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Design and Construction of Drilled Piers

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Covers the design and construction of foundation piers 30 in. (760 mm) in diameter or larger made by excavating a hole in the earth and then filling it with concrete. Smaller-diameter piers have been used in noncollapsing soils. The two-step design procedure includes: (1) determination of overall pier size, and (2) detailed design of concrete pier element itself. Emphasis is on the former, which involves interaction between soil and pier. Construction methods described include excavation, casing, placement of concrete and reinforcing steel, and installation by the slurry displacement method. Criteria for acceptance are presented along with recommended procedures for inspection and evaluation.

Keywords: axial loads; bearing capacity; bending; bending moments; caps (supports); concrete construction; deflection; excavation; foundations; lateral pressure; linings; loads (forces); moments; observation; piers; placing; quality control; reinforced concrete; slurry displacement method; soil mechanics; structural design; tolerances (mechanics); tremie concrete.

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ACI 336.3R-93 supersedes ACI 336.3R-72 (Revised 1985) became effective May 1, 1993.

CHAPTER 1—GENERAL

1.1—Scope

This report deals with design and construction of drilled pier foundations that are constructed by digging, drilling, or otherwise excavating a hole in the earth that is subsequently filled with plain or reinforced concrete. Engineers and constructors have used the terms caissons, foundation piers, bored piles, drilled shafts, subpiers, and drilled piers interchangeably. Only the term drilled pier will be used in this report.

Structural design and construction of drilled pier foundations are the primary objectives of this report. Yet geotechnical considerations are vital because variations in the soil properties have a critical influence on design and construction. Therefore, relevant aspects of soil mechanics are also discussed herein. For the successful design and construction of the drilled pier foundation, it is necessary that a reliable set of data on the supporting earth be obtained. For this task, combined attention and cooperation of the Geotechnical Engineer, Structural Engineer and constructor is essential because limitations of construction often govern the design.

This report is intended primarily for use in building construction, but the sections on construction methods, inspection and testing are equally applicable to bridge and other construction.

1.2—Notation

Dimensioning method: F = force, L = length, and D = dimensionless.

A_b = base area of pier, L^2
A_o = surface area of pier shaft, L^2
c = soil cohesion, FL^{-2}
D = net dead loads, F
D_f = depth of soil overburden, L
D_g = gross dead load, F
D_p = pier length, L
d = diameter of pier shaft, L
d_b = diameter of bearing area, L
d_p = embedded length of pier, L
E, E_c = modulus of elasticity of concrete, FL^{-2}
E_q = load effects of earthquake, F
e = height above ground of horizontal load, L
F = vertical load, F
FS = factor of safety, D
FS_1 = factor of safety for bearing resistance
FS_2 = factor of safety for side resistance
f_c' = compressive strength of concrete, FL^{-2}
f_o = average side resistance, FL^{-2}
f_r = modulus of rupture of concrete, FL^{-2}
f_z = unit load transfer from shaft to soil at depth z, FL^{-2}
f_z-w_z = vertical load-deflection curve at an element of pier, FL^{-1}, L
H = length of pier above ground surface, L
H_g = horizontal shear at ground surface, F
I, I_c = moment of inertia of concrete, L^4
I_{cr} = moment of inertia of the transformed cracked section of concrete, L^4
I_e = effective moment of inertia, L^4
I_g = moment of inertia of gross concrete section, L^4
K = soil reaction coefficient, D
K_1 = constant, FL^{-3}
K_2 = constant, dimension to be selected in each individual case so that the dimensions of k_s becomes L^{-3}
K_f = coefficient of rotational restraint, D
K_m = moment coefficient, D
K_p = passive pressure coefficient, D
K_y = deflection coefficient, D
k_s = modulus of horizontal soil beam reaction, FL^{-2}
L = live loads, F
M = bending moment, FL
M_{cr} = cracking moment, FL
M_g = moment at ground surface, usually applied to pier by superstructure, FL
M_{max} = maximum bending moment, FL
N = number of blows required in standard penetration test to drive a 2 in. (5 cm) sampling spoon 12 in. (30 cm) into the ground, using a 140 lb (64 kg) weight dropping 30 in. (76 cm), D
n = exponent, D
n_h = constant of horizontal modulus of subgrade reaction, FL^{-3}
P_{an} = anchorage resistance, F
P_q = bearing forces acting at the base, F
P_t = total allowable pier resistance, F
P_{ULT} = ultimate lateral load, F
P_{up} = uplift due to submergence, F
p = soil reaction, FL^{-1}
p-y = lateral load-deflection curve at an element of pier, FL^{-1}, L
Q = ultimate compressive capacity, F
q_a = allowable end bearing pressure, FL^{-2}
q_p = ultimate end bearing pressure, FL^{-2}
R = used to denote R_1 or R_2
R_1 = relative stiffness factor for constant k_s (defined in Section 3.4.1), L
R_2 = relative stiffness factor for variable k_s (defined in Section 3.4.1), L
S = slope of elastic curve, D
S_n = negative side resistance, F
S_p = positive side resistance, F
S_u = undrained shear strength, FL^{-2}
T = relative stiffness factor
V = shear, F
W = wind load, F
w = distributed load along pier length, FL^{-1}
w_b = deflection at base of pier, L

w_z = movement of the shaft at depth z, L
x = distance along pier, L
Y_t = distance from centroidal axis of gross section, neglecting reinforcement, to extreme fiber in tension, L
y = lateral deflection of pier, L
z = vertical depth below ground surface, L
α = factor for determining adhesion as part of the soil cohesion value, D
γ = unit weight of soil, FL^{-3}
Σ = base of Napierian logarithms, D
θ = angle of rotation, deg
ρ = ratio of reinforcement, D
ϕ = capacity reduction factor, D
ϕ_n = angle of internal friction in soil, deg

1.3—Limitations

This report is generally limited to piers of 30 in. (760 mm) or larger diameter, made by open construction methods, where water control inside the excavated hole does not require pneumatic provisions. Smaller diameter piers have been installed where soils are consistently stable or casings are left in place. However, it is difficult to detect sidewall collapse in small-diameter piers during concrete placement and casing extraction.

Piers installed by the use of hollow stem augers are not part of this report. Rectangular piers on spread footings in deep excavations or foundations constructed without excavations by methods such as mortar intrusion or mixed-in-place are also beyond the scope of this report.

1.4—Definitions

architect/engineer—the person who is responsible for the aesthetic and overall design of the structure and carries out the responsibilities defined in this report.

bearing stratum—the soil or rock stratum supporting the load transferred to it by a drilled pier or similar deep foundation unit.

bearing type pier—a pier that receives its principal vertical support from a soil or rock layer at the bottom of the pier.

bell—an enlargement at the bottom of the shaft for the purpose of spreading the load over a larger area or for the purpose of engaging additional soil mass for uplift loading conditions.

cap—an upper termination of the shaft, usually placed separately, for the purpose of correcting deviations from desired shaft location, facilitating setting of anchor bolts or dowels within acceptable tolerances, or combining two or more piers into a unit supporting a column.

casing—protective steel tube, usually of cylindrical shape, lowered into the excavated hole to protect workmen and inspectors entering the shaft from collapse or cave-in of the sidewalls, and/or for the purpose of excluding soil and water from the excavation.

combination bearing and side resistance type pier—a pier that receives a portion of its vertical support from bearing at the bottom and a portion from side resistance developed along the shaft.

Table 1—Typical slurry properties

Item to be measured	Range of results at 68°F	Test method
Density prior to concrete, lb/ft^3 a. Friction pier b. End bearing	 85 max 70 max	API 13B Section1
March funnel viscosity, seconds prior to concreting	26 to 45	API 13B Section 2 (marsh funnel and quart)
Sand content by volume, percent before concreting a. Piers with design end bearing b. Piers with no design end bearing	 4 max 10 max*	API 13B Section 4 (sand-screen set)
pH, during excavation	8 to 12	API 13B Section 6 (paper test strips or glass—electrode pH meter)
Sand in polymer slurry immediately prior to concreting	1% max	—
Density of polymer slurry	63.5 lb/ft^3 max	—
Viscosity of polymer slurry	50 max	—

*Higher sand contents have been successfully used in some locations.

construction manager—the person, firm, or corporation with whom the owner enters into an agreement to act in the owner's behalf during construction.

constructor—the person, firm, or corporation with whom the owner enters into an agreement for construction of the work.

controlled slurry—slurry that is made to conform to the specified properties given in Table 1.

design bearing pressure—the vertical load per unit area that may be applied to the bearing stratum at the level of the pier bottom. Design bearing pressure is selected by the geotechnical engineer on the basis of soil samples, tests, analysis, judgement, and experience; with due regard for the character of the loads to be applied and the settlements that can be tolerated.

design vertical side resistance—the allowable vertical frictional resistance in force per unit area that may be applied on the shaft of a pier to resist vertical load. Design side resistance is selected by the geotechnical engineer.

drilled pier—concrete cast-in-place foundation element with or without enlarged bearing area extending downward through weaker soils or water, or both, to a rock or soil stratum capable of supporting the loads imposed on or within it. The drilled pier foundation has been referred to as a drilled shaft, drilled caisson, or large-diameter bored pile. The drilled pier foundation with an enlarged base may be referred to as a belled caisson, belled pier, or drilled-and-under-reamed footing. Drilled pier foundations excavated and concreted with water or slurry in the hole have been known as slurry shafts, piers installed by wet-hole methods, or piers installed by slurry displacement methods.

flexible pier—a pier with a length-to-diameter ratio that will allow significant flexural deformations from lateral loads; the theoretical point of fixity is within the pier shaft.

geotechnical engineer—an engineer with experience in soil mechanics and foundations who is designated to carry out the responsibilities defined in this report.

head—the top of the pier.

inspection—visual observation of the construction, equipment, and materials used therein, to permit the geotechnical engineer to render a professional opinion as to the constructor's conformance with the geotechnical engineer's recommendations or contract documents. Inspection does not include supervision of construction nor direction of the constructor. Inspection may range from the down-hole observation of each pier by the geotechnical engineer or the use of down-hole cameras, to surface observations and testing.

Kelly bar—the stem of the drill used to advance the drilled pier.

owner—party that contracts for approved work performed in accordance with the contract documents.

permitted—permitted by the architect/engineer.

pig—a disposable device inserted into a tremie or pump pipe to separate the concrete from the pier excavation fluid inside the pipe.

project documents—documents covering the required work and including the project drawings and project specifications.

project drawings—part of the project documents; drawings that accompany contract specifications and complement the descriptive information for drilled pier construction work required or referred to in the contract specifications.

project specifications—the specifications that are stipulated by contract for a project and may employ ACI 336.1 by reference and that serve as the instrument for defining the mandatory and optional selections available under the specification.

qualified—qualified by training and by experience on comparable projects.

rabbit—same as **pig**.

rigid pier—a pier with a small depth-to-diameter ratio that will have insignificant flexural deformations under lateral load. Lateral movements will be rotational type involving the entire length of the pier.

rock-socketed pier—pier supported by both side resistance and end bearing within rock.

side resistance type pier—a pier that receives its principal vertical support from side resistance along the shaft.

shaft—drilled pier above bearing surface exclusive of the toe or bell, if any.

side resistance—soil or rock friction or adhesion developed along the side surface of the pier.

slurry—drilling fluid that consists of water mixed with one or more various solids or polymers. Refer to Table 1.

slurry displacement method (SDM)—method of drilling and concreting where controlled slurry is used to stabilize the hole. The slurry may be used (a) for the maintenance of the stability of the unlined drilled pier hole; or (b) to allow acceptable concrete placement when water seepage in a drilled pier hole is too severe to permit concreting in the dry.

socket—portion of pier within bearing stratum.

structural engineer—an engineer contractually designated to carry out the structural design and other defined duties.

submitted—submitted to the architect/engineer for review.

testing agency—the firm retained to perform required tests on the contract construction materials to verify conformance with specifications.

toe—the bottom of the pier.

various pulverized solids—approved solids used to make slurry including bentonite, attapulgite, and site clay.

wet-hole method—method used when a pier extends through a caving stratum. One method is by drilling an oversized hole through a caving stratum and inserting casing, or by loosening the soils without excavating, or by using approved slurry displacement methods to allow casing placement. The casing is inserted into the hole after the caving soils are fully penetrated, and then it is seated. The loosened soils or slurry inside the casing are bailed or pumped out, permitting the hole to continue to advance by drilling dry.

CHAPTER 2—GENERAL CONSIDERATIONS

2.1—General

The function of a pier foundation is to transfer axial loads, lateral loads, torsional loads, and bending moments to the soil or rock surrounding and supporting it. To perform this function, the pier interacts with the soil or rock around and below it and with the superstructure above. The relationship of the pier to the earth is one of the most important variables in the pier design.

In the absence of a theory that can encompass all of the factors involved, simplifying assumptions must be made. However, subtle aspects of construction often govern the design.

2.2—Factors to be considered

Computational results and expected behavior must be evaluated on the basis of the following variables:

2.2.1 *Subsurface conditions*—Soil stratification, groundwater conditions and the depth, thickness and nature of the rock, sand or other material constituting the bearing stratum influence the construction method and the foundation design. Specifically, the design bearing pressure determines the size of the bell or bottom area of the shaft. The properties of the materials above and in the vicinity of the bottom and the effect of disturbance due to construction activity on the soil properties determine the feasibility of constructing a bell without slurry. Permeability, groundwater, and soil properties determine whether the use of casing, slurry, or dewatering will be required; dictate the method of placing the concrete; and may influence ground-loss considerations. Shear strength and deformation characteristics of the soil penetrated by the shaft determine whether side resistance will be a design factor. Side resistance may act to support superstructure loads or it may be a major applied downdrag load on the shaft.

2.2.2 *Site conditions*—Available construction area, site access, and headroom, as well as existing facilities to be protected against settlement, ground loss, noise, or contamination, influence the choice of construction method and thus the design. The effects of the design and the construction methods used for new piers may include subsidence, which is caused, for example, by removing fine-grained materials from the surrounding soil by water flow due to dewatering or consolidation. These effects on adjacent and new structures must be evaluated.

2.2.3 *Inspection and quality control*—The validity of simplifying assumptions made on the basis of field exploration obtained by borings or in-situ testing results should be confirmed by observation by the geotechnical engineer. The scope and method of observation, obtainable tolerances, and quality control influence the refinement to which the design can reasonably be carried. Conversely, allowable tolerances may determine construction methods, scope of observation, and quality control.

The design and installation of drilled piers are multiphase tasks in which proper quality control and quality assurance in construction is vital to the success of the as-installed pier. Without proper quality control and quality assurance, the probability of a successful foundation is reduced. Even the highest quality structural element can have its capacity as a load-carrying member significantly reduced due to installation details and the relationship of the installed concrete element to the surrounding soil.

The presence of the geotechnical engineer should be required during the pier installation. The geotechnical and structural engineers, together, should develop the specifications, which should include clearly defined requirements for testing laboratory services and inspection.

2.2.4 *Constraints*—Construction and design are both affected by available construction expertise and equipment, available materials, and building· code requirements. The limitations of construction will often govern the design.

2.2.5 *Design considerations*—In conjunction with the considerations mentioned above, the designer must compute vertical and lateral loads and moment imposed on the pier. The length and section properties of the pier, distribution of load on end bearing, lateral resistance, and side resistance are determined on the basis of loads and subsurface conditions.

2.2.6 *Laterally loaded pier*—The pier stiffness *EI*, subgrade response, and their interaction are important in the analyses of laterally loaded piers. Soil response is the least predictable variable. Pier deflection is often the limiting factor in determining acceptable lateral loads rather than failure load.

2.3—Pier types

It is convenient to divide piers into types according to the manner in which axial loads are transferred to the soil or rock and according to the response of the pier to lateral load. To which type or types a given pier may be assigned depends on the qualities of the soil and rock around the shaft and at the bottom of the bell or pier, the character of the contact surface between pier and soil or rock, the relative stiffness factor, and the embedded length of the pier.

