load-carrying resistance and/or stiffness of the pile. Very slowly applied loadings can cause a decrease in load-carrying resistance and/or stiffness of the pile. The resultant influence of cyclic loadings will be a function of the combined effects of the magnitudes, cycles, and rates of applied pile loads, the structural characteristics of the pile, the types of soils, and the factors of safety used in design of the piles. See Commentary.

The design pile penetration should be sufficient to develop an effective pile capacity to resist the design static and cyclic loadings as discussed in Section G.3.4. The design pile penetration can be confirmed by performing pile response analyses of the pile-soil system subjected to static and cyclic loadings. Analytical methods to perform such analyses are described in the commentary to this Section. The pile-soil resistance-displacement (t-z, Q-z) characterizations are discussed in Section G.7.

**G.6.3 Overall Axial Response of Piles.** When any of the above effects are explicitly considered in pile response analysis, the design static and cyclic loadings should be imposed on the pile top and the resistance-displacements of the pile determined. At the completion of the design loadings, the maximum pile resistance and displacement should be determined. Pile deformations should meet structure serviceability requirements. The total pile resistance after the design loadings should meet the requirements of Section G.3.4.

## G.7 SOIL REACTION FOR AXIALLY LOADED PILES

**G.7.1 General.** The pile foundation should be designed to resist the static and cyclic axial loads. The axial resistance of the soil is provided by a combination of axial soil-pile adhesion or load transfer along the sides of the pile and end bearing resistance at the pile tip. The plotted relationship between mobilized soil-pile shear transfer and local pile deflection at any depth is described using a t-z curve. Similarly, the relationship between mobilized end bearing resistance and axial tip deflection is described using a Q-z curve.

**G.7.2 Axial Load Transfer (t-z) Curves.** Various empirical and theoretical methods are available for developing curves for axial load transfer and pile displacement, (t-z) curves. Theoretical curves described by Kraft et al (1981) Reference G5 may be constructed. Empirical t-z curves based on the results of model and full-scale pile load tests may follow the procedures in clay soils described by Coyle and Reese (1966) Reference G2 or granular soils by Coyle and Sulaiman (1967) Reference G91. Additional curves for clays and sands are provided by Vijayvergyia (1977) Reference G92.

Curves developed from pile load tests in representative soil profiles or based on laboratory soil tests that model pile installation may also be justified.

In the absence of more definitive critiera, the following t-z curves are recommended for non-carbonate soils. These recommended curves are shown in Figure G.7.2-1.

| Clays | z/D | $t/t_{max}$ |
|---|---|---|
| | .0016 | 0.30 |
| | .0031 | 0.50 |
| | .0057 | 0.75 |
| | .0080 | 0.90 |
| | .0100 | 1.00 |
| | .0200 | 0.70 to 0.90 |
| | ∞ | 0.70 to 0.90 |
| **Sands** | **z (in)** | **$t/t_{max}$** |
| | 0 | 0 |
| | 0.100 | 1.00 |
| | ∞ | 1.00 |

where:

z = local pile deflection

D = pile diameter

t = mobilized soil pile adhesion (in stress units)

$t_{max}$ = maximum soil pile adhesion or unit skin friction capacity computed according to Section G.4 (in stress units).

The shape of the t-z curve at displacements greater than $z_{max}$ as shown in Figure G.7.2-1 should be carefully considered. Values of the residual adhesion ratio $t_{res}/t_{max}$ at the axial pile displacement at which it occurs ($z_{res}$) are a function of soil stress-strain behavior, stress history, pile installation method, pile load sequence and other factors.

The value of $t_{res}/t_{max}$ can range from 0.70 to 0.90. Laboratory, in situ or model pile tests can provide valuable information for determining values of $t_{res}/t_{max}$ and $z_{res}$ for various soils. For additional information see References G2, G5, G91, and G92.

**G.7.3 Tip Load — Displacement Curve.** The end bearing or tip load capacity should be determined as described in Sections G.4.2 and G.4.3. However, relatively large pile tip movements are required to mobilize the full end bearing resistance. A pile tip displacement up to 10 percent of the pile diameter may be required for full mobilization in both sand and clay soils. In the absence of more definitive criteria the following curve is recommended for both sands and clays.

| z/D | $Q/Q_p$ |
|---|---|
| .002 | 0.25 |
| .013 | 0.50 |
| .042 | 0.75 |
| .073 | 0.90 |
| .100 | 1.00 |
| ∞ | 1.00 |

where:

