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GEOTECHNICAL ENGINEERING

In a broad sense, geotechnical engineering is that branch of civil engineering that employs scientific methods to determine, evaluate, and apply the interrelationship between the geologic environment and engineered works. In a practical context, geotechnical engineering encompasses evaluation, design, and construction involving earth materials.

The broad nature of this branch of civil engineering is demonstrated by the large number of technical committees comprising the Geo-Institute of the American Society of Civil Engineers (ASCE). In addition, the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE) includes the following 31 Technical Committees: Calcareous Sediments, Centrifuge and Physical Model Testing, Coastal Geotechnical Engineering, Deformation of Earth Materials, Earthquake Geotechnical Engineering, Education in Geotechnical Engineering, Environmental Geotechnics, Frost, Geophysical Site Characterization, Geosynthetics and Earth Reinforcement, Ground Improvement, Ground Property Characterization from In-situ Testing, Indurated Soils and Soft Rocks, Instrumentation for Geotechnical Monitoring, Landslides, Limit State Design in Geotechnical Engineering, Micro-geomechanics, Offshore Geotechnical Engineering, Peat and Organic Soils, Pile Foundations, Preservation of Historic Sites, Professional Practice, Risk Assessment and Management, Scour of Foundations, Soil Sampling Evaluation and Interpretation, Stress-Strain Testing of Geomaterials in the Laboratory, Tailings Dams, Tropical and Residual Soils, Underground Construction in Soft Ground, Unsaturated Soils, and Validation of Computer Simulations.

Unlike other civil engineering disciplines, which typically deal with materials whose properties are well defined, geotechnical engineering is concerned with subsurface materials whose properties, in general, cannot be specified. Pioneers of geotechnical engineering relied on the "observational approach" to develop an understanding of soil and rock mechanics and behavior of earth materials under loads. This approach was enhanced by the advent of electronic field instrumentation, wide availability of powerful personal computers, and development of sophisticated numerical techniques. These now make it possible to determine with greater accuracy the nonhomogeneous, nonlinear, anisotropic nature and behavior of earth materials for application to engineering works.

Geotechnical engineers should be proficient in the determination of soil and rock properties, engineering mechanics, subsurface investigation methods and laboratory testing techniques. They should have a thorough knowledge of design methods, construction methods, monitoring/inspection procedures, and specifications and contracting practices. Geotechnical engineers should have broad practical experience, in as much as the practice of geotechnical engineering involves art as much as science. This requirement was clearly expressed by

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Karl Terzaghi, who made considerable contributions to the development of soil mechanics: "The magnitude of the difference between the performance of real soils under field conditions and the performance predicted on the basis of theory can only be ascertained by field experience."

Geotechnical engineering is the engineering science of selecting, designing, and constructing features constructed of or upon soils and rock. Shallow foundations, deep foundations, earth retaining structures, soil and rock embankments and cuts are all specialty areas of geotechnical engineering.

Foundation engineering is the art of selecting, designing, and constructing for engineering works structural support systems based on scientific principles of soil and engineering mechanics and earth-structure interaction theories, and incorporating accumulated experience with such applications.

7.1 Lessons from Construction Claims and Failures

Unanticipated subsurface conditions encountered during construction are by far the largest source of construction-related claims for additional payment by contractors and of cost overruns. Failures of structures as a result of foundation deficiencies can entail even greater costs, and moreover jeopardize public safety. A large body of experience has identified consistently recurring factors contributing to these occurrences. It is important for the engineer to be aware of the causes of cost overruns, claims, and failures and to use these lessons to help minimize similar future occurrences.

Unanticipated conditions (changed conditions) are the result of a variety of factors. The most frequent cause is the lack of definition of the constituents of rock and soil deposits and their variation throughout the construction site. Related claims are for unanticipated or excessive quantities of soil and rock excavation, misrepresentation of the quality and depth of bearing levels, unsuitable or insufficient on-site borrow materials, and unanticipated obstructions to pile driving or shaft drilling. Misrepresentation of groundwater condition is another common contributor to work *extras* as well as to costly construction delays and emergency redesigns. Significant claims have also been generated by the failure of geotechnical investigations to identify natural hazards, such as swelling soils and rock minerals, unstable natural and cut slopes, and old fill deposits.

Failures of structures during construction are usually related to undesirable subsurface conditions not detected before or during construction, faulty design, or poor quality of work. Examples are foundations supported on expansive or collapsing soils, on solutioned rock, or over undetected weak or compressible subsoils; foundation designs too difficult to construct properly; foundations that do not perform as anticipated; and deficient construction techniques or materials. Another important design-related cause of failure is underestimation or lack of recognition of extreme loads associated with natural events, such as earthquakes, hurricanes, floods, and prolonged precipitation. Related failures include soil liquefaction during earthquakes, hydrostatic uplift or water damage to structures because of a rise in groundwater level, undermining of foundations by scour and overtopping, or wave erosion of earth dikes and dams.

It is unlikely that conditions leading to construction claims and failures can ever be completely precluded, inasmuch as discontinuities and extreme variation in subsurface conditions occur frequently in many types of soil deposits and rock formations. An equally important constraint that must be appreciated by both engineers and clients is the limitations of the current state of geotechnical engineering practice