2.3.1 *Axially supported piers*—With respect to axial load support, there are three types of piers.

2.3.1.1 *Bearing type pier (Fig. 2.3.1.1)*—A straight-sided or belled pier sunk through weaker soils and terminating on a layer of satisfactory bearing capacity is an example of a bearing type pier.

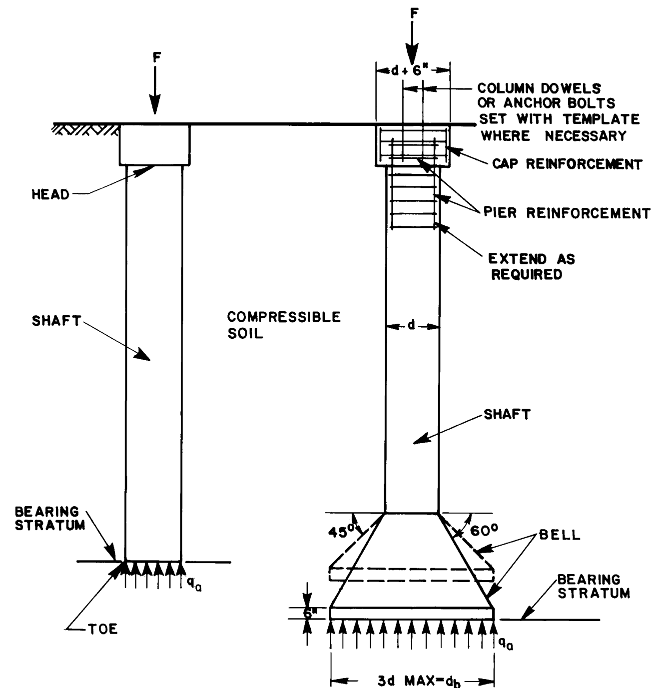


Fig. 2.3.1.1—Example of bearing type piers.

The bearing area may be increased by a bell at the bottom of the shaft. However, the soils in which the bell is constructed must have sufficient cohesion to permit the excavated void to stay open until the concrete is placed. In caving soils, the bell may require grouting or installation by SDMs. Alternatively, the shaft may be enlarged to eliminate the need for the bell or extended into a material in which a bell can be excavated.

2.3.1.2 *Combination bearing and side resistance type pier (Fig. 2.3.1.2)*—A shaft extended (socketed) into a bearing stratum in such a manner that a part of the axial load is transferred to the sides of the pier and the rest of the load is carried in end bearing.

2.3.1.3 *Side resistance type pier (Fig. 2.3.1.3)*—A pier built into a bearing stratum in such a manner that the load is carried by side resistance because the end bearing is negligible or unreliable; for example, in cases in which cleanup of the bottom of the hole is impractical.

2.3.2 *Laterally loaded piers*—On the basis of response to lateral load, there are two pier types.

2.3.2.1 *Rigid pier (Fig. 2.3.2.1)*—A pier so short and stiff in relation to the surrounding soil that lateral deflections are primarily due to rotation about a point along the length of the pier and/or to horizontal translation of the pier. The rotational resistance of a rigid pier is governed in part by the load deformation characteristics of the soil adjacent to and under the embedded portion of the pier, and also by the restraint, if any, provided by the structure above.

2.3.2.2 *Flexible pier (Fig. 2.3.2.2)*—A pier of sufficient length and with flexural rigidity (*EI*) relative to the surrounding soil such that lateral deflections are primarily due to flexure.

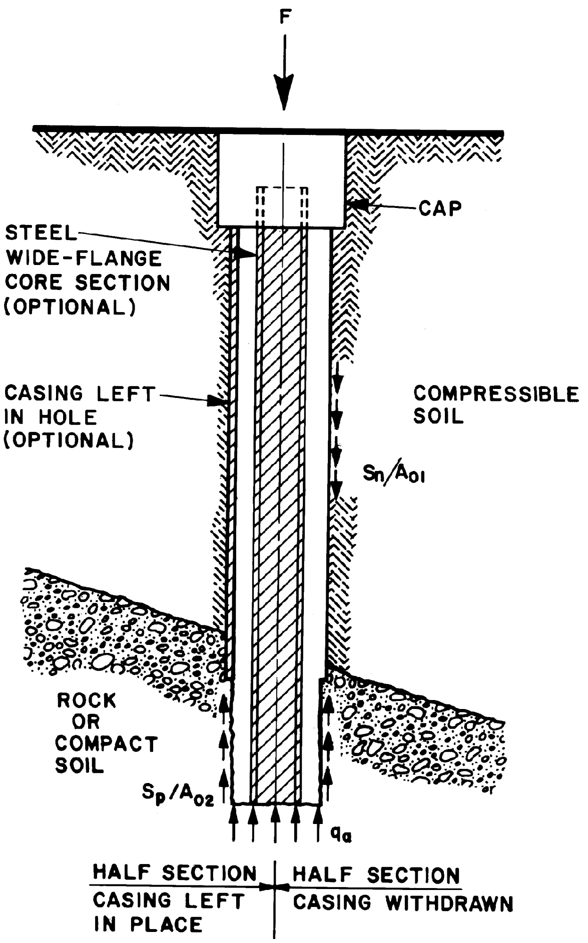


Fig. 2.3.1.2—Combination bearing and side resistance type pier.

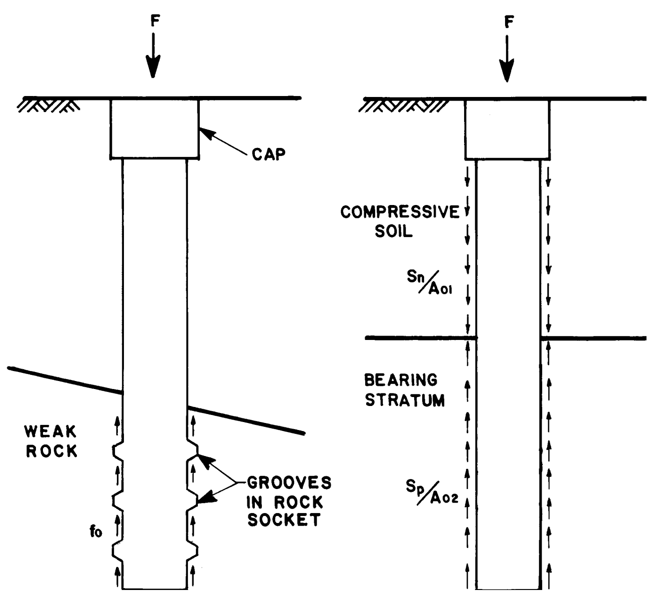


Fig. 2.3.1.3—Side resistance type pier.

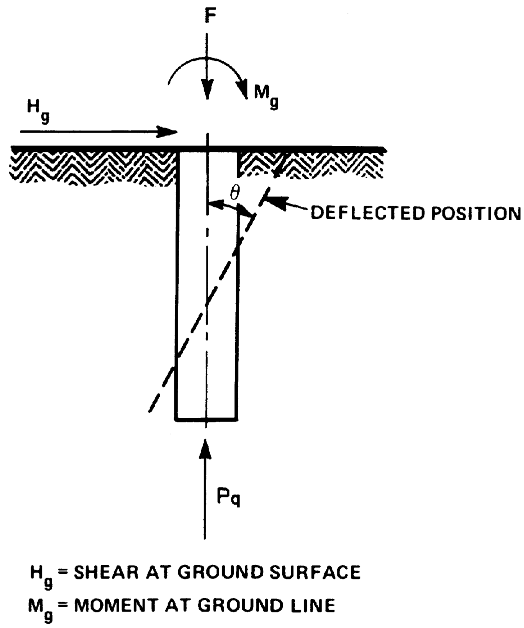


Fig. 2.3.2.1—Rigid type pier (after Davisson [1969], with notation altered).

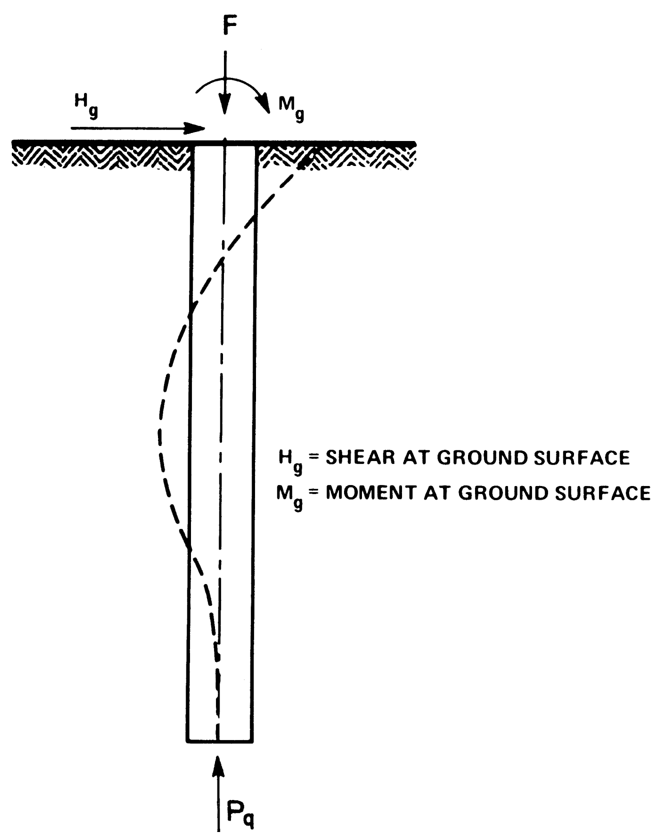


Fig. 2.3.2.2—Flexible type pier (after Davisson [1969], with notation altered).

2.4—Geotechnical considerations

It is necessary for the designer to have an adequate knowledge of the underground site conditions in order to select a foundation system that is constructible and economical. The subsurface exploration should be thorough, with enough samples and data to adequately establish the soil properties within the zones of interest. The investigation should consider the effect of geologic details on foundation design and performance. Such considerations as collapsing soils, fill, shrinkage and swelling conditions, slope stability, rock cavities, potential rock collapse, and weathering profiles should be evaluated as needed. The geotechnical engineer should determine the scope of investigation needed for pier design.

The scope of the investigation should include:

2.4.1 *Number of borings*—A sufficient number of borings should be made to establish with reasonable certainty the subsurface stratification (profile) and the location of the water table. Where the piers are to terminate in rock, the bedrock surface profile and character should also be established with reasonable accuracy.

2.4.2 *Depth of borings in soil deposits*—Boring depth should be adequate to investigate the settlement of the bearing stratum below the pier. Where practical, at least one boring should go into the bedrock.

2.4.3 *Water table and dewatering*—If water is encountered within the zone of pier penetration, the site exploration should obtain pertinent information so any necessary dewatering systems or required slurries may be specified. This should include, as a minimum, the water table elevation(s) (there may be more than one), anticipated fluctuations, if any, and permeability data.

2.4.4 *Piers to bedrock level*—Where piers are to be socketed into bedrock, probes or cores should be extended into the bedrock a depth of at least twice the diameter of the bearing area below the base level, but not less than 10 ft (3 m). This depth is necessary to determine rock strength and condition (if fractured), and to ensure that the pier does not terminate on a suspended boulder. Cores are preferred when the pier capacity is high and the rock quality is critical to establishing maximum pier capacity.

2.4.5 *Soil strength*—In cohesive soil, a sufficient number of undisturbed soil samples should be taken to obtain the unit weight and the soil strength parameters and to obtain depth trends because individual samples may be erratic. In cohesionless soils, it is common practice to estimate the soil density and determine the allowable soil pressure based on the standard penetration test (SPT), cone penetration test (CPT), dilatometer, or pressure meter.

2.4.6 *Load tests*—For large projects or in cases of uncertainty, load tests are desirable. Reaction can be provided by belled or socketed shafts. In addition, Osterberg (1989) has developed a method using a jack seated on grout at the base of the pier with flanges that fill the full pier diameter. The pier is filled with concrete with the hydraulic pipe and telltales extending to the ground surface. Both end bearing and side resistance are tested. The maximum test load is when failure is reached either in side resistance or in end bearing. If the maximum side resistance is exceeded, the test can be conventionally extended further with a reaction frame and dead weights or anchor shafts to permit applying higher end bearing load.

Where the water level would not be a problem, it is also possible to drill a full-sized shaft and perform a plate load test at the shaft base level to establish the bearing capacity. This has the additional advantage of allowing visual observation of the subsurface conditions prior to design. Soil samples should be obtained and probes performed in soils below the plate after load testing. Load tests may also be performed on small-diameter, instrumented, drilled piers and the results extrapolated to larger-diameter piers.

2.4.7 *Lateral response*—At present, the most complete method for evaluating the lateral response of piers is some form of a beam on an elastic foundation mathematical model using a computer (Reese 1977ab, 1984, 1988; Penn. D.O.T.). The major variables are the subgrade response and stiffness (EI). Subgrade reaction may be modeled as a linear spring or as an elastic-plastic material using *p-y* data (Reese 1977ab, 1984, 1988; Penn. D.O.T.). Since no unique method of modeling the subgrade response is universally accepted, the geotechnical engineer should develop the subgrade response model based on the model for which the engineer has had the best local experience.

CHAPTER 3—DESIGN

3.1—Loads

The design of piers consists of two steps:

(a) Determination of pier size or overall concrete dimensions; and

(b) Design of the concrete pier element itself.

In Step (a), which involves interaction between soil and pier, all loads should be service loads and all soil stresses should be at allowable values (refer to Section 3.2). The applied service loads do not include load factors.

In Step (b), the pier is designed by the strength method. Normally, the service loads are used to calculate the resulting moments, shears, and axial forces, which are multiplied by the appropriate load factors for the various cases of loading to structurally design the pier. In the case of a non-linear *p-y* curve and/or variations of shaft axial load (resulting from nonlinear *t-z* curves for side friction), loads must be multiplied by the load factors. The soil pressures required to maintain equilibrium with these factored loads are fictitious and serve no other purpose than to obtain the moments, shears, and axial forces necessary for strength design of the concrete pier (refer to Section 3.3). Where moments or eccentric loading conditions are involved, the fictitious soil pressures required to resist factored loadings may have distributions different from those found for the service load conditions.

3.1.1 *Axial loads*—Axial loads may consist of the axial components of:

D = dead loads from the supported structure and weight of the pier, less weight of material displaced by the pier (net weight of the pier)

D_g = dead loads from the supported structure and weight of the pier (gross weight of the pier)

L = live loads from the supported structure, including impact loads, if any, reduced in accordance with the applicable building code

W, E_q = axial effects from wind or earthquake, respectively

S_p1 = positive side resistance, acting upward on the pier; normally caused by downward movement of the pier relative to the surrounding soil

S_p2 = downward side resistance to resist upward load, acting downward on the pier

S_n = negative side resistance acting downward on the pier; caused by settlement of the surrounding soil relative to the pier, normally an ultimate value. It does not include a factor of safety

P_{an} = anchorage capacity from rock or soil anchors

P_q = bearing resistance acting at the base

P_{up} = uplift force due to submergence of the structure

Refer to Section 1.2

3.1.2 *Lateral loads and moments*—Lateral loads are caused by unbalanced earth pressures, thermal movement of the superstructure, wind- and/or earthquake-generated forces. Moments may be generated by axial loads applied with eccentricity and by lateral loads and may be induced by the superstructure through connections to the pier.

3.2—Loading conditions

The forces interacting between the soil and the pier are determined from the following combinations of loading, whichever produces the greater value for the item under investigation.

3.2.1 *Axial loads*—Maximum and minimum loading conditions should be investigated for pertinent stages of construction and for the completed structure.

3.2.1.1 *Maximum loading*—Excess weight of the pier foundation over the weight of the excavated soil, negative side resistance (downdrag), and long-term redistribution effects on side resistance should be considered. For example, an initial upward-acting side resistance may lessen, disappear or reverse with time from downdrag.