z = axial tip deflection

D = pile diameter

Q = mobilized end bearing capacity in force units

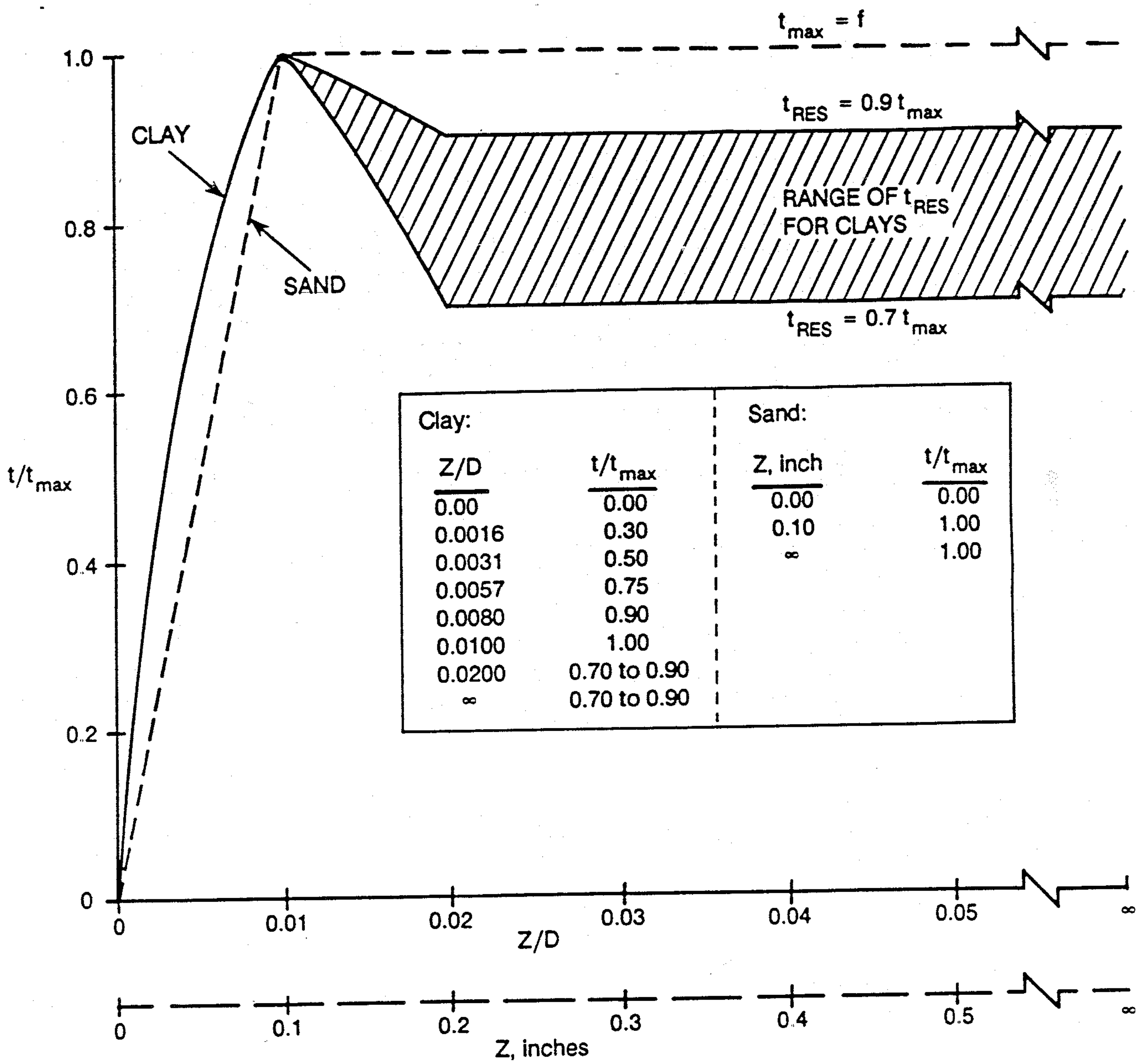
$Q_p$ = total end bearing computed according to Section G.4.

This recommended curve is shown in Figure G.7.3-1.

## G.8 SOIL REACTION FOR LATERALLY LOADED PILES

**G.8.1 General.** The pile foundation should be designed to sustain factored lateral loads, whether static or cyclic. The lateral resistance of the soil near the surface is significant to pile design, and the possible effects on this resistance due to scour and soil disturbance during pile installation should be considered.

In the absence of more definitive criteria, procedures recommended in Sections G.8.2 through G.8.7 may be used for constructing ultimate lateral bearing capacity curves and p-y curves.




| Clay: Z/D | Clay: $t/t_{max}$ | Sand: Z, inch | Sand: $t/t_{max}$ |
|---|---|---|---|
| 0.00 | 0.00 | 0.00 | 0.00 |
| 0.0016 | 0.30 | 0.10 | 1.00 |
| 0.0031 | 0.50 | ∞ | 1.00 |
| 0.0057 | 0.75 | | |
| 0.0080 | 0.90 | | |
| 0.0100 | 1.00 | | |
| 0.0200 | 0.70 to 0.90 | | |
| ∞ | 0.70 to 0.90 | | |

FIG. G.7.2-1
TYPICAL AXIAL PILE LOAD TRANSFER-DISPLACEMENT (t-z) CURVES

**G.8.2 Lateral Bearing Capacity for Soft Clay.** For static lateral loads the ultimate unit lateral bearing capacity of soft clay $p_u$ has been found to vary between 8c and 12c except at shallow depths where failure occurs in a different mode due to minimum overburden pressure. Cyclic loads cause deterioration of lateral bearing capacity below that for static loads. In the absence of more definitive criteria, the following is recommended:

$p_u$ increases from 3c to 9c as X increases from 0 to $X_R$ according to:

$$p_u = 3c + \gamma X + J\frac{cX}{D} \quad \text{(G.8-1)}$$

and

$$p_u = 9c \text{ for } X \geq X_R \quad \text{(G.8-2)}$$

where:

$p_u$ = ultimate resistance, in stress units

c = undrained shear strength of undisturbed clay soil samples, in stress units

D = pile diameter

$\gamma$ = effective unit weight of soil, in weight density units

J = dimensionless empirical constant with values ranging from 0.25 to 0.5 having been determined by field testing. A value of 0.5 is appropriate for Gulf of Mexico clays.