(a) Dead load, live load, side resistance, and uplift:
When positive (upward-acting) side resistance is present:

$$D + L - P_{up} < P_q/FS_1 + S_p1/FS_2 \qquad (3\text{-}1)$$

When negative side resistance is present:

$$D + L - P_{up} < (P_q + S_p1)/FS - S_n \qquad (3\text{-}2)$$

Equation (3-1) or (3-2), whichever applies to the condition investigated, should always be satisfied.

(b) Dead load, live load, side resistance, uplift and wind or earthquake:
When positive side resistance is present:

$$0.75(D + L + W - P_{up}) < (P_q/FS_1 + S_p1/FS) \qquad (3\text{-}3)$$

When negative side resistance is present:

$$0.75(D + L + W - P_{up}) < (P_q + S_p1)/FS - S_n \qquad (3\text{-}4)$$

In Eq. (3-3) and (3-4), W should be entered at its maximum downward-acting value. Side friction resistance and end bearing developed at different displacements are dependent on soil properties. Side resistance is often developed at low displacements of 0.1 to 0.4 in. (3 to 10 mm), while tip resistance is developed at large displacements (2 to 5% of pier diameter in cohesive soils and elastic parts of the resistance in granular soils) (Reese and O'Neill 1988). Factors of safety should be applied separately to these resistances when considering relative displacement. The value of S_n in Eq. (3-4) is sometimes reduced due to pile strain from applied vertical loading, F.

For earthquake resistance designs, $1.1E_q$ should be substituted for W in Eq. (3-3) through (3-7), if the former is greater.

In Eq. (3-1) through (3-4), uplift P_{up} should be entered at its lowest permanent value only.

3.2.1.2 *Minimum loading*—In Eq. (3-5) through (3-7), uplift P_{up} is entered at its maximum value. If:

$$0.9D_g - 1.25W - P_{up} > 0 \qquad (3\text{-}5)$$

no further investigation is needed. Otherwise:

$$P_{up} - 0.9D_g < S_n + P_{an}/FS_2 \qquad (3\text{-}6)$$

$$P_{up} - 0.9D_g + 1.25W < S_n + P_{an}/FS_2 \qquad (3\text{-}7)$$

should both be satisfied. If sufficient side resistance is available, anchors to rock or soil, P_{an} will normally not be necessary. In Eq. (3-5) and (3-7), W should be entered at its maximum upward-acting value.

3.2.2 *Combined loadings*—The effects of lateral loads and moments are to be superimposed on the effects of any simultaneously occurring axial loads in any of the combinations listed in Section 3.2.1.

3.3–Strength design of piers

Foundation piers embedded in soil of sufficient strength to provide lateral support (Section 3.7.5) may be constructed of plain or reinforced concrete. Design of plain concrete piers is governed by the provisions in ACI 318.1. Piers that cannot be designed using plain concrete with practical or desirable dimensions may be designed using reinforced concrete in accordance with the provisions in ACI 318, Chapter 7, Section 7.10 and Chapter 10, Sections 10.2, 10.3, 10.8.4, 10.9, and 10.15. In either case, the design may be based on the strength design method. Reinforced concrete may also be designed by the alternate design method.

If the strength design method is used, all loadings (on the left side of the equations in Section 3.2), whether axial, transverse, or moment, are to be multiplied by the appropriate load factors given below, and all reactions (on the right side of the equations) are evaluated from them. It is emphasized that these reactions have no relationship whatsoever to ultimate soil values but are only intended to balance the factored loadings (refer to Section 3.1). The pier should also satisfy the compatibility requirements of soil reaction with upper estimates of working load. It is recommended that the strength design method be used for analysis regarding load capacity, but concerning settlement and lateral motions, no load factors should be incorporated and only service loads should be used.

In the strength design method, the concrete section and reinforcing steel requirements may be determined by applying load factors to computed shears and bending moments from working loads except for cases noted in Section 3.1.

If the alternate design method is used, all loadings should be service loads with unity load factors as allowed in Appendix B of ACI 318. Soil pressure for resistance should be allowable values that contain factors of safety.

3.3.1 *Load factors for strength design*—A load factor of 1.4 should be used for dead load D, uplift P_{up}, and other loadings caused by liquid pressures on the structure where the maximum pressure can be well-defined. Otherwise, use a load factor of 1.7.

A load factor of 1.7 should be used for live load L, wind load W, earthquake forces of magnitude $(1.1E_q)$, and other loading caused by lateral earth pressures on the structure.

Structural effects of differential settlement, creep, shrinkage, and temperature changes should be included with the dead load D if they are significant. Evaluation should be based on a realistic assessment of their occurrence in service.

3.3.2 *Strength reduction factors*—Strength reduction factors ϕ are given in Section 9.3 of ACI 318.

3.3.3 *Pier reinforcing*—Pier reinforcing is required to resist applied tensile forces or adequately transfer load from the structure to pier.

3.4—Vertical loads capacity

3.4.1 *Capacity from soil or rock*—The total ultimate compressive and tensile capacities may be a combination of end bearing and side friction. The maximum theoretical ultimate capacity is expressed in the following equation

$$Q = S_p1 + P_q \qquad (3\text{-}8)$$

where

Q = ultimate compressive capacity

S_p1 = ultimate side friction that may be taken as the sum of friction on the shaft walls at given elevations

P_q = ultimate end bearing

The designer should consider strain compatibility and deflection in determining the factor of safety.

Factors of safety may vary from 1.5 to 5 for side friction or end bearing, depending on the subsurface conditions, structural loads, and degree of confidence in the subsurface parameters. The side friction and end bearing may be described further by the following equations inconsistent units.

$$S_p1 = f_oA_o \text{ and } P_q = q_pA_b \qquad (3\text{-}9)$$

where

f_o = average unit side friction of a shaft element

A_o = embedded surface area

q_p = unit end bearing pressure

A_b = gross area of the shaft base (or bell)

The geotechnical engineer should estimate values for f_o and q_p using the soil and/or rock properties and construction method. The values of f_o and q_p vary widely and are depth dependent. Determination of these values may require iterative estimates of the allowable capacity of the drilled shaft foundation in collaboration with the structural engineer to satisfy both factor of safety and allowable settlement requirements. The total ultimate capacity will be less than the maximum theoretical if the residual resistance is less than peak side resistance because peak side resistance typically develops much faster than maximum end bearing resistance.

3.4.2 *Estimate of pier settlement where unit loading and soil properties are a design consideration*—The soil compression properties should be determined to permit estimates of total and differential settlement. In-situ tests, such as cone penetrometer, pressure meter or plate load at pier subgrade, full-scale load tests, and laboratory tests of undisturbed pier subgrade soil are commonly used. Total pier settlement is the sum of pier base movement plus elastic pier shortening considering the effect of side resistance.

3.5—Laterally loaded piers

3.5.1 *Lateral loads and moments*—Drilled piers will be subject to large lateral loads along the pier length in cases when piers are used as retaining walls, walls to arrest slope movement, power pole foundations, or anchors. Also, when the earth pressures on the basement walls are unequal or insufficient to resist the lateral loads from the superstructure, the necessary resistance must be provided by the foundations. This condition occurs when there is no basement, when the depth of the basement walls below the ground surface is too shallow, or when the lateral movements associated with the mobilization of adequate earth pressures are too large to be tolerated. The piers will then be loaded with lateral forces at the top, axial forces from overturning and, usually, moments at the top.

The allowable pier head deflection in each design case may be a few tenths of an inch or a few inches, depending on the project requirements.

Piers that must sustain lateral load can be, and have been, designed successfully, by approximate methods. The allowable lateral load on a vertical pier can be obtained from a table of presumptive values found in some handbooks, building codes, or from simplified solutions that assume a rigid pier and one soil type. However, these allowable loads may not be appropriate compared to values that may be computed by the recommended method herein, and they provide no information on pier deflection. Use of simplified solutions may be misleading for many drilled pier foundations.

3.5.1.1 *Batter piers*—To avoid analyzing a pier for lateral loads, some designers assume, according to the approach used for driven piles, that the lateral loads are resisted by the lateral component of axial loads taken by piers installed on a batter. Most methods that are available for the analysis of a pier group that includes batter piers are approximate in that the movements of the pier head under load are not considered. Battered piers in design should be used with caution because the constructor often cannot properly construct the piers to the batter angle desired.

3.5.1.2 *Beam on elastic foundation*—Theory and experience have shown that a more rational and satisfactory solution of the lateral loaded pier design is obtained by using the method of soil-structure interaction with the theory of a beam on an elastic foundation. Variable pier stiffness and multilayered soil systems are fundamental parameters that can be addressed in the analysis using the beam on an elastic foundation theory. Because soil or subgrade response is the most critical element of the analysis, the geotechnical engineer should develop the soil response model. Although either the geotechnical or structural engineer may analyze the pier, analysis by the Geotechnical Engineer is recommended to minimize possible miscommunication or misinterpretation of the soil response model.

3.5.2 *Laterally loaded pier problem*—The application of a lateral load to the head of a pier causes lateral deflection of the pier. The reactions that are generated in the soil must be such that the equations of static equilibrium are satisfied, and the reactions must be consistent with the deflections. Also, because no pier is completely rigid, the amount of pier bending must be consistent with the soil properties and the pier stiffness.

Thus, the problem of the laterally loaded pier is a "soil-structure-interaction" problem. The solution of the problem requires that numerical relationships between pier deflection and soil reaction be known and that these relationships be considered in obtaining the deflected shape of the pier.

Technological advances have allowed the mathematical problem to be solved with relative ease, but the subgrade response characteristics are still uncertain in many cases. Strain gauge measurements have made possible the determination of soil response during the testing of full-scale piers, and numerical solutions allow the deflected shape (lateral deflection) of a pier to be computed rapidly and accurately even though the soil reaction against the pier is a nonlinear function of pier deflection and of depth below the ground surface.

Although several methods (GAI Consultants 1982; Borden and Gabr 1987; Poulos and Davis 1980) are available for the analysis of drilled piers, the method reported by Reese (1984) is shown in the following sections along with approximate methods that may be used for preliminary analysis. The approximate methods are more suitable in a single-layer soil system.

3.5.3 *Pier-soil interaction*—The soil-structure interaction problem can be illustrated by considering the behavior of a strip footing, as shown in Fig. 3.5.3.1. The assumption is usually made that the bearing stress is uniform across the base of the footing as shown in the figure. However, under the stress distribution that is shown, the cantilever portion of the footing will deflect such that the downward movement at "b" is less than the downward movement at "a." The footing is probably stiff enough that the deflection of "b" with respect to "a" is small; however, the concept is established that the base of the footing does not remain planar. Therefore, the bearing stress across the base of the footing conceptually should not be uniform.

Although the behavior of a strip footing involves soil-structure interaction, the potential economic advantage available by taking the curvature of the footing into account is marginal. Figure 3.5.3.2 shows a model of an axially loaded pier with the

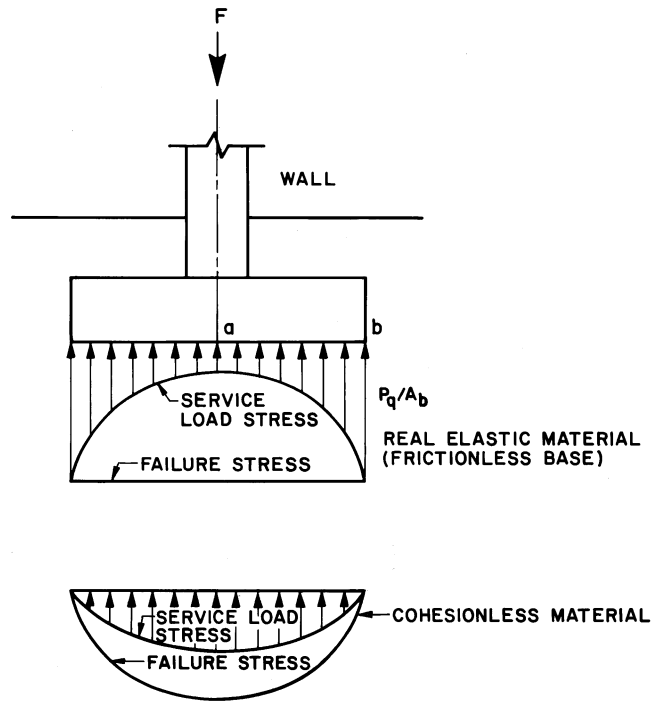


Fig. 3.5.3.1—Soil structure interaction for a strip footing.

soil replaced by a set of mechanisms. The mechanisms show that the load transfer in side resistance and in end bearing are nonlinear functions of the downward movement of the pier. A nonlinear curve showing axial load versus pile-head movement can easily be obtained (Reese 1984) if the mechanisms can be described numerically.

The two examples of soil-structure interaction given illustrate the kind of problem that must be solved. A model for a laterally loaded pier is shown in Fig. 3.5.3.3. A pier is shown with lateral loading at its top. Again, the soil has been replaced by a set of mechanisms that conceptually define soil-response curves. Such curves give the soil resistance p (force per unit length along the pier) as a function of pier deflection y. The mechanisms define bilinear curves as shown in Fig. 3.5.3.3, and it can be seen that the curves vary with position along the pier. Therefore, p is a nonlinear function of both y and x. The p-y concept, though two-dimensional, is based on the synthesis of full-scale pile and pier load tests and soil properties. Shear at the base of the pier is neglected because the pier is considered sufficiently long that lengths are assumed to extend below the theoretical depth of fixity. The determination of the p-y curves by the geotechnical engineer and the selection of pier stiffness are the two most important considerations in the analysis of laterally loaded piers.

3.5.4 *General methods of solution of an individual pier*—Among the available methods, five are considered for the solution of a single pier under lateral loads. These are: elastic method, curves and charts, static method, computer method with nonlinear soil response using a beam on an elastic foundation, and nondimensional curves. The elastic method has a limited application and large numbers of curves and charts would be needed in the general case of the curves and charts

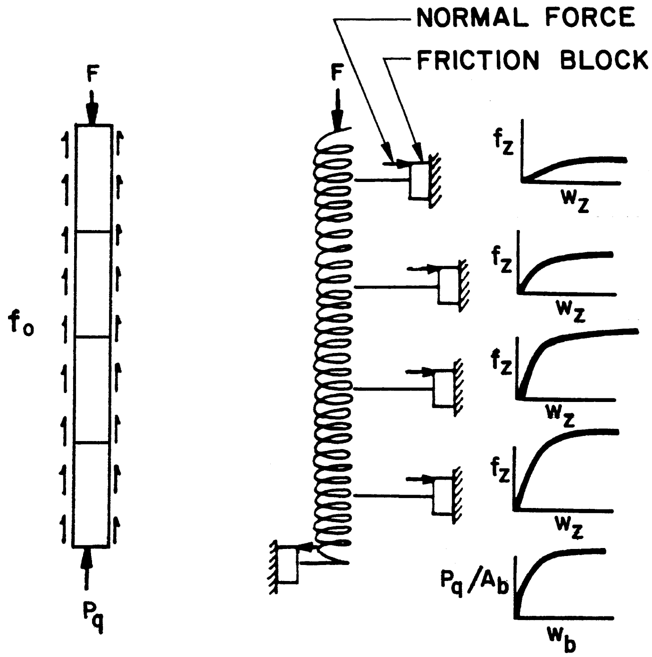


Fig. 3.5.3.2—Model of a pier under axial load (after Reese and O'Neill [1988], with notation altered).

method, so these two methods are not discussed. The other three methods are presented in appropriate detail.

The computer method using the beam on an elastic foundation concept should be used in the design of piers under lateral load. Although the method is easy to employ with modern computers, the subgrade response characteristics still remain complex and obscure. A unique method with consistent parameters is not available. Geotechnical experience and judgement are the principal elements of the analysis; hence, analysis by the geotechnical engineer is recommended.

The nondimensional and static methods have a place in the design process, but these methods are primarily for small-diameter piles. These methods can be employed for preliminary design or as a check of the computer output in simple cases. The simplified methods are limited in that multi-layered systems and complicated ground geometries cannot be considered. Frequently there is uncertainty regarding some of the parameters that enter design computations; for example, in the strength and deformation characteristics of the supporting soil. The computer method not only allows the geotechnical engineer to investigate the influence of these uncertainties because the response of the pier to small variations in parameters can be readily seen, but also enhances the engineer's judgment.

3.5.4.1 *Preliminary design*—For preliminary design, several methods of analysis can be used to evaluate the capacity and deformation of laterally loaded piers. The method of Broms (1965) and Singh et al. (1971) are presented herein.

3.5.4.2.1 *Ultimate capacity (Broms method)*—The ultimate lateral load capacity of a pier defines a loading condition in which a pier can fail with the development of a plastic hinge (long pier) or by unlimited deflection (short pier). The Broms method can be used to compute the ultimate lateral resistance of small piers in cohesive and cohesionless

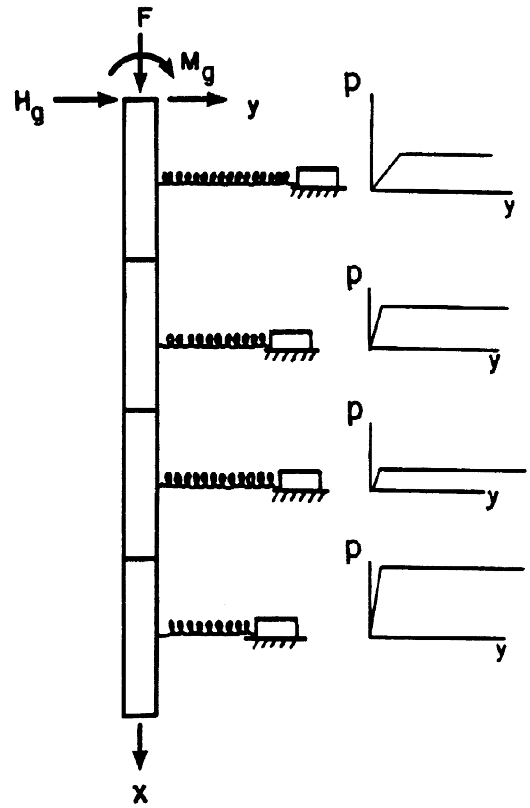


Fig. 3.5.3.3—Model of a pier under lateral loading showing concept of bilinear soil response curves (after Reese and O'Neill [1988], with notation altered).

soils as a function of the pier dimension, type of loading, and fixity at the head. In general, the capacity of short piers is covered by soil failure, while for long piers, capacity is governed by structural failure of the pier. Deflections can be computed using the theory of subgrade reaction (Terzaghi 1955) by assuming a linear relationship between load and deflection.

Piers in cohesive soil—For piers in cohesive soil, P_{ULT} may be determined as the smaller of values obtained from Fig. 3.5.4.1 in which the pier is considered either short or long. For piers in which the embedment length controls (that is, long piers), the maximum bending moment is determined using the following relationships (Broms 1964a):

- For free-head piers

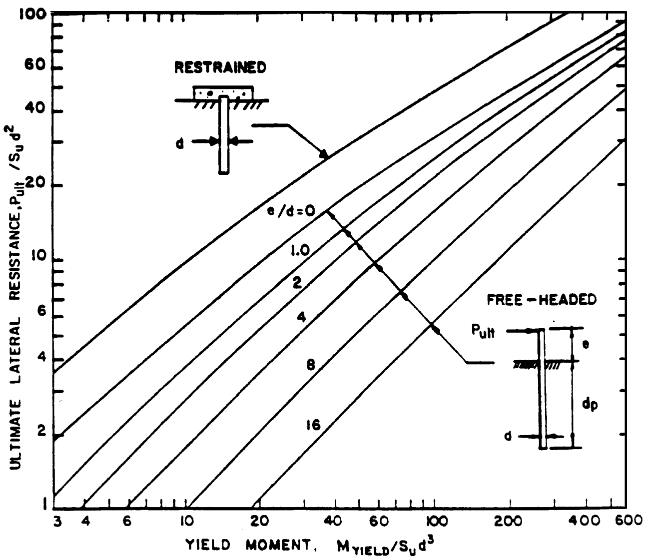
$$M_{MAX} = BP_{ULT}[1.5 + (0.055P_{ULT}/S_UB^2) + (e/B)] \quad (3\text{-}10)$$

- For fixed-head piers

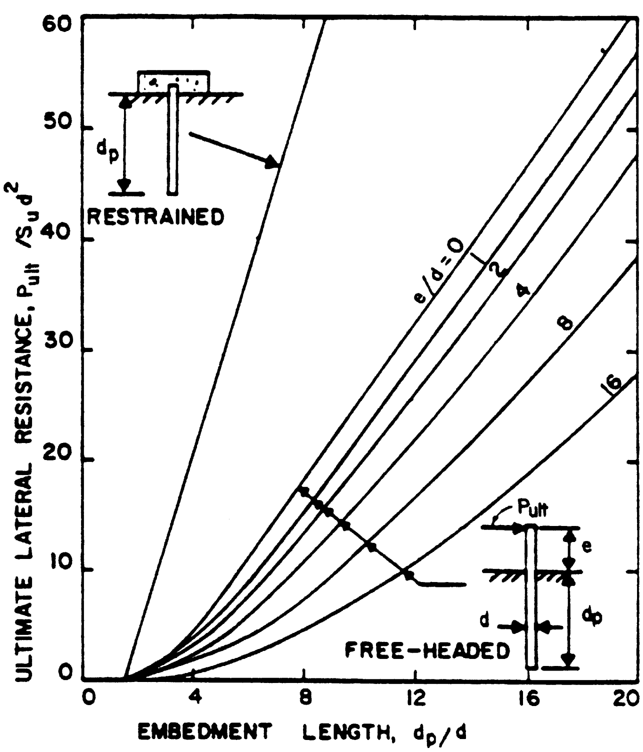
$$M_{MAX} = BP_{ULT}[0.75 + (0.028P_{ULT}/S_UB^2)] \quad (3\text{-}11)$$

Piers in cohesionless soil (C)—For piers in cohesionless soil, P_{ULT} may be determined as the smaller of values obtained from Fig. 3.5.4.2 in which the pier is considered either short or long. For piers in which the embedment length controls (that is, long piers), the maximum bending moment is determined using the following relationships (Broms 1964b)

- For free-head piers



(a) P_{ult} *related to yield moment*



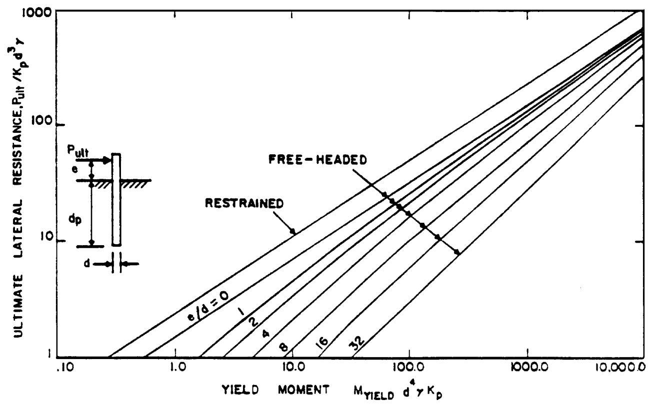
(b) P_{ult} *related to embedment length*

Fig. 3.5.4.1—Ultimate lateral resistance of cohesive soils [after Broms (1964)].

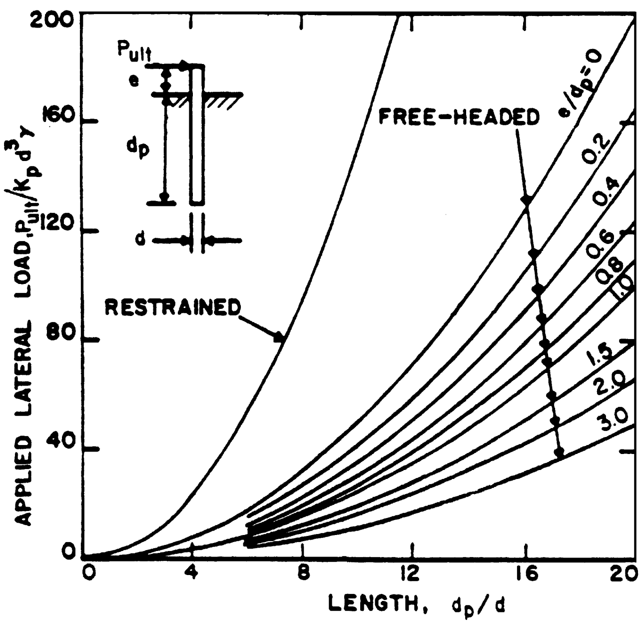
$$M_{MAX} = P_{ULT}[e + 0.55(P_{ULT}/K_pB)^{0.5}] \qquad (3\text{-}12)$$

- For fixed-head piers

$$M_{MAX} = 0.5P_{ULT}[e + 0.55(P_{ULT}/K_pB)^{0.5}] \qquad (3\text{-}13)$$



(a) P_{ult} *related to yield moment*



(b) P_{ult} *related to embedment length*

Fig. 3.5.4.2—Ultimate lateral resistance of cohesionless soils [after Broms (1964)].

3.5.4.3 *Displacement analysis*—Methods for evaluating the displacement of laterally loaded piers for preliminary design include subgrade reaction analyses (Reese 1984) for typical soil profiles and nondimensional solutions.

Subgrade reaction analysis—The displacement of laterally loaded piers is based on the beam-on-elastic-subgrade theory using simplifying assumptions regarding soil stress-strain behavior. The method of Singh et al. (1971) can be used to compute the lateral capacity, displacement, and maximum moment of piers in cohesive and cohesionless soils as a function of pier dimensions, type of loading, and fixity of the head. The method is applicable provided the ratio of pier length (D_p) to the relative stiffness factor (T) is greater than 5.

The lateral load capacity and displacement may be determined using Fig. 3.5.4.3 through 3.5.4.6. The value of T is

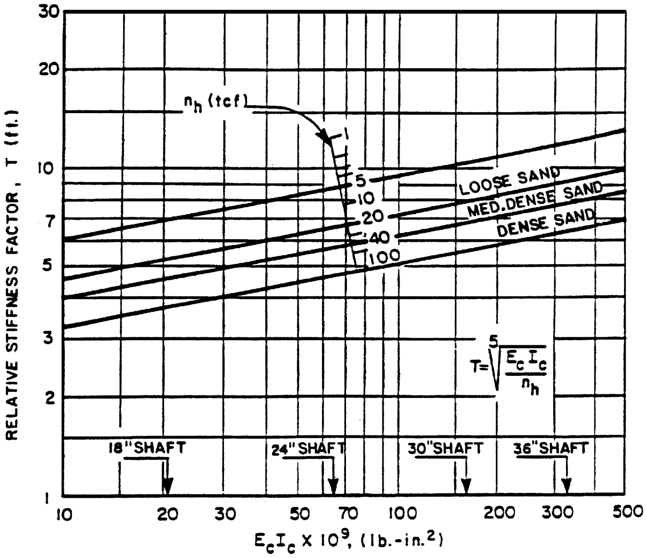


Fig. 3.5.4.3—Relative stiffness factor (modified after Singh et al. [1971]).

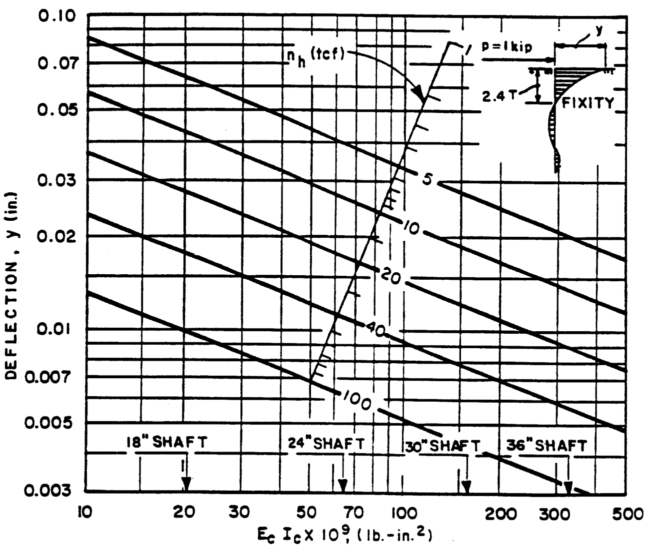


Fig. 3.5.4.4—Deflection for free-headed shaft subjected to 1 kip lateral load (modified after Singh et al. [1971]).

determined using Fig. 3.5.4.3 or the following relationship (Singh et al. 1971):

$$T = (E_c I_c / n_h)^{1/5} \tag{3-14}$$

The values of $E_c I_c$ for common shaft dimensions (assuming $E_c = 3 \times 10^6$ psi) (0.02069×10^6 MPa) are presented along the base of Fig. 3.5.4.3 through 3.5.4.6.

If $T > 5$ ft (1.5 m), use Fig. 3.5.4.4, 3.5.4.5, or 3.5.4.6, depending on the type of loading (that is, force or moment) and head fixity (that is, free or fixed), to determine the lateral deflection for a unit load or moment. For piers with a fixed-head head, the maximum moment (M_{max}) for a 1 kip (4500 N) horizontal load applied to the top of the pier should be determined as follows (Singh et al. 1971)

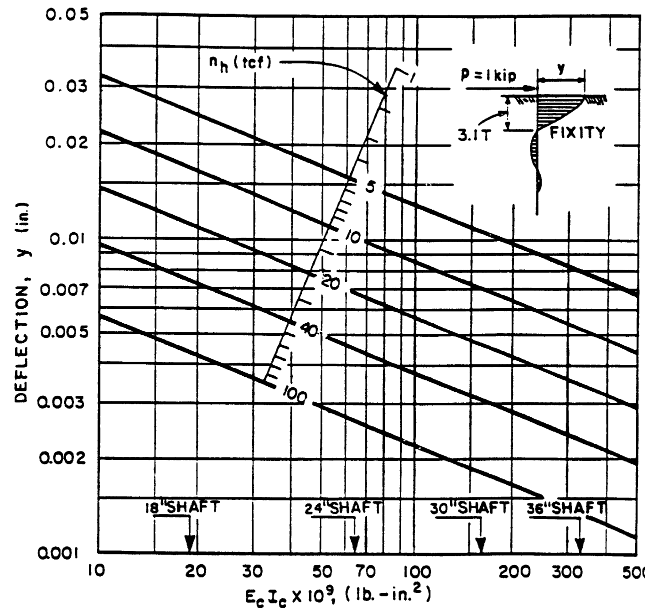


Fig. 3.5.4.5—Deflection of fixed-headed shaft subjected to 1 kip lateral load (modified after Singh et al. [1971]).

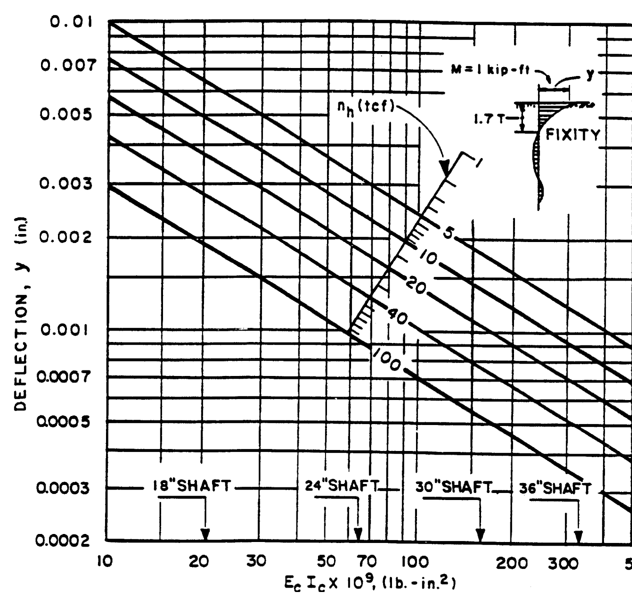


Fig. 3.5.4.6—Deflection for free-headed shaft subjected to 1 kip lateral moment (modified after Singh et al. [1971]).

$$M_{MAX} = -0.92T \text{ (k-ft) at } z = 0 \tag{3-15}$$

and

$$M_{MAX} = 0.26T \text{ (k-ft) at } z = 2.15T \tag{3-16}$$

For piers with a free head, the maximum bending moment should be taken as the larger of the following (Singh et al. 1971)

$$M_{MAX} = 0.98M + 0.45PT \text{ (k-ft) at } z = 0.5T \text{ (ft)} \tag{3-17}$$

and

$$M_{MAX} = 0.85M + 0.73PT \text{ (k-ft) at } z = 1.0T \text{ (ft)} \quad (3\text{-}18)$$

Finite difference method with nonlinear soil response—Preliminary design of laterally loaded drilled piers may be based on the results of computer methods with nonlinear soil response as reported by Reese (1984). The nonlinear flexural rigidity effects of the pier may be incorporated into the analysis to consider the composite properties of the pier.