X = depth below soil surface

$X_R$ = depth below soil surface to bottom of reduced resistance zone. For a condition of constant strength with depth, Equations G.8-1 and G.8-2 are solved simultaneously to give:

$$X_R = \frac{6D}{\frac{\gamma D}{c} + J}$$

Where the strength varies with depth, Equations G.8-1 and G.8-2 may be solved by plotting the two equations, i.e., $p_u$ vs. depth. The point of first intersection of two equations is taken to be $X_R$. These empirical relationships may not apply where strength variations are erratic. In general, minimum values of $X_R$ should be about 2.5 pile diameters.

**G.8.3 Load-Deflection (p-y) Curves for Soft Clay.** Lateral soil resistance-deflection relationships for piles in soft clay are generally nonlinear. The p-y curves for the short-term static load case may be generated from the following table:

| $p/p_u$ | $y/y_c$ |
|---|---|
| 0 | 0 |
| 0.5 | 1.0 |
| 0.72 | 3.0 |
| 1.00 | 8.0 |
| 1.00 | $\infty$ |

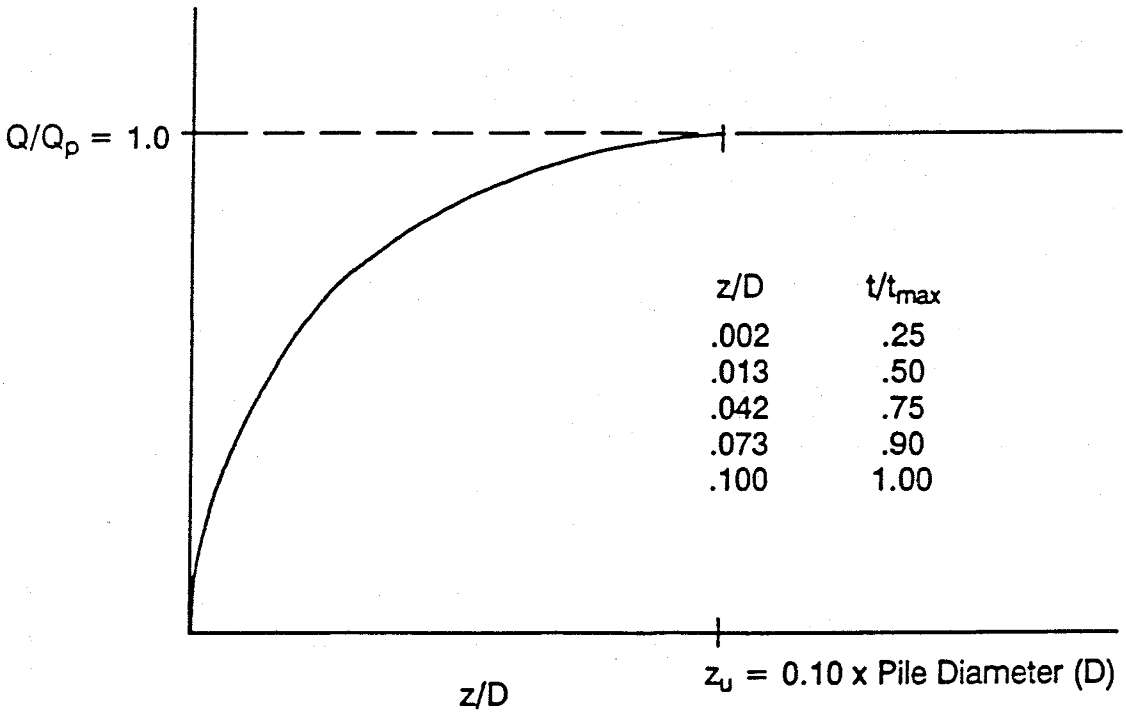



**FIG. G.7.3-1**
**PILE TIP-LOAD-DISPLACEMENT (Q-z) CURVE**

where:

p = actual lateral resistance, in stress units

y = actual lateral deflection

$y_c$ = 2.5 $\varepsilon_c D$

$\varepsilon_c$ = strain which occurs at one-half the maximum stress on laboratory undrained compression tests of undisturbed soil samples

For the case where equilibrium has been reached under cyclic loading, the p-y curves may be generated from the following table:

| $X > X_R$ | | $X < X_R$ | |
|---|---|---|---|
| $p/p_u$ | $y/y_c$ | $p/p_u$ | $y/y_c$ |
| 0 | 0 | 0 | 0 |
| 0.5 | 1.0 | 0.5 | 1.0 |
| 0.72 | 3.0 | 0.72 | 3.0 |
| 0.72 | $\infty$ | $0.72X/X_R$ | 15.0 |
| | | $0.72X/X_R$ | $\infty$ |

**G.8.4 Lateral Bearing Capacity for Stiff Clay.** For static lateral loads, the ultimate bearing capacity, $p_u$, of stiff clay (c > 96 kPa or 1 Tsf) as for soft clay would vary between 8c and 12c. Due to rapid deterioration under cyclic loadings, the ultimate static resistance should be reduced for cyclic design considerations.