Nondimensional solutions—Preliminary design of laterally loaded drilled piers may be based on the results of analyses using nondimensional solutions as reported by Reese (1984).

3.5.4.4 *Final design*—Although several methods are available (GAI Consultants 1982; Poulos and Davis 1980; Borden and Gabr 1987), the analysis and design method reported by Reese (1984) is presented herein. The design should ensure that construction methods and design assumptions used in the analyses are consistent.

3.5.4.5 *Computer design procedure*—The computer design procedure considers the soil-structure interaction problem using relationships (*p-y* curves) to define the ground reaction (p) versus pier deflection (y) along the length of the pier. The use of *p-y* curves requires both static equilibrium and compatibility of reaction between the pier and ground, with pier deflections consistent with the stiffness of the pier and ground. Drilled piers are classified as either long (flexible) or short (rigid) and as either fixed against rotation or free to rotate at the ground surface. The extent of head fixity depends on the relative stiffness between the pier(s) and cap (if present).

The analysis is first performed using a stiffness based on the concrete modulus of elasticity and a gross moment of inertia. The analysis is then refined using reduced pier stiffness values based on the composite section, loading conditions, and code requirements.

3.5.4.6 *Response of pier and soil to lateral loads and moments*—Friction along the bottom of the cap should be disregarded for design purposes because the slightest soil consolidation beneath the cap eliminates it unless special measures are taken to ensure continued lateral soil shear resistance. The passive pressure against the cap should also be disregarded wherever excavation for repair or alteration of underground installations will render it ineffective. Passive soil pressures mobilized against pier and pier cap can be effective in resisting lateral loads, provided the displacements to mobilize them can be tolerated.

The solution of the theory of a beam on elastic foundation following the procedures in Reese (1984) of the pier under lateral load must meet two general conditions. The equations of equilibrium must be satisfied and deflections and deformations must be consistent and compatible. These two requirements are fulfilled by finding a solution to the following differential equation

$$E_c I \frac{D^4 y}{dx^4} + F \frac{D^2 y}{ds^2} - p - w = 0 \quad (3\text{-}19)$$

where

F	=	axial load on the pier
y	=	lateral deflection of the pier at a point x along the length of the pier
p	=	soil reaction per unit length
$E_c I$	=	flexural rigidity
w	=	distributed load along the length of the pier

Other beam formulas that are useful in the analysis are

$$E_c I \frac{d^3 y}{dx^3} = V \quad (3\text{-}20)$$

$$E_c I \frac{d^2 y}{dx^2} = M \quad (3\text{-}21)$$

and

$$\frac{dy}{dx} = S \quad (3\text{-}22)$$

where

V	=	shear
M	=	bending moment of the pier
S	=	slope of the elastic curve

The soil response is modeled as *p-y* data, where p is defined as force per unit length along the pier (Reese 1984). The method for solving the governing equations and *p-y* curves are available in the computer program reported by Reese (1984).

3.5.4.7 *Scour*—For building structures and bridges located in rivers or bays, the potential for loss of lateral capacity due to scour should be considered in the design. Refer to Richardson (1991) for general guidelines and methods to estimate and design bridge structure foundations to resist scour.

3.5.4.8 *Cyclic loading*—The effects of cyclic loading on the load-deformation behavior of laterally loaded drilled piers should be considered in the design. The effects of cyclic lateral loading are most pronounced for free-headed piers in stiff cohesive soils. Cyclic loading in loose granular soils also causes reduced resistance to lateral loading, but the effect is much less pronounced than in clays (Reese 1984). In general, cyclic loading has the effect of progressively increasing deflections of piers in clays due to strain softening.

3.5.4.9 *Group action*—Drilled piers in a group are considered to act individually when the center-to-center (CTC) spacing perpendicular to the direction of the applied load is greater than $3d$ and when the spacing parallel to the direction of the applied load is greater than or equal to $8d$. When the pier layout does not conform to these spacings, the effects of shaft interaction should be considered in the design. For the case of closely spaced drilled piers in a group, the interaction behavior is typically accounted for indirectly using empirical procedures proposed by Reese (1984) and Poulos and Davis (1980). One procedure assumes a reduction in the coefficient of lateral subgrade reaction for a pier in a group from that of a single pier using the following ratios (Reese 1984):

CTC shaft spacing (parallel to direction of applied load)	Ratio of lateral resistance of shaft in group to single shaft*
8*d*	1.00
6*d*	0.70
4*d*	0.40
3*d*	0.25

*The reduction does not apply to the lead pier. It does apply to all piles in the shadow of the lead shaft.

The amount of pier deflection should also be considered when evaluating interference. For other spacing, interpolation may be used (Davisson 1969; Prakash 1962).

3.5.4.10 *Combined axial and lateral loading*—The ground-line deflections and maximum moments of drilled piers subject to lateral loads increase with increasing axial load. The effects of combined axial and lateral loading are most pronounced for free-head, short, small-diameter piers in loose or soft ground conditions. Combined loadings can be accounted for in computer analyses (Reese 1984).

3.5.4.11 *Sloping ground*—For drilled piers that extend through or below sloping ground, the potential for additional lateral loading should be considered during design. Placement of drilled piers above or on slopes will reduce the lateral capacity and increase displacement compared to similarly sized and loaded shafts constructed in level ground. One method of analysis for piers in stable slopes is given by Borden and Gabr (1987). Additional consideration should be given for piers in slopes having low factors of safety (marginally stable slopes) or showing ground creep and for piers extending through fills overlying soft soils bearing into more competent underlying soil or rock formations. Lateral loading from slope movement can be large and can fail piers in both bending and shear.

The geotechnical engineer is required to give additional consideration to the slope condition and its effects on both soil reaction and lateral load. Lateral load from an unstable slope may be modeled by negative *p-y* data to consider the three-dimensional loading effects on the pier.

3.5.4.12 *Allowable lateral displacements*—Allowable lateral displacements for drilled piers should be developed by the structural engineer and should be consistent with the function and type of structure, the anticipated service life, and the consequences of unacceptable displacements on the performance of the structure.

3.5.4.13 *Pier stiffness*—The flexural behavior of a drilled pier subjected to bending is dependent on its flexural stiffness *EI*. The value of *EI* is the product of the pier material modulus of elasticity and the moment of inertia of the cross section about the axis of bending. Stiffness *EI* is essentially constant for the level of loading to which a structural-steel member is subjected, but both *E* and *I* vary as the stress conditions change for a reinforced concrete member (Reese 1984). For concrete, the value of *E* varies because of nonlinearity in stress-strain relationships, and the value of *I* is reduced because the concrete in the tension zone becomes ineffective due to cracking. The tensile weakness of concrete and the ensuing cracking is the major factor contributing to the nonlinear behavior of reinforced concrete (Wang and Reese 1987).

In most cases, the *EI* value of a reinforced-concrete pier is assumed to be constant for simplicity in the analysis, although this assumption is not rational. Analyses by Wang and Reese (1987) have shown that the magnitude of error associated with this assumption can be large.

3.5.4.13.1 *Crack mechanism*—Cracks form when the flexural stress due to bending exceeds the tensile strength of concrete. Immediately after the formation of the first crack, the stresses in the concrete near the cracking zone are redistributed. As loading continues, additional cracks open up on occasion, but in general the initial cracks penetrate more deeply with an increase in load.

Many variables affect the development and characteristics of cracks. The major ones, percentage of reinforcement, bond characteristics and tensile strength of concrete where cracks occur at random, and the location and spacing of cracks, are subjected to considerable variation. Studies have shown that the crack spacing and crack width follow a normal distribution and are influenced by each other.

3.5.4.13.2 *ACI 318 recommendations*—The stiffening effect due to tension in the concrete must be considered in making a realistic prediction of short-term deflections of reinforced-concrete beams and, likewise, piers. The American Concrete Institute has developed an approximate method of computing stiffness, taking into account the effect of cracking (ACI 318). The method is summarized below.

$$E_cI_e = E_c[(M_{cr}/M_a)^3I_g + (1-(M_{cr}/M_a)^3)I_{cr}]$$

where

$$M_{cr} = f_rI_g/Y_t \quad (3\text{-}24)$$

$$f_r = 7.5\sqrt{f_c'} \quad (3\text{-}25)$$

and

f_c' = specified compressive strength of concrete, psi
f_r = modulus of rupture of concrete, psi
I_{cr} = moment of inertia of the transformed cracked section of concrete
I_e = effective moment of inertia for computation of deflection
I_g = moment of inertia of gross concrete section about centroidal axis, neglecting reinforcement
E_c = modulus of elasticity of concrete
M_a = maximum moment in member at stage deflection is computed
M_{cr} = cracking moment
Y_t = distance from centroidal axis of gross section, neglecting reinforcement, to extreme fiber in tension

The effective moment of inertia, I_e, described in Eq. (3-23) provides a transition between the upper and lower bounds of I_g and I_{cr} as a function of the level of cracking in the form of M_{cr}/M_a.

The flexural stiffness computed directly from the transformed cracked section, in terms of the *EI* at the instant of cracking, has been recognized to be unrealistic in use. With the modification recommended by ACI 318, the flexural stiffness first shows a constant range of stiffness for an uncracked section. After the cracks are initiated by a higher moment, a rather smooth transition of *EI* in terms of progressive cracking is represented. When a section is fully cracked due to tensile stresses, *EI* reaches a minimum value that can be calculated by the transformed cracked-section method.

3.5.4.13.3 *Recommended stiffness*—Pier flexural stiffness *EI* should be calculated as an upper bound using the gross concrete section as a first approximation. If the applied moments and lateral loads are of sufficient magnitude to cause cracking when acting with concurrent axial loads, then the flexural stiffness, using the effective moment of inertia, should be reduced as provided in ACI 318. Use of a flexural stiffness between the gross section and cracked section values, in accordance with analysis simplified by the computer program PMEIX (Reese 1984) should provide a more realistic pier analysis model and deflection prediction.

3.5.5 *Factor of safety*—Two principal types of failure associated with soil-structure interaction are identified: a soil failure and a pier structural failure. The soil failure is characterized by excessive deflection of the pier under lateral load and is usually associated with short piers. The factor of safety against lateral-load failure of a short pier can be increased significantly by increasing the pier penetration. A pier structural failure occurs when the bending moment becomes greater than the bending resistance of the pier.

The factor of safety should consider the kind of loadings applied to the foundation, soil properties, and the structure's significance. Generally, four types of lateral loadings on piers are recognized: short-term static, repeated, sustained, and dynamic. The associated soil responses are different for each loading type. In dealing with seismic loading, the soils should be tested to determine their susceptibility to collapse or liquidity, the reduction of strength under rapid cyclic loading and, where possible, the *p-y* characteristics under rapid oscillation. These data may be used in the dynamic analysis of the structure as a whole to calculate the transverse loads on the piers. The dynamic analysis of a pier-supported structure involves considerations beyond the scope of this document.

With regard to sustained loadings, such as those which occur due to earth pressures, *p-y* curves for short-term loading can be used. No significant pier deflections will occur with time if the supporting soil is granular. Some long-term deflections will occur if the pier is in stiff clay. Consolidation will occur in soft clay, and the designer must use available theory and judgement to estimate the additional deflection and bending moment that will occur. Obviously, it is not possible to establish a specific procedure for dealing with sustained loading because of the multitude of parameters that are involved, including the consolidation characteristics of the particular clay or silt. The selection of a factor of safety, if sustained loading occurs, must involve the use of considerable judgment by the geotechnical engineer.

The factor of safety for the design of a laterally loaded pier is principally influenced by the confidence of the geotechnical engineer in the values of soil properties that are selected for design. The geotechnical engineer must formulate judgments on soil properties that are based on interpolations, extrapolations, and geological continuity. Uncertainties arise because of the small volume of soil that is sampled and because of unavoidable sample disturbance and other errors in soil testing. A simple numerical value for the factor of safety is not appropriate.

3.5.6 *Design organization*—A comment is desirable about the design team that is responsible for the design of piers under lateral loading. One possibility is that the geotechnical engineer could provide soil-response curves to the structural engineer, who would do the computations of bending moment and deflection. An alternate recommended arrangement is to allow the geotechnical engineer to also perform the lateral load analysis and have the structural engineer furnish the applied loads. In this latter case, the structural engineer should review the lateral load analysis to ensure that the pier design and estimated response are compatible with the design of the superstructure.

3.6—Piers socketed in rock

This type of pier is socketed into rock to a depth of one to six times the diameter of the pier for the purpose of developing high service loading capacity. The drilled pier consists of a heavy wall permanent casing fitted with a cutting shoe and seated into the top of rock. A steel core or heavy reinforcing cage is encased in the concrete, which extends into the rock socket. A column cap is designed to transfer loads from the superstructure to one or more rock-socketed piers. The pier is designed to support all load in the rock.

3.6.1 *Load transfer to rock*—Test loading on high-quality rock has demonstrated that, when the depth of socket exceeds twice the diameter, service loads are usually fully supported in resistance on the sides of the rock socket (Horvath and Kenney 1979; Koutsoftas 1981) prior to mobilizing the available end bearing capacity.

One of the following design assumptions is usually used:

(a) The capacity is estimated based on end bearing only including the effects of embedment within the rock

(b) Capacity is derived solely from side resistance particularly if it is difficult to thoroughly clean the bottom of the drilled pier

(c) The drilled pier capacity is estimated using both end bearing and side resistance

The allowable end bearing is estimated by careful examination of site-specific rock cores, compression tests on rock core and using local code limits or local practice. Pressure meter tests and compressive strength tests on rock cores, adjusted to account for discontinuities in the rock mass, are also used to estimate end bearing capacity (Canadian 1985). Rock coring or rock drilling below the socket in each drilled pier is necessary where rock quality is variable.

The allowable unit shear within the rock socket is also determined by examination of the rock core and relating rock quality and type to local code limits. The compressive strength of the rock and concrete have been used to estimate unit socket shear (Canadian 1985). The unit socket shear that can be developed is also dependent on the roughness of the socket side walls. Grooves are sometimes made in the side walls to enhance shear. Correlations between a roughness factor and the ratio of unit socket shear to compressive strength of the weaker of the concrete or rock are available (Canadian 1985). Churn drill methods are sometimes used in hard rock in lieu of rotary drilling to provide higher socket shear. Shear stresses, however, cannot exceed the allowable bond stress between the concrete and the sides of the socket, 200 psi (1.4 MPa) for Building Code of NYC, 1990; Mass. St. Building Code, 1988, or as given by local code unless confirmed by load test.

Observation of the rock socket is desirable to determine rock quality and roughness of the side wall. Observation methods include diver surveys, entering a dewatered hole, surface examination with a drop light, probing in shallow holes, and underwater television survey.

3.6.2 *Settlement of piers*—Settlement of piers founded on rock generally does not significantly exceed the elastic compression shortening of the structural member. Settlement is the result of open joints or seams of compressible material in the rock. Settlement can also result from inadequate removal of soft or weak material from the bottom of the rock socket. Pressure-meter tests or plate load tests may be used as an aid in estimating drilled pier settlement in soft rock or hard soil. Elastic solutions for estimating caisson settlement have been made and are included in the *Canadian Foundation Engineering Manual* (1985).

3.6.3 *Structural design of rock-socketed piers*—The pier is often designed as a composite column using the procedures outlined in Section 3.3. The steel core can be an H-section representing as much as 5 to 25% of the total pier area. Depending on local code requirements, the casing is sometimes used as a structural member in soils that are non-corrosive. Concrete usually has a compressive strength of 3000 to 6000 psi (20 to 41 MPa) at 28 days. High strengths may be considered where they are readily available. Where the rock socket is below the groundwater table and the hole cannot be pumped dry, current practice is to place the concrete using tremie methods for the full height of the pier.

3.7—Pier configuration

3.7.1 *Bells*—The sides of the bell should slope at an angle of not less than 45 degrees with the horizontal for piers installed in the dry. Based on research and experience the following limitations for allowable end bearing are recommended as a function of bell slope angle:

Maximum allowable bearing pressure	Minimum bell angle from horizontal
6 tsf (600 KPa)	45 degrees
6 to 10 tsf (600 to 1000 KPa)	50 degrees
10 to 15 tsf (1000 to 1500 KPa)	55 degrees
Greater than 15 tsf (1500 KPa)	60 degrees

The thickness at the edge of the bell should be at least 6 in. (152 mm) (refer to Fig. 2.3.1.1). A 3 in. edge of the bell is sometimes used for piers with bearing pressures less than 6 tsf. The diameter of the bell should not exceed three times the diameter of the shaft. The constructor should be required to provide a positive means of demonstrating that the belled section has not caved in and is clean of soil whenever the bell angle is less than 60 degrees.

3.7.2 *Caps*—Where used, the depth of the cap should be sufficient to accommodate development of the vertical reinforcement from the shaft and the dowels or anchor bolts for the column.

3.7.3 *Permanent steel casing*—Permanent steel casing, if used for maximum confinement effects or as a structural member, should have a minimum thickness of 0.0075 of the diameter of the pier shaft, unless the soil and water pressure expected before or experienced during excavation, and the maximum anticipated degree of out-of-roundness requires greater thickness. The critical buckling stress varies inversely with the cube of the radius (Broms 1964b). Poor welds and damage due to placement or handling also affect the ability of the casing to withstand external pressure. The casing should not be included in calculations of area or moment of inertia of the pier section, except for full-length, continuous, noncorrugated casing.

3.7.4 *Joints*—Construction joints in the shaft should be avoided if possible. When a construction joint becomes unavoidable, the surface of the concrete should be roughened and, in unreinforced piers, vertical dowels of sufficient length to develop the bars below and above the joint should be set in the plastic concrete. The cross-sectional area of the dowels should be not less than 0.01 times the gross area of the section in the shaft. Where vertical reinforcing steel is already present, the dowels need only bring the percentage of steel up to 0.01. Where vertical reinforcement equals or exceeds $\rho = 0.01$, no dowels are required. There will normally be a construction joint between the shaft and the cap. In high-seismic-risk territory, the cap and the upper part of the pier may be subjected to moments of magnitude matching the ultimate moment capacity of the supported column and high shear loads. The combination of the two may constitute a severe splitting condition, requiring careful attention to the arrangement of splices in vertical reinforcing steel at the two joints (pier to cap and cap to column), and to the provision of adequate spiral or hoop reinforcing in the cap and affected part of the pier.

3.7.5 *Unbraced piers*—Soil having a standard penetration value $N > 1$ or undrained shear strength greater than 100 lb/ft^2 (5 KPa) may be considered to provide lateral support and effectively prevent buckling of piers at service loads. Piers extending above the soil surface or penetrating caverns or large voids and standing in air, water, or material not capable of providing lateral support, should be designed as columns. Effective column length should be estimated based on end fixity. It can be approximated when the top is pin-connected by using the clear unsupported length plus two shaft diameters.

3.7.6 *Cyclic loading*—Cyclic loading tends to reduce the lateral subgrade modulus, which may be reduced to approximately 30% of the initial value. The combination of group action and cyclic loading may reduce it to as little as 10% of the initial k_s value (Davisson 1969). The stress level, expressed as a percentage of the stress at maximum deflection, should be considered. Small stress levels may result in a negligible reduction in subgrade modulus due to cyclic loading. The actual reduction should be determined by the geotechnical engineer, using tests where feasible.

3.7.7 *Interconnecting ties*—Design for earthquake and other lateral loading may require individual piers or pier caps to be interconnected by ties. While the Uniform Building Code (UBC 1985) reference recommends that each tie should be capable of carrying in tension or compression equal to 10% of the larger pier loading, ties may also be required to resist substantial bending moments. The bending moments will redistribute horizontal loading on the piers, resulting in a need for a more rigorous analysis. Ties are also used to develop passive pressure in the soil under wind loads, and to reduce differential settlements.

3.7.8 *Concrete protection for reinforcement, steel columns stubs or cores*—Where permanent casing is used, the protective concrete thickness should be no less than 3 in. (76 mm), or three times (preferably five times) the maximum size of the coarse aggregate, whichever is larger. Larger clearance between reinforcing and the inside edge of casing should be considered where the casing is to be removed, particularly when the reinforcing cage and casing are long. This will reduce the chance that the casing will bind against the reinforcing cage during removal.

CHAPTER 4—CONSTRUCTION METHODS

4.1—Excavation and casing

4.1.1 Construction methods should be used that ensure that the pier hole is properly located and plumb, that the soil adjacent to the hole is not unduly disturbed and that a clean hole of specified dimensions is provided for its entire length. Each pier should be founded into or on the designated bearing stratum. Concreting should be continuous and have the strength and minimum dimensions specified.

4.1.2 Excavation may be performed by hand, auger drill, bucket drill, clamshell, or any other type of equipment or combination that will obtain the specified results. Overdrilling should be avoided. Where a good bearing stratum of limited thickness overlies poorer material or a water bearing stratum under significant head, drilling to excessive depths may result in costly construction extras or produce an inadequate foundation due to reduced pier bearing capacity.

4.1.2.1 During the excavation of the shaft, the constructor should make frequent checks on the plumbness of the shaft. Rough checks can be made by placing a carpenter's level flush to the Kelly bar. Some causes of excessive deviations from plumb are:

(a) Failure to initially position and then hold the drill rig and auger on the design center of the shaft. Sometimes it is necessary to rotate the drill rig or move from the hole during excavation. Care should be taken to reposition and replumb the auger prior to resuming the drilling.

(b) When the auger encounters obstructions, such as boulders, old foundations, or rubble fill, the auger may tend to veer off and slant the hole.

(c) If the drill rig itself is situated on soft ground, uneven settlements may cause the Kelly bar to veer out of alignment.

(d) The additional force or torque applied when drilling in very dense soils may change the Kelly bar alignment.

4.1.3 In hard, firm soils with little or no groundwater seepage, casing may not be required except for safety. Loose casings, used solely for the purpose of protecting workmen and observers, should be of at least 1/4 in. (6 mm) thick steel. It is normally removed from the hole when observation is completed. Under other circumstances, depending on the method of installation, the ground and water conditions and the surrounding facilities, a tight casing, slurry, or other means may be required to retain the earth. Where subsidence must be prevented or kept to a minimum, retaining of the earth should be required in the contract documents.

4.1.4 Belling of piers, where specified, may be performed by machine or by hand. Where the nature of the soil being belled dictates, or if excessive groundwater is present, special procedures may be required. Some procedures used successfully in the past are described in Baker (1986).

4.1.5 When penetration into rock is specified, such penetration should be obtained by approved methods such as drilling, coring, chipping, and chopping. Blasting should not be permitted in confined areas where such blasting may cause damage to casing or affect the surrounding soil and property.

4.1.6 When piers are to be founded on rock, the shaft should be cut into the rock and the bottom stepped or leveled so that the bearing is obtained on surfaces sloping not more than 10 degrees.

4.1.7 When straight-shaft piers are designed for bearing in shales or other hard formations, all or a portion of the bearing load may be transferred to the formation by side resistance developed between the formation and lower part of the pier. It is the practice, in some localities, to cut a series of groove or key rings into the sides of the pier hole near the bottom (refer to Fig. 2.3.1.3), but under other circumstances shear rings are not required to develop side resistance. Grooving is accomplished by cutters attached to the boring equipment.

4.1.8 A drilled pier should not be excavated too close to another drilled pier in which concrete has been freshly placed and has had insufficient time to set. The minimum distance to prevent a blow-in or collapse of fluid concrete from one hole to the other depends on the soil properties, pier geometry, and setting time for the concrete and should be determined by the geotechnical engineer.

4.2—Placing reinforcement

4.2.1 Steel reinforcement, steel stubs, or core sections should be accurately placed and adequately supported in the correct locations. Should the method of pier construction employed require the removal of casing, care should be exercised to ensure that the reinforcement or other embedded metal is not disturbed or exposed to surrounding soil during the removal process. Spacers, capable of sliding on the casing, should be securely attached to the reinforcement.

4.2.2 The clear spacing between vertical reinforcing bars should be at least three times (preferably five times) the size of the maximum coarse aggregate or three times the bar diameter, whichever is larger.

4.2.3 Vertical splices of reinforcing bars are permitted and should conform to the requirements of ACI 318. Generally, no more than 50% of the bars should be lap-spliced at one location.

4.3—Dewatering, concreting, and removal of casing

4.3.1 Casing should be used to effect a seal and cut off water infiltration into the pier when the casing can be installed into an impervious stratum. Where such a seal against groundwater is not possible, a dewatering system should be installed that will permit proper excavation, observation, and concreting of the pier. Should the dewatering system employed contemplate pumping inside the pier, extreme caution should be used to make sure that the unbalanced water head that is created will not cause a "blow" (bottom heave or quick condition) and disturb the proposed bearing stratum or surrounding facilities.

4.3.2 Infiltration of groundwater, from a source at or near the bottom, at a rate of less than 1/4 in. (6 mm) rise per minute at the bottom of the pier should be considered a dry pier and concrete may be placed by buckets, chuting, tremie pipe, or elephant trunks in a manner that minimizes aggregate segregation; however, the total height of water in the bottom of the pier should not exceed 2 in. (51 mm) at the time that sufficient concrete has been placed to balance the water head. It is also permissible to allow freefall of concrete as long as it can be directed vertically on the centerline of the shaft and it does not hit the sides of the shaft or the reinforcement cage. If a leak occurs some distance above the bottom, an appreciable amount of water could enter the pier in the time needed to fill the pier up to the point of leak. Casing through the permeable stratum is recommended for this case. If infiltration of groundwater exceeds a rise of 1/4 in. (6 mm) per minute, the pier should be considered a wet pier, and concrete should be placed by an approved tremie method such as gravity flow through a watertight vertical pipe or by concrete pumping. (ACI 304R, Chapter 8, Concrete Placed Under Water). Before placing concrete by the tremie method, the pier should be filled with water to the natural water level so that the water head inside and outside the pier are balanced. In all cases where concrete is placed by an approved tremie method, casing or slurry should be used to retain the sides of the excavation during the concreting process. When concrete is to be placed by tremie methods, a redesign of the concrete mixture is required to permit greater slump with no decrease in the specified design strength. It is recommended that every reasonable effort be made to obtain a dry hole. If these efforts fail, or if economy dictates, it may be necessary to tremie the pier.

4.3.2.1 *Compaction of concrete*—The free, unobstructed fall of concrete with a minimum slump of 4.0 in. (102 mm) produces adequate compaction up to the top 5 ft (1.5 m) of depth. Vibration of the concrete is required only for this upper depth. Segregated concrete and laitance should be removed before proceeding with construction of the cap. Discharge of concrete through a hopper with a short downpipe carefully centered on the pier shaft is recommended for free, unobstructed fall. The presence of a reinforcing cage in a pier of small diameter may require dropping the concrete through a long downpipe. This could then require a greater depth of top vibration or proportioning of the concrete mixture for a higher slump and smaller maximum aggregate size similar to tremie concrete.

4.3.3 If ground conditions are such that the casing may be removed during the concreting of piers, the equipment and procedures used should ensure that the concrete will not be disturbed, pulled apart, or pinched off by earth movement. It is of extreme importance to establish that this can be accomplished before the removal of the casing and to check the concrete level during removal. The level of the concrete should always be maintained a minimum of 5 ft (1.5 m) above the bottom of the casing during the placing of concrete, but sometimes a much higher head is required, depending on the water head outside the casing. Proper consideration must be given to the proportioning of the mix, including cement factor, slump, and admixtures, in the placing of concrete and the pulling of casings.

4.3.4 Casings should be maintained in good shape and free from old concrete on the inside surfaces that would tend to make pulling difficult. When ground movements are suspected, frequent measurements by the constructor of the shaft diameter should be made in at least two perpendicular horizontal directions, and at reasonably spaced elevations, to ensure that the specified minimum shaft diameter is being maintained and that out-of-roundness does not indicate imminent collapse. Casing should be of sufficient length to extend past caving soil. The casing diameter should closely match the diameter of the hole when removal of casing is intended.

4.3.5 When piers penetrate very unstable strata, such as peat, and casing is required to maintain the shape of pier through these layers, then the casing should not be removed.

4.3.6 The theoretical volume of concrete required to fill the pier should be computed. If the actual volume (estimated by delivery tickets, for example) installed is appreciably less than the theoretical volume, the pier may have experienced pinching, collapse of side walls, or contamination of tremie concrete. If a pier defect is suspected, then options include investigation of the as-installed pier or immediate reinstallation before the concrete sets up. Rejection of an unacceptable pier will require installation of one or more replacement piers at locations that will facilitate load transfer from the structure above.

4.3.7 The concrete in a pier should, if at all possible, be placed in one continuous operation. If a construction joint is unavoidable, it should be treated in accordance with Section 3.7.4. If the casing is to be removed, and the joint was unintended, it may be necessary to cut the casing at the joint and leave the portion below the joint permanently in place.

4.4—Slurry displacement method

4.4.1 *Excavation*—Methods and equipment used for shaft excavation should leave the side of the hole and the bottom free of loose material that would prevent intimate contact of the concrete with firm, undisturbed soil or rock. The auger should be raised and lowered at a slow enough rate so the slurry does not swirl or cause suction on the side walls as the tool is withdrawn. Swirl of the slurry can cause scouring. Suction can cause cave-ins. Reclean the hole if more than 6 in. (152 mm) of cuttings have settled in the bottom of piers designed with no end bearing.

All spoil and excavated materials should be kept away from each open-shaft excavation to avoid contamination of the excavation after final cleanout.

4.4.2 *Installation method*—Where drilled piers are to be installed below the groundwater level and in caving or sloughing soils, a casing or slurry should be used to stabilize the excavation. The slurry level in the excavation should be maintained at a minimum of 5 ft (1.5 m) above the static groundwater level and above any unstable zones a vertical distance that will prevent caving or sloughing of those zones into the excavation. The constructor should be required to demonstrate that stable conditions are being maintained. This can be accomplished by taping and sounding the bottom condition when no drilling is occurring.

Where the purpose of the slurry is only to maintain an open hole until a casing is placed (wethole method), the initial drilling fluid may be water unless experience has shown that a slurry is required. If no casing is planned, then control of the slurry is much more critical. In some soil profiles, it may be possible to use only water, but this is uncommon. Increasing the slurry viscosity by mixing various mineral solids or chemical polymers with the water will be required if water does not stabilize the pier excavation and lift the drill cuttings.

The slurry should be mixed in mud tanks on-site or arrive at the site premixed; combining or mixing slurry in the shaft should not be permitted in the slurry displacement method but is permissible in the casing method. Polymer slurry may be mixed in the shaft.