**G.8.5 Load-Deflection (p-y) Curves for Stiff Clay.** While stiff clays also have nonlinear stress-strain relationships, they are generally more brittle than soft clays. In developing stress-strain curves and subsequent p-y curves for cyclic loads, consideration should be given to the possible rapid deterioration of load capacity at large deflections for stiff clays.

**G.8.6 Lateral Bearing Capacity for Sand.** The ultimate lateral bearing capacity for sand has been found to vary from a value at shallow depths determined by Equation G.8-3 to a value at deep depths determined by Equation G.8-4. At a given depth the equation giving the smallest value of $p_u$ should be used as the ultimate bearing capacity.

$$p_{us} = (C_1X+C_2D)\,\gamma' X \quad \text{(G.8-3)}$$

$$p_{ud} = C_3 D \gamma' X \quad \text{(G.8-4)}$$

where

$p_u$ = ultimate resistance (force/unit length) (s=shallow, d=deep)

$\gamma'$ = effective soil weight, in weight density units

X = depth

$\phi'$ = angle of internal friction in sand

$C_1$, $C_2$, $C_3$ = Coefficients determined from Figure G.8-1 as a function of $\phi'$

D = average pile diameter from surface to depth

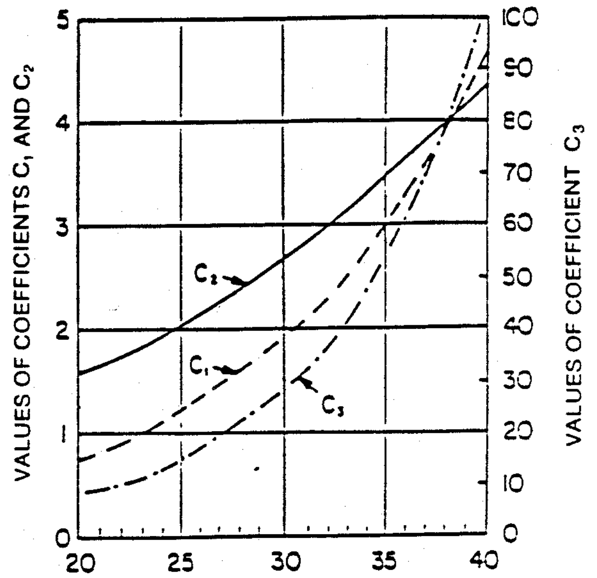



**FIG. G.8-1**
**COEFFICIENTS AS FUNCTION OF $\phi'$**

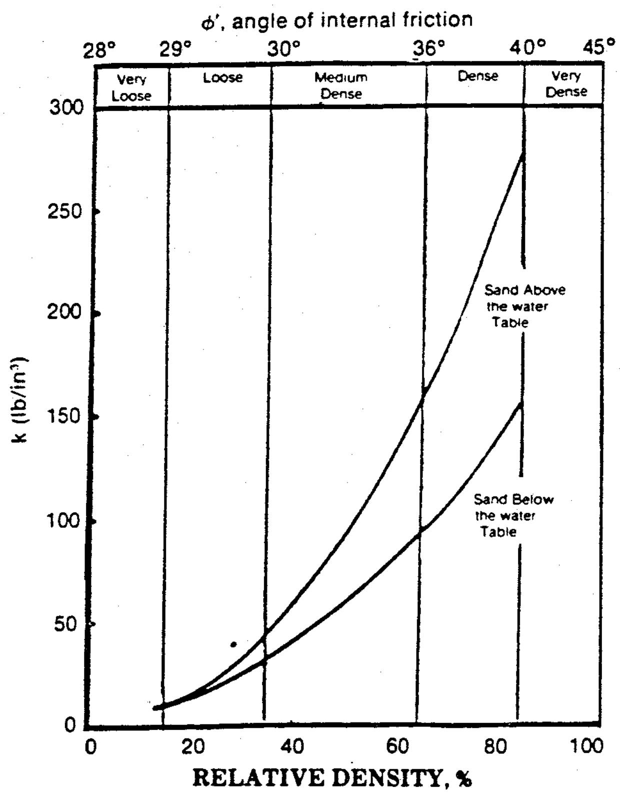



**FIG. G.8-2**

**G.8.7 Load-Deflection (p-y) Curves for Sand.** The lateral soil resistance-deflection (p-y) relationship for sand is also nonlinear and in the absence of more definitive information may be approximated at any specific depth X, by the following expression:

$$P = A\, p_u \tanh\left[\frac{kX}{A p_u}y\right] \qquad \text{(G.8-5)}$$

where

A = factor to account for cyclic or static loading continued. Evaluated by:

$A = 0.9$ for cyclic loading.

$A = \left(3.0 - 0.8\frac{X}{D}\right) \geq 0.9$ for static loading.

$p_u$ = ultimate bearing capacity at depth X in units of force per unit length

k = initial modulus of subgrade reaction in force per volume units. Determine from Figure G.8-2 as function of angle of internal friction, $\phi'$.

y = lateral deflection

X = depth

## G.9 PILE GROUP ACTION

**G.9.1 General.** Consideration should be given to the effects of closely spaced adjacent piles on the load and deflection characteristics of the pile group. Generally, for pile spacing less than eight diameters, group effects may have to be evaluated.

**G.9.2 Axial Behavior.** For piles embedded in clays, the group capacity may be less than a single isolated pile capacity multiplied by the number of piles in the group; conversely, for piles embedded in sands, the group capacity may be higher than the sum of the capacities of the isolated piles. The group settlement in either clay or sand would normally be larger than that of a single pile subjected to the average pile load of the pile group.

**G.9.3 Lateral Behavior.** For piles with the same pile head fixity conditions and embedded in either cohesive or cohesionless soils, the pile group would normally experience greater lateral deflection than that of a single pile under the average pile load of the corresponding group. The major factors influencing the group deflections and load distribution among the piles are the pile spacing, the ratio of pile penetration to the diameter, the pile flexibility relative to the soil, the dimensions of the group, and the variations in the shear strength and stiffness modulus of the soil with depth.

**G.9.4 Pile Group Stiffness and Structure Dynamics.** When the dynamic behavior of a structure is determined to be sensitive to variations in foundation stiffness, parametric analyses such as those described in Comm. G.9.3 should be performed to bound the vertical and lateral foundation stiffness values to be used in the dynamic structural analyses.

## G.10 PILE WALL THICKNESS

**G.10.1 General.** The wall thickness of the pile may vary along its length and may be controlled at a particular point by any one of several loading conditions or requirements which are discussed in the paragraphs below. The pile hammers evaluated for use during driving should be noted by the designer on the installation drawings or specifications.

**G.10.2 Pile Loads.** The internal pile loads caused by factored external loads should be checked as permitted by Section D.2 of this practice. A rational analysis considering the restraints placed upon the pile by the structure and the soil should be used to check the internal pile loads for the portion of the pile which is not laterally restrained by the soil. General column buckling of the portion of the pile below the mudline need not be considered unless the pile is believed to be laterally unsupported because of extremely low soil shear strengths, large computed lateral deflections, or for some other reason.

**G.10.3 Pile Design Checks** The pile wall thickness in the vicinity of the mudline, and possibly at other points, is normally controlled by the combined axial load and bending moment which results from the factored loading conditions for the platform. The moment curve for the pile may be computed with soil resistance determined in accordance with Section G.8, giving consideration to possible soil removal due to scour. When lateral deflections associated with cyclic loads at or near the mudline are relatively large (e.g., exceeding $y_c$ as defined in Section G.8.3 for soft clay), consideration should be given to reducing or neglecting the soil-pile adhesion through this zone.

**G.10.4 Load Check Due to Weight of Hammer During Hammer Placement.** Each pile or conductor section on which a hammer (pile top, drilling rig, etc.) will be placed should be checked for loads due to placing the equipment. These loads may be the limiting factors in establishing maximum length of add-on sections. This is particularly true in cases where piling will be driven or drilled on an incline or batter. The most frequent effects which must be resisted include static bending, axial loads, and lateral loads that are generated during initial hammer placement.

Experience indicates that reasonable protection from failure of the pipe wall due to the above loads is provided if the static capacity is calculated as follows:

1. The projecting add-on section should be considered as a freestanding fixed-end column with its appropriate effective length factor, K (e.g., 2.3 for battered piles and 2.4 for nearly vertical conductors).
2. Bending moments and axial loads should be calculated using the full factored weight of the hammer, pile cap, and leads acting through the center of gravity of their combined masses, ($\gamma_D = 1.3$ or $\gamma_L = 1.5$, depending on how well the weight of each item is known) and the factored ($\gamma_D = 1.3$) weight of the

add-on section taking into account the batter and center-of-mass eccentricities. Nearly vertical add-ons should be considered as inclined cantilevers having an initial or realistically small out-of-plumb inclination or batter of at least 2% when determining their design moment. The secondary bending moment, also to be determined, is the sum of the P-Δ moments due to the determinate or first-order lateral deflections at the top and midheight of the add-on (considered as a fixed-end cantilever) and their associated factored gravity load components.