The slurry should consist of water or a stable colloidal suspension of various pulverized solids or polymers thoroughly mixed with water so that appropriate properties are maintained. Attapulgite and bentonite should meet API Specification 13A. The type of various solids used will depend on the subsurface conditions and the characteristics of the mixing water. A test report from the supplier giving the physical and chemical properties of the mud should be supplied to the geotechnical engineer at the start of the work.

The slurry should be mixed, stored, and transported using equipment normally used on drilled-pier projects. The water used to mix the slurry should be clean, fresh water, of a quality approved by the geotechnical engineer. Any physical or chemical treatment of the water or slurry that is considered necessary to have the slurry meet the specification is the responsibility of the drilled pier constructor.

The slurry should meet the specifications given in Table 1. Water may be used in place of mixed slurry, but it must provide pier excavation stabilization and be subject to the approval of the geotechnical engineer. Slurry testing should be performed and recorded for quality control. All field test equipment should be provided by the drilled-pier constructor. The frequency of testing should be a minimum of two sets of tests per work shift, the first test being done at the beginning of the shift. Field conditions and the requirements of the drilled pier constructor and the geotechnical engineer may make more frequent testing necessary, such as multiple tests per pier, to assure an acceptable slurry. The drilled-pier constructor should have a slurry sampler capable of obtaining slurry samples at any depth within the drilled shaft excavation available at the site, if requested by the geotechnical engineer.

The constructor should use such drilling tools and excavation procedures that excessive negative fluid pressure in the excavations is prevented. At the completion of excavation, the drilled shaft bottom should be cleaned with an airlift system or a cleanout bucket equipped with a one-way flap gate that prevents spoils in the bucket from reentering the shafts, or other suitable tools. The hole slurry should meet the specifications prior to concreting. If cleaning, recirculating, desanding, or replacing the slurry is necessary, the contractor should be prepared to do so.

Concreting the drilled piers should be completed the same day that the excavation is complete. If this is not possible, the excavation should be redrilled, cleaned, and slurry tested before concreting. The theoretical concrete volume may be plotted graphically for each pier size and compared with the actual volume of concrete placed after each truck load (or at regular intervals if pumped) and noted on the graph.

For piers with end bearing, the slurry sand content should be limited to 4%; otherwise the sand content should be limited to 10%, but sand contents up to 25% have been used successfully in some locations. The slurry density should not exceed 75 lb/ft^3 (1.2 mg/m^3) for end bearing piers. The closer restrictions in the case of end bearing piers are necessary to facilitate feeling the bottom with a sounding rod or weighted tape.

Slurry should be sampled and tested from the mud tank and from samples recovered within 1 ft (0.3 m) from the bottom of the drilled pier.

The bottom of the drilled shaft should be checked to confirm that drill cuttings and hole sides are not falling to the bottom. "Sounding" a tremie pipe or airlift pipe or "feeling" with a weighted line are acceptable methods for checking bottom buildup of material. The permissible amount of drill cuttings at the bottom of the pier should be less than 6 in. (152 mm) in depth without recleaning the hole or modifying the slurry for piers designed without end bearing.

4.4.3 *Reinforcing steel*—Reinforcing steel should be placed in a shaft excavation after the geotechnical engineer has completed his observations and has approved placement.

Following approval of the excavation, the reinforcing steel should be centered in the hole at the correct position and elevation, using spacers as necessary.

Reinforcing steel should not touch the sidewall of the excavation and should be completely encased in concrete, using appropriate spacers as necessary. Minimum concrete cover of 3 in. (76 mm) should be maintained and the minimum clear spacing between reinforcing steel should be 4.0 in. (102 mm) between horizontal reinforcement and three times (preferably five times) the size of the maximum coarse aggregate between vertical reinforcement a minimum of 3 in. [76 mm]).

4.4.4 *Concrete*—All concrete should satisfy the following requirements:

(a) Concrete used in the slurry displacement method should have a slump of 7 to 9 in. (178 to 229 mm).
(b) Maximum aggregate size should be 3/4 in. (19 mm).
(c) Adequate retarder to produce a 5 in. (127 mm) or greater slump after 4 hours. The type of retarder should be approved by the engineer.

Placement of concrete should be made following approval of the excavation by the geotechnical engineer and placement of the reinforcing steel.

Pier excavations should be clean before the start of concrete placement. An airlift or clean-out bucket equipped with a one-direction flow gate should be used to clean the bottoms of each shaft depending on the contract requirements.

If the design includes end bearing, the bottoms should be sounded again with the weighted tape after the steel placement and just prior to concrete placement. All concrete must be placed within the time limit during which the excavation remains clean and stable and the concrete remains fluid. Placement should not begin until adequate concrete supply is assured.

4.4.5 *Concreting methods*—Concrete may be placed by tremie methods or by pumping. In either case, a plugged, capped, or "rabbit"-plugged tremie should be inserted and seated in the excavation. The tremie should extend to the bottom of the shaft prior to the commencement of concrete placement and care should be taken to ensure that all slurry suspension is expelled from the pipe during the initial charging process if the "rabbit" plug approach is used. The tremie pipe should be embedded in fresh concrete a minimum of 3 to 5 ft and maintained at that depth throughout concreting to prevent entry of slurry into the pipe. The concrete should be placed while keeping the tremie tip embedded in the concrete. The first portion of concrete flow should be wasted by overflow at the top of the pier, as it is usually contaminated with mud. Rapid raising or lowering of the tremie should not be allowed. Equipment and procedures should be described in writing to the engineer for prior approval.

In the capped tremie pipe approach, the tremie pipe or pump line should have a seal, consisting of a bottom plate or approved equal, that seals the bottom of the pipe until the pipe reaches the hole bottom and enough concrete has been placed to seal off water flow into the tremie pipe. The use of a disposable "rabbit" or "pig" inserted in the pipe to separate the concrete from the slurry is acceptable.

In the "rabbit" plug approach, the open tremie pipe should be set on the bottom, the "rabbit" plug inserted at the top and then concrete placed, pushing the "rabbit" plug ahead separating the concrete from the slurry. When the pipe is fully charged, the pipe should be lifted off the bottom only enough to start the concrete flowing.

During tremie placement, the bottom of the tremie pipe should not be lifted above the concrete level. If the seal is lost, the pipe should be withdrawn, the seal replaced, and the tremie operation restarted using the "capped" tremie approach.

Aluminum pipe or equipment should not be used for placing concrete.

Exposed concrete should be protected against damage and should be cured and protected to prevent moisture loss and temperature extremes in accordance with ACI 301.

4.5—Safety

The following safety provisions should be regarded as a minimum. Government regulations and the Occupational Safety and Health Administration (OSHA) may impose stricter measures.

4.6.1 *Gas in hole*—All personnel involved in the excavation of a pier hole should be alert to toxic and explosive gases which may be released into the hole. Gas masks, gas detectors, adequate first aid equipment, and blowers to force fresh air to the bottom of the hole should be readily available on the job site to assist personnel in emergencies. If gas is encountered or anticipated, absolutely no personnel entry will be permitted until the shaft has been properly vented and tested.

4.6.2 The use of full-body safety harness and safety rope is recommended for any person entering a pier hole. Protective steel casing to retain soil in the shaft walls should be employed when any person is in the shaft or working at the bottom. A protective cage (diving bell) may sometimes be used instead of full-length casing.

4.6.3 The top of any pier hole should be covered when excavation work is discontinued or finished and the hole left open for any reason. The cover should be substantial and strong enough to prevent any person from falling into the hole.

CHAPTER 5—INSPECTION AND TESTING

5.1—Scope

The purpose of inspection and testing is to see that the pier is constructed in accordance with the design assumptions and specifications, within acceptable tolerances or, if excessive deviations occur, to furnish information needed for corrective measures. It involves, but it is not limited to, checking the location, excavation, plumbness of the shaft, and bell and shaft dimensions if applicable; determining the proper depth of excavation and the strength of the bearing stratum; and observing that proper materials and concrete placement procedures have been used.

5.1.1 *Common causes of faulty piers*—Some of the most frequent conditions that lead to faulty piers are:

(a) Development of voids in a concrete shaft due to improper pulling of casing and the use of concrete having too little slump
(b) Concrete placed in seepage water in the pier
(c) Side cave-in of soil, resulting in contaminated concrete
(d) Improper location or plumbness of pier, or improper reinforcement
(e) Surface cave-in of soil, resulting in contaminated concrete or soil-filled voids

(f) Improperly placed tremie concrete, resulting in segregation or intermixing concrete with slurry or water
(g) Squeeze of shaft or bell excavation prior to concreting or during concreting
(h) Casing collapse
(i) Excess water or contaminated concrete at cold joints resulting in a reduction in concrete quality
(j) Migration of water, resulting in dilution of the concrete mixture and a reduction in concrete quality
(k) Poor concrete delivered to site
(l) Inadequate bell sizes
(m) Inadequate bearing material
(n) Incomplete slurry displacement during concrete tremie placement
(o) Sand settling out of slurry during concrete placement, resulting in stiffening the concrete (slump loss) and folding over of concrete trapping the sand sediments

5.2—Geotechnical field representative

The geotechnical engineer should be responsible for drilled pier installation inspection. The field representative should be on the staff of the geotechnical engineer, should be qualified in pier construction work, and have sufficient technical educational background to interpret observations correctly and to communicate what is seen. Continuous full-time observation of each pier installation is recommended.

5.3—Preliminary procedures

Prior to design, a subsoil study should be made by registered geotechnical engineers using experienced personnel to perform borings, sampling and field testing. For this study, no unreasonable restraints on the amount of soil exploration should be imposed by the owner, architect, or structural engineer.

In addition to determining the location, strength, and compressibility of bearing materials at boring locations, it is desirable to anticipate problems that may be encountered during construction. It is sometimes necessary to drill exploration holes at each pier location. This may be the case when the piers will pass through fills containing concrete and other obstructions or where bearing depths or bearing materials are quite variable. In some cases, large exploratory auger holes or test piers are required to assist in the evaluation of actual conditions to be encountered.

Subsurface boring and test data and reports should be made available to the constructor before bidding. Negotiated contracts with qualified foundation constructors are recommended. If more than one constructor is considered qualified by the geotechnical engineer, then competitive bids may be obtained. Before a contract is signed, it is important that there should be agreement between the geotechnical engineer, the structural engineer or architect, and the constructor as to foundation design and problems that may be encountered during construction. The constructor should submit a plan for foundation construction work for review by the geotechnical engineer and the structural engineer or architect, and any differences of opinion should be resolved. A preconstruction meeting to review the procedure is important.

Provisions for handling unforeseen conditions should be available, and methods for such conditions should be agreed upon and become part of the contract.

5.4—Inspection procedures

Adequate inspection requires downhole observations and tests on each end bearing pier hole wherever practical (refer to *Drilled Shaft Inspectors Manual*). Observation from the ground surface may be required because of unsafe conditions or slurry placement. Procedures for observation from the top are described in Section 5.4.3.9.

Side resistance piers do not normally require downhole observation unless shear rings are specified.

5.4.1 *Location*—The horizontal deviation of the actual pier center at the top (cutoff) from the design center should be determined with instruments and recorded. If a protective casing is used, the top must be checked with a plumb bob from staked reference lines and not be referenced to the surface casing.

5.4.2 *Plumb deviation*—Plumb deviation refers to the horizontal deviation (from the vertical) of the pier and is also known as bottom plumbness. The horizontal deviation of the lower end of the shaft or bell should be referenced to the vertical plumb line of the pier design center at the top.

If for some reason the pier cannot be entered, the plumb deviation can be estimated by moving the plumb line carefully from the design center at the top to the edge of the shaft at the bottom in all four directions and measuring the distance from the center. Care should be exercised for this type of measurement because the plumb bob suspended from the long line will tend to move excessively, or may adhere to the pier walls at the bottom.

5.4.3 *Removal of obstructions*—The geotechnical representative should check that all obstacles that could be detrimental to the proper construction of the piers are removed and logged.

5.4.3.1 *Casing*—Where casing thickness, length, diameter, or other properties have been specified, the geotechnical representative should see that these requirements are satisfied.

5.4.3.2 *Loss of ground*—Ground subsidence can occur from surface caving, squeezing of clays into the pier excavation, flow of saturated silts and sands into the hole, and the removal of soil during pumping operations.

The geotechnical field representative should periodically check the water discharge from the pumps to determine the percent of fines carried in the water. Sediment tanks with overflow weirs are useful, and frequently necessary, for this purpose. Soil movements that could cause damaging loss of ground should immediately be brought to the attention of the constructor and the architect or engineer by the geotechnical field representative. Difficulties in inserting and removing the drilling tools in the shaft excavation provide an indication of squeezing of soils. Volume comparison of theoretical versus actual concrete quantities can also be helpful.

5.4.3.3 *Control of groundwater*—Water in the pier excavation can occur from loss of seal around the temporary casing (or surface inflow if no top casing is used), seepage from a granular saturated layer in the shaft or bell, or inflow from the bottom of the excavation. The geotechnical field representative should observe the water control methods and report on any inadequacies noted. For proper observation of bottom cleanup, testing of bearing material and proper placement of concrete, the water level in the bottom of the hole should be held to a depth not exceeding 2 in. (51 mm). Where the top flow cannot be adequately controlled, special procedures will be required (refer to Section 4.3.2). The special procedures adopted must have the specific approval of the architect or structural engineer and the geotechnical engineer. A representative coring of concrete placed by special techniques in or through water or slurry is often required to check the soundness and continuity of the concrete. Sonic and gamma logging techniques used in preplaced access tubes have also proven useful for this purpose.

When use of a slurry precludes physical observation of end bearing pier bottoms, definitive evidence of reaching the specified bearing material must be obtained by the use of a remote testing device, borings, special sampling and sounding techniques, or other means.

5.4.3.4 *Depths of pier*—The geotechnical representative must determine when the bearing material has been reached and when the depth of the pier is adequate. This determination can be made by appropriate tests on samples of the bearing material and correlation with the original design boring logs.

5.4.3.5 *Belled piers*—Bell bottoms should be reasonably flat. Downhole observation is advisable to check the bell shape, dimensions, and concentricity and verify that the bell roof and shaft walls are stable. The geotechnical field representative should make actual measurements to see that the bell meets the specification requirements. When shaft and bell entry is not considered safe, special remote or indirect measuring procedures are required as described in Section 5.4.3.9 and in the references.

5.4.3.6 *Cleanout*—Good cleanout of soft, loose, disturbed soil in the bottom of an end bearing straight shaft or belled pier should be obtained. The amount of spoil should not cover more than 10% of the bearing area to a maximum depth of 2 in. (51 mm). Downhole observation is necessary to see that the specified limitations are being met. It should be noted that some buckets tend to ride up on spoil as they cut the bell walls, and buildup of spoil can occur. If satisfactory bottom cleanup cannot be accomplished by the drilling equipment, hand cleanup by workers is necessary.

In some unfavorable soil and groundwater conditions, caving of bell roofs and soil movement into the bottom from seepage water can occur after initial cleanup. For these reasons, a final observation should be made just prior to the start of concrete placement. Any additional disturbed material should be removed at this time to the same quality of cleanup as that recommended above.

If it is considered unsafe by the contractor to send workers into the hole because of water seepage and running silt or an unstable bell roof condition, then special measures are required. These normally would be discussed with the geotechnical engineer and owner and be considered an extra cost and paid for as an extra. They may include casing off the problem area and belling below, if soil conditions permit, enlarging the bell under slurry, tremie-placing a cement grout to fill the bell, allowing the grout to set up overnight, and then redrilling the bell partly in the grout and partly below so that the remaining grout collar provides support for the unsuitable soil. The geotechnical field representative must carefully observe these special procedures to see that the design foundation criteria is accomplished.

In certain situations in which the bell undercut is small, direct construction of the design bell under controlled slurry (refer to Sections 4.4.1.2 and 4.4.1.5) may be permissible. However, in this case, bottom-cleaning with an airlift and sounding of the bottom to check the bottom condition prior to concreting are required.