3. The following beam-column resistance checking equation should not be exceeded:

$$\frac{f_c}{\phi_c F_{cn}} + \frac{f_b}{\phi_b F_{bn}\,(1 - \Sigma P\Delta/M)} \leq 1.0 \ldots\ldots\ \text{G.10-1}$$

where:

$\Sigma P\Delta$ = the first-order P-Δ moments due to factored gravity loads

$f_c, F_{cn}, f_b, F_{bn}, M, \phi_c$, and $\phi_b$ are as defined in Sections D.2.2 and D.2.3.

**G.10.5 Stresses During Driving.** Consideration should also be given to the stresses that occur in the freestanding pile section during driving. The sum of the stresses due to the impact of the hammer (the dynamic stresses) and the stresses due to axial load and bending (the static stresses) should not exceed the minimum yield stress of the steel.

A method of analysis based on wave propagation theory should be used to determine the dynamic stresses, (see G.2.1). In general it may be assumed that column buckling will not occur as a result of the dynamic portion of the driving stresses. The unfactored dynamic stresses should not exceed 80 to 90 percent of yield depending on specific circumstances such as the location of the maximum stresses down the length of pile, the number of blows, previous experience with the pile-hammer combination and the confidence level in the analyses. Separate considerations apply when significant driving stresses may be transmitted into the structure and damage to appurtenances must be avoided.

The static stress during driving may be taken to be the stress resulting from the weight of the pile above the point of evaluation plus the pile hammer components actually supported by the pile during the hammer blows, including any bending stresses resulting therefrom. A dead load factor of 1.6 should be applied to all static loads. The pile strength should be checked using Sections D.2 and D.3.

**G.10.6 Minimum Wall Thickness.** The D/t ratio of the entire length of a pile should be small enough to preclude local buckling at stresses up to the yield strength of the pile material. Consideration should be given to the different loading situations occurring during the installation and the service life of a piling. For inservice conditions and for those installation situations where normal pile-driving is anticipated or where piling installation will be by means other than driving, the limitations of Section D.2 should be considered to be the minimum requirements. For piles that are to be installed by driving where sustained hard driving is anticipated (800 blows per meter or 250 blows per foot with the largest size hammer to be used), the minimum piling wall thickness used should not be less than:

$$t = 6.35 + D/100 \quad \text{for } t\,(\text{mm}),\ D\,(\text{mm}) \ldots\ (\text{G.10.2})$$

$$t = 0.25 + D/100 \quad \text{for } t\,(\text{in}),\ D\,(\text{in})$$

where:

t = wall thickness

D = diameter

Minimum wall thickness for normally used pile sizes should be as listed in the following table:

**MINIMUM PILE WALL THICKNESS**

| Pile Diameter, D | | Nominal Wall Thickness, t | |
|---|---|---|---|
| mm | in. | mm | in. |
| 610 | 24 | 13 | ½ |
| 762 | 30 | 14 | 9/16 |
| 914 | 36 | 16 | ⅝ |
| 1067 | 42 | 17 | 11/16 |
| 1219 | 48 | 19 | ¾ |
| 1524 | 60 | 22 | ⅞ |
| 1829 | 72 | 25 | 1 |
| 2134 | 84 | 28 | 1⅛ |
| 2438 | 96 | 31 | 1¼ |
| 2743 | 108 | 34 | 1⅜ |
| 3048 | 120 | 37 | 1½ |

The preceding requirement for a lesser D/t ratio when hard driving is expected may be relaxed when it can be shown by past experience or by detailed analysis that the pile will not be damaged during its installation.

**G.10.7 Allowance for Underdrive and Overdrive.** With piles having thickened sections at the mudline, consideration should be given to providing an extra length of heavy wall material in the vicinity of the mudline so the pile will not be overstressed at this point if the design penetration is not reached. The amount of underdrive allowance provided in the design will depend on the degree of uncertainty regarding the penetration that can be obtained. In some instances an overdrive allowance should be provided in a similar manner in the event an expected bearing stratum is not encountered at the anticipated depth.

**G.10.8 Driving Shoe.** The purpose of driving shoes is to assist piles to penetrate through hard layers or to reduce driving resistances, thereby allowing greater penetrations to be achieved than would otherwise be the case. Different design considerations apply for each use. If an internal driving shoe is provided to drive through a hard layer it should be designed to ensure that unacceptably high driving stresses do not occur at and above the transition point between the normal and the thickened section at the pile tip. Also it should be checked that the shoe does not reduce the end bearing capacity of the soil plug below the value assumed in the design. External shoes are not normally used as they tend to reduce the skin friction along the length of pile above them.

**G.10.9 Driving Head.** Any driving head at the top of the pile should be designed in association with the installation contractor to ensure that it is fully compatible with the proposed installation procedures and equipment.

## G.11 LENGTH OF PILE SECTIONS

In selecting pile section lengths, consideration should be given to: 1) the capability of the lift equipment to raise, lower and stab the sections; 2) the capability of the lift equipment to place the pile driving hammer on the sections to be driven; 3) the possibility of a large amount of downward pile movement immediately following the penetration of a jacket leg closure; 4) stresses developed in the pile section while lifting; 5) the wall thickness and material properties at field welds; 6) interference with the planned concurrent driving of neighboring piles; and 7) the type of soil in which the pile tip is positioned during driving interruptions for field welding to attach additional sections. In addition, static and dynamic stresses due to the hammer weight and operation should be considered as discussed in Section G.10.4 and Section G.10.5.