5.4.3.7 *Bearing material*—In hard soil and soft rock formations, close observation and testing of the bearing material is important, and it requires experienced judgment. Upon reaching the bearing stratum and prior to belling and cleaning, a strength test or a penetrometer test should be made on the bottom material. One or more probe holes may be drilled below the design level where water or gas problems would not result therefrom and if useful information is expected. The use of probe holes and the information to be gained from them should be discussed at the preconstruction meeting prior to commencement of shaft excavation.

In clay soils, hardpan, or soft shales, tests should be made for shear strength in order to check that the required bearing capacity is met. Care should be exercised to make sure that the sampling and testing is performed on undisturbed material that is representative of the materials over the bearing area. It is good practice to make several tests over large bottom areas to establish the degree of uniformity of material and the representativeness of the original test samples.

When piers are placed on or in formations of expansive clay or shale, the loads on the bearing material must be sufficient to resist uplift as determined by appropriate tests, or the pier bearing level must be situated at sufficient depth to preclude that moisture content at that level will vary after construction. Surface sources of water must be avoided through proper drainage systems to prevent free water from percolating along the perimeter of the pier. Otherwise, the pier must be designed to resist uplift. The magnitude of uplift forces as well as the ability of the pier shaft and/or bell to resist the uplift should be determined by the geotechnical engineer.

Where the bearing material is granular and contains less clay, triaxial shear tests may be performed as part of the original exploration program to confirm strength properties and to avoid field delays. Also, in-place pressure meter tests, split-barrel penetration tests, Dutch Cone Penetration Test, or other specified suitable tests may provide meaningful data. Where the properties and range of variability of a bearing stratum have been adequately determined during the geotechnical exploration, visual corroboration of the bearing stratum may be sufficient. This is more cost-effective than time-consuming laboratory or in-place tests.

In some areas where cavernous rock conditions are present, a borehole at each drilled pier location may be needed in advance of pier installation.

Drilling of small-diameter probe holes into the bearing strata to provide information on the adequacy or uniformity of the bearing stratum with depth should be done with caution in clay or hardpan when water-bearing sand and silt layers or lenses are suspected within or below the bearing stratum. If such layers are penetrated, serious construction problems may result from infiltration and the washing of silt or sand into the pier. Probe holes are recommended only where soft layers are suspected underneath the bearing layer or when it is felt necessary to supplement original boring investigations. When probe holes indicate soft material below the bearing stratum, the pier should be extended deeper or undisturbed samples of the soft soils should be secured, appropriate laboratory tests made, and the results analyzed by the geotechnical engineer to determine the allowable bearing capacity of the pier, and the bell enlarged appropriately.

The need for probe holes should be minimized by performing more extensive exploration in advance of construction.

5.4.3.8 *Bearing material under slurry*—If construction of the pier is performed under controlled slurry so that physical bottom observation is not practical, the bottom can be checked by the geotechnical representative by putting his hand on the airlift pipe and "feeling" the bottom as the drill operator raises the pipe 6 in. (152 mm) off the bottom and then lets it drop. A sharp impact is felt when it hits the bottom if adequately cleaned. The geotechnical representative should also check the bottom with a wire-attached sounding rod. A 3/4 in. (19 mm) diameter bar about 3 ft (0.9 m) in length makes a good sounding rod. If desired, samples of the bottom can be obtained by attaching special split-barrel sampling tubes either to the drill rig Kelly bar or to the airlift pipe and then dropping them on the bottom with sufficient force to penetrate the bearing material.

5.4.3.9 *Bearing material hard rock formations*—It is often difficult to determine the necessary depth of excavation into the rock without a boring at each pier in advance. The geotechnical representative should make a serious effort to observe that the pier is founded on a sound formation and not on boulders. Soft, weathered and broken rock should be removed to hard solid formations unless the design is based on support directly on the weathered rock. In some cases, weathered rock may extend tens or hundreds of feet deep. Where piers are socketed into the rock and part of the load is carried by side resistance (refer to Section 3.6), the properties of the rock assumed in the design need to be known by the geotechnical representative so that the adequacy of the rock can be checked in the field. Sound, competent rock is normally required for the entire depth of the socket as well as for the bottom bearing material, except where engineering analysis indicates that this is not necessary.

Inspection holes are recommended to determine if seams of clay or other soft material exist in the rock close enough to the bearing level to affect the design of the pier. One to three probe holes are recommended, depending upon the size of the pier, the design bearing pressure, and the anticipated quality of rock. Additional probe holes may be made to determine the extent and thickness of seams before a decision is made to remove the rock down through the unacceptable layers. The probe holes can be drilled with an air drill and should extend into the rock a minimum depth equal to the shaft diameter. The speed of air drilling should be noted by the inspector and the sides of the drill hole checked with a feeler rod, a borescope, or TV camera to determine the location and size of cracks and clay seams. Tolerable settlement usually governs the permissible size and location of cracks and clay seams.

5.4.3.10 *Bearing material under water or slurry: hard formation*—If construction of the pier is performed while under water or controlled slurry so that bottom observation and probing are not practical, a probe hole should be made in the rock at each pier location prior to pier construction in order to assess rock quality and determine socket length and bearing elevation. "Sounding" of the bottom can be performed as indicated in Section 5.4.3.7.1.

5.4.3.11 *Pier inspections*—The constructor should check each pier hole for methane and carbon dioxide gas before entering. If gas exceeds acceptable limits, it must be purged before any personnel enter, or gas masks must be used. No smoking or welding should be permitted until the absence of gas is determined. If purging proves impractical so that inspection of the bottom is not possible, determine the bell size by observing the position of previously calibrated marking on the Kelly or bell bucket when the bell bucket is fully extended during belling. If there is any question about the adequacy of the mechanical cleanup at the bottom, the bell can be oversized and any remaining spoil back-bladed to the oversize area at the edge of the bell.

5.4.3.12 *Safety precautions*—All OSHA safety regulations should be followed for personnel entering a shaft for observation, or cleaning, or any other purpose.

5.5—Concreting

The following monitoring should be followed:

5.5.1 Observe if the reinforcing steel is clean and properly placed and whether bar sizes and lengths are as designed.

5.5.2 Observe the condition of the pier bottom just prior to placing of concrete. If sediment has accumulated on the bottom, reclean and recheck prior to concreting.

5.5.3 Visually observe to the extent practical the condition of the shaft walls or steel casing that will be in contact with the fresh concrete, and note the position of the water level behind the casing. The concrete should be placed immediately after these observations.

5.5.4 Visually observe to the extent practical the method of placing concrete in the pier to see that there is no segregation or contamination of the materials when such methods as freefall through a hopper, tremie pipe, or back chute are used.

5.5.5 Perform tests on fresh concrete such as slump, air content, and wet unit weight when required, and cast test specimens. It is recommended, in addition to a set of cylinders for each specific batch of concrete yardage placed (typically 50 to 100 yards or 1 day's pour, if less), one test cylinder be taken from every truckload and tested at 7 days to provide indicator information to detect potential problems.

5.5.6 Observe that concrete is placed continuously without interruption or long delays and that a sufficient head of concrete is maintained inside the casing to balance the head of water outside the casing. This will prevent inflow of water and soil quick condition into the fresh concrete as the casing is withdrawn. Compare the theoretical volume with the actual volume of concrete placed (refer to Section 4.3.6).

5.5.7 Observe the level of the concrete during the initial casing pull. If the concrete is observed to rise, immediately stop operations and make necessary corrections in the procedures and investigate damage caused by a potential discontinuity in the concrete.

5.5.8 The geotechnical field representative should be constantly alert for evidence that soil is included in the concrete by any mechanism.

5.5.8.1 *Soil cave-ins when placing concrete in uncased holes of questionable stability*—Close observation of the hole during concrete placement is necessary to hopefully detect a cave-in. During winter months, when there is an excess of vapor from the hot concrete, detection is more difficult. When a cave-in takes place, the pier must be cleaned out, cased, and the concrete placed again. It is always possible for a soil cave-in to go undetected. Placing in uncased holes should be avoided except in soils of reliable stability, such as cohesive soils of adequate strength. The constructor is responsible for constructing a continuous pier.

5.5.8.2 *Surface soil cave-in over top of casing*—This can occur only when the casing does not protrude sufficiently above ground surface or when there is a spoil pile too close to the pier location.

5.5.8.3 *Side soil cave-in as casing is pulled and concrete placed*—This results either from inadequate head of concrete in the casing at the time of pulling to balance the forces tending to cause soil cave-in, or it can occur if the concrete becomes "hung up" in the casing either by arching due to low slump or because of a fast set. The suction created as the casing and concrete are pulled tends to cause an influx of soil and water beneath the casing. The geotechnical field representative and the constructor should confirm that these conditions do not occur, or that they are corrected if they do occur. This can require redrilling the hole using proper procedures after deciding what went wrong.

5.5.9 When concrete is placed underwater either by tremie procedures or by pumping, the following observations should be carefully noted:

(a) Water level in the pier is static prior to concreting
(b) Adequate initial separation of concrete and water. Check the dimensions of the pig to see that the fit is close. If a plate is attached to the bottom of the pipe, check the release mechanism
(c) Placement of tremie or pump pipe on the bottom of the pier
(d) Pipe is fully charged with concrete before lifting pipe off bottom
(e) Monitor the level of concrete in the pier with a weighted tape and observe the position of the tremie or the pump pipe. Continue monitoring the relative position of concrete and pipe to see that the pipe is always in the concrete
(f) Compare calculated theoretical concrete volumes with actual placed volumes after placement

5.5.9.1 If concrete is placed under slurry without casing, and a specially designed slurry has been specified for a side resistance pier design, measure viscosity, density, and sand content of slurry to see that it meets specifications in addition to the procedures outlined in Section 5.5.9.

5.6—Exploration methods to determine soundness of piers

Critical soil or groundwater conditions or questionable construction methods may raise doubt about the integrity of certain piers on a project. The most frequently used method for determining the soundness of a pier is core drilling with associated probing on the sides of the core hole, TV inspection, borescope inspection, and sonic testing between two core holes. Sonic devices and techniques without core holes, such as surface reflection techniques or buried sonic devices in the bottom of the pier have also been used (Terzaghi and Peck 1967). However, sonic testing from the surface is unlikely to detect unsatisfactory bottom conditions.

For concrete placement underwater or with slurry using appropriate tremie or concrete-pumping procedures, a satisfactory and relatively convenient method for checking concrete integrity and quality after placement is to use either sonic-testing or gamma-logging techniques in preplaced access tubes extended to the base of the pier (Baker 1986).

5.7—Reports

Daily geotechnical field representative's reports, signed by the geotechnical field representative and drilled shaft constructor, should be summarized promptly for the architect, structural engineer, and general contractor. These reports should cover the following items when applicable:

(a) Accurate location and dimensions of pier holes bored
(b) Accurate top and bottom elevations; the degree of accuracy necessary should have been previously specified by the structural and the geotechnical engineer)
(c) Type of shaft-excavating methods used
(d) Description of materials encountered during excavation
(e) Description of groundwater conditions encountered
(f) Description, location, and dimensions of obstructions encountered and whether removal was attained
(g) Description of temporary or permanent casing placed, including purpose, length, wall thickness, and anchorage or seal obtained, if any
(h) Description of any soil and water movement, bell and wall stability, loss of ground, methods for control, and pumping requirements
(i) Data secured for all shaft, bell and shear ring measurements
(j) Description of cleanout methods and adequacy of initial cleanout
(k) Elevation at which bearing material encountered. Description of bearing material, probe holes made,

method of probing, rate of drilling in rock, samples taken, tests made, and conclusions reached with regard to adequacy of bearing material. (If inspection probing and testing is done independently by the geotechnical engineer, the conclusions of the geotechnical engineer should be included.)

(l) Description of adequacy of cleanout just prior to concrete placement

(m) Record of depth of water in hole and rate of water infiltration prior to concrete placement

(n) Record of reinforcing steel inspection for position and adequacy

(o) Method of concrete placement and removal of casing, if any. Record head of concrete during removal of casing. Record elevation of concrete when vibration started

(p) Record of any difficulties encountered. This should include possible soil inclusion, possible voids, possible shaft squeeze-in and possible casing collapse

(q) Condition of concrete delivered to site including record of slump, unit weight, air content, and other tests and cylinders made for compression testing

(r) Record of any deviations from the specifications and decisions required

5.8—Criteria for acceptance

The following minimum criteria for acceptance of completed piers should be included in the specifications. Additional or closer tolerances may be specified by the architect or structural engineer in the contract documents. Any variations should be brought to the attention of the architect, structural engineer, and geotechnical engineer prior to acceptance.

5.8.1 *Location and plumbness*—Unless compensated for in the structural design, permissible construction tolerances for plumbness should be in accordance with ACI 336.1. The tolerance on location at the top of the pier should not be more than 4% of the pier diameter or 3 in. (76 mm) in any direction, whichever is less. Tolerances smaller than 2 in. (51 mm) are difficult to obtain.

5.8.2 *Shafts, bells, and shear rings*—Measurements of shafts, bells, and shear rings should be made by the geotechnical field representative. The area of the shaft and bell bottom should not be less than 98% of that specified. In no case should the bell roof slope at an angle less than 45 degrees with the horizontal and the design edge should be at least 6 in. (152 mm) thick. The bell roof slope should be a straight line or be concave downward. In no case should it be concave upward by more than 3 or 6 in. (76 or 152 mm) thick per Section 3.7.1.

5.8.3 *Cleanout*—All loose material and spoil should be cleaned out of shafts and bells prior to placing concrete. In the case of end bearing piers, the volume of loose material and spoil should never exceed an amount that would cover 10% of the area to a depth of 2 in. (51 mm).

5.8.4 *Reinforcement*—Refer to ACI 117.

5.8.5 *Pier acceptability for concrete placement*—A pier should not be considered acceptable for concrete placement until the inspection procedures described in Section 5.4 have been accomplished as applicable.

5.9—Corrective measures

Should allowable tolerances be exceeded or should concrete be undersize or understrength, corrective measures described in Baker et al. (1971) may be used. There is no simple economical method to conclusively determine if an as-installed pier is defective although several methods are available that may be sufficiently reliable for certain low-stress situations (Davis 1991). If necessary, suspect piers can be load-tested by either conventional static means or by large strain dynamic testing (Baker et al. 1992). In the case of a shaft being off-location or out-of-plumb, if the out-of-tolerance deviation is discovered prior to concreting, it may be possible to correct most economically by increasing structural reinforcement of the pier and/or adding structural lateral restraint. This approach requires analysis by the structural engineer. If the out-of-tolerance is too severe to make this approach practical, it may be necessary to backfill the hole with a lean mix grout and to carefully redrill, periodically checking and correcting for plumb tolerance. For this case, the strength of the grout at the time of redrilling the hole should be designed to be reasonably comparable to the strength and hardness of the ground being drilled. Alternatively, it may be possible that the shaft can be redrilled sufficiently over-size that a design-size shaft of the correct size and plumbness falls within the overall dimensions of the new oversize-as-drilled shaft.

CHAPTER 6—REFERENCES

6.1—Recommended references

American Concrete Institute

117	Standard Specification for Tolerances for Concrete Construction and Materials
301	Specifications for Structural Concrete for Buildings
318	Building Code Requirements for Reinforced Concrete
318.1	Building Code Requirements for Structural Plain Concrete
336.1	Standard Specification for the Construction of Drilled Piers
364R	Guide for Measuring, Mixing, Transporting, and Placing Concrete

American Petroleum Institute

13A	Specification for Drilling-Fluid Materials
13B	Recommended Practice Standard Procedure for Field Testing Oil-Based Drilling Fluids

Deep Foundations Institute and the International Association of Foundation Drilling (ADSC)

Drilled Shaft Inspector's Manual—Prepared by the Joint Caisson-Drilled Shaft Committee, 1989

The above publications may be obtained from the following organizations:

American Concrete Institute
38800 Country Club Dr.
Farmington Hills, MI 48331

American Petroleum Institute
1220 L Street Northwest
Washington, DC 20005

Deep Foundations Institute
Box 281
Sparta, NJ 07871

6.2—Cited references

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