Each pile section on which driving is required should contain a cutoff allowance to permit the removal of material damaged by the impact of the pile driving hammer. The normal allowance is 0.5 to 1.5 m (2 to 5 ft.) per section. Where possible, the cut for the removal of the cutoff allowance should be made at a conveniently accessible elevation.

## G.12 SHALLOW FOUNDATIONS

Shallow foundations are those foundations for which the depth of embedment is less than the minimum lateral dimension of the foundation element. The design of shallow foundations should include, where appropriate to the intended application, consideration of the following:

1. Stability, including failure due to overturning, bearing, sliding or combinations thereof.
2. Static foundation deformations, including possible damage to components of the structure and its foundation or attached facilities.
3. Dynamic foundation characteristics, including the influence of the foundation on structural response and the performance of the foundation itself under dynamic loading.
4. Hydraulic instability such as scour or piping action due to wave pressures, including the potential for damage to the structure and for foundation instability.
5. Installation and removal, including penetration and pull out of shear skirts or the foundation base itself and the effects of pressure build up or draw down of trapped water underneath the base.

Recommendations, pertaining to these aspects of shallow foundation design are given in Sections G.13 to G.17.

## G.13 STABILITY OF SHALLOW FOUNDATIONS

The equations to be considered in evaluating the stability of shallow foundations are given below and in Comm. G.13. These equations are applicable to idealized conditions, and a discussion of the limitations and of alternate approaches is also given. Where use of these equations is not justified, a more refined analysis or special solutions should be considered.

**G.13.1 Shallow Foundation Capacity.** The ultimate foundation capacity should satisfy the following conditions:

Bearing: $P_{DB} \leq \Phi_{SB}\, Q_{DB}$

Sliding: $P_{DS} \leq \Phi_{SS}\, H_{DS}$

where:

$Q_{DB}$ = ultimate bearing capacity of the foundation as determined in Sections G.13.2 and G.13.3.

$H_{DS}$ = ultimate sliding capacity of the foundation as determined in Section G.13.3.

$P_{DB}$ = bearing load (under extreme or operating conditions using factored loads)

$P_{DS}$ = sliding load (under extreme or operating conditions using factored loads)

$\Phi_{SB}$ = shallow foundation resistance factor on bearing capacity (= 0.67).

$\Phi_{SS}$ = shallow foundation resistance factor on sliding capacity (= 0.80).

**G.13.2 Undrained Bearing Capacity ($\phi$ = 0).** The maximum gross vertical load which a footing can support under undrained conditions is

$$Q = (cN_c\, K_c + \gamma X)A' \quad \text{(G.13-1)}$$

where:

Q = maximum vertical load at failure in force units

c = undrained shear strength of soil in stress units

$N_c$ = a dimensionless constant, equal to 5.14 for $\phi = 0$

$\phi$ = undrained friction angle = 0

$\gamma$ = total unit weight of soil

X = depth of embedment of foundation

A′ = effective area of the foundation depending on the load eccentricity

$K_c$ = correction factor which accounts for load inclination, footing shape, depth of embedment, inclination of base, and inclination of the ground surface.

Methods for determining the correction factor and the effective area are given in Comm. G.13. Two special cases of Equation G.13-1 are frequently encountered. For a vertical centric load applied to a foundation at ground level, where both the foundation base and ground are horizontal, Equation G.13-1 is reduced below for two foundation shapes.

1. Infinitely Long Strip Footing.

$$Q = 5.14\, cA_o \quad \text{(G.13-2)}$$

where:

Q = maximum vertical load per unit length of footing at failure

$A_o$ = actual foundation area per unit length

2. Circular or Square Footing

$$Q = 6.17\, cA \quad \text{(G.13-3)}$$

where:

Q = Maximum vertical load at failure

A = actual foundation area

**G.13.3 Drained Bearing Capacity.** The maximum net vertical load which a footing can support under drained conditions is

$$Q' = (c'N_cK_c + qN_qK_q + 1/2\gamma'BN_\gamma K_\gamma)A' \quad \text{(G.13-4)}$$

where:

Q′ = maximum vertical load at failure

c′ = effective cohesion intercept of Mohr Envelope

$N_q$ = $(\text{Exp}[\pi \tan \phi'])\tan^2(45° + \phi'/2)$, a dimensionless function of $\phi'$

$N_c$ = $(Nq-1)\cot\phi'$, a dimensionless function of $\phi'$

$N\gamma$ = an empirical dimensionless function $\gamma$ of $\phi'$ that can be approximated by $2(Nq + 1)\tan\phi'$

$\phi'$ = effective friction angle

$\gamma'$ = effective unit weight

q = $\gamma'X$, where X = depth of embedment of foundation

B = minimum lateral foundation dimension

A′ = effective area of the foundation depending on the load eccentricity

$K_c, K_q, K_\gamma$ = correction factors which account for load inclination, footing shape, depth of embedment, inclination of base, and inclination of the ground surface, respectively. The subscripts c, q, and $\gamma$ refer to the particular terms in the equation.

A complete description of the K factors, as well as curves showing the numerical values of $N_q$, $N_c$, and $N_\gamma$ as a function of $\phi'$ are given in Comm. G.13.

Two special cases of Equation G.13-4 for c′=0 (usually sand) are frequently encountered. For a vertical, centric load applied to a foundation at ground level where both the foundation base and ground are horizontal, Equation G.13-4 is reduced below for two foundation shapes.

1. Infinitely, Long Strip Footing,

$$Q = 0.5\gamma'BN_\gamma A_o \quad \text{(G.13-5)}$$

2. Circular or Square Footing,

$$Q = 0.3\,\gamma'BN_\gamma A_o \quad \text{(G.13-6)}$$

**G.13.4 Sliding Stability.** The limiting conditions of the bearing capacity equations in Sections G.13.1 and G.13.2, with respect to inclined loading, represent sliding failure.

For sliding failure the following equations apply.

1. Undrained Analysis:

$$H = cA \quad \text{(G.13-7)}$$

where:

H = maximum horizontal load at failure

2. Drained Analysis:

$$H = c'A + Q'\tan\phi' \quad \text{(G.13-8)}$$

**G.13.5 Capacity of Shallow Foundations.** The ultimate capacities should be determined after cyclic loading effects have been taken into account. For further discussion of foundation capacity, see Commentary.

**G.14 STATIC DEFORMATION OF SHALLOW FOUNDATIONS.** The maximum foundation deformation under static or equivalent static loading affects the structural integrity of the platform, its serviceability, and its components. Equations for evaluating the static deformation of shallow foundations are given in Sections G.14.1 and G.14.2 below. These equations are applicable to idealized conditions. A discussion of alternative approaches is given in the Commentary.

**G.14.1 Short Term Deformation.** For foundation materials which can be assumed to be isotropic and homogenous and for the condition where the structure base is circular, rigid, and rests on the soil surface, the deformations of the base under various loads are as follows:

Vertical: $u_v = \left(\frac{1-\nu}{4GR}\right) Q$ ............ (G.14.1)

Horizontal: $u_h = \left(\frac{7-8\nu}{32(1-\nu)GR}\right) H$ ............ (G.14-2)

Rocking: $\theta_r = \left(\frac{3(1-\nu)}{8GR^3}\right) M$ ............ (G.14-3)

Torsion: $\theta_t = \left(\frac{3}{16GR^3}\right) T$ ............ (G.14-4)

where:

$u_v, u_h$ = vertical and horizontal displacements

Q, H = vertical and horizontal loads

$\theta_r, \theta_t$ = overturning and torsional rotations

M, T = overturning and torsional moments

G = elastic shear modulus of the soil

$\nu$ = Poisson's ratio of the soil

R = radius of the base

These solutions can also be used for approximating the response of a square base of equal area.

**G.14.2 Long Term Deformation.** An estimate of the vertical settlement of a soil layer under an imposed vertical load can be determined by the following equation:

$$u_v = \frac{hC}{1+e_o} \log_{10} \frac{q_o + \Delta q}{q_o} \quad \text{.......... (G.14-5)}$$

where:

$u_v$ = vertical settlement

h = layer thickness

$e_o$ = initial void ratio of the soil

C = compression index of the soil over the load range considered

$p'_o$ = initial effective vertical stress

$\Delta q$ = added effective vertical stress

Where the vertical stress varies within a thin layer, as in the case of a diminishing stress, estimates may be determined by using the stress at the midpoint of the layer. Thick homogeneous layers should be subdivided for analysis. Where more than one layer is involved, the estimate is simply the sum of the settlement of the layers. Compression characteristics of the soil are determined from one-dimensional consolidation tests.

## G.15 DYNAMIC BEHAVIOR OF SHALLOW FOUNDATIONS.

Dynamic loads are imposed on a structure-foundation system by current, waves, ice, wind, and earthquakes. Both the influence of the foundation on the structural response and the integrity of the foundation itself should be considered. See Commentary also.

## G.16 HYDRAULIC INSTABILITY OF SHALLOW FOUNDATIONS.

**G.16.1 Scour.** Positive measures should be taken to prevent erosion and undercutting of the soil beneath or near the structure base due to scour. Examples of such measures are (1) scour skirts penetrating through erodible layers into scour resistant materials or to such depths as to eliminate the scour hazard, or (2) riprap emplaced around the edges of the foundation. Sediment transport studies may be of value in planning and design.

**G.16.2 Piping.** The foundation should be so designed to prevent the creation of excessive hydraulic gradients (piping action conditions) in the soil due to environmental loadings or operations carried out during or subsequent to structure installation.

## G.17 INSTALLATION AND REMOVAL OF SHALLOW FOUNDATIONS.

Installation should be planned to ensure the foundation can be properly seated at the intended site without excessive disturbance to the supporting soil. Where removal is anticipated, an analysis should be made of the forces generated during removal to ensure that removal can be accomplished with the means available. See Commentary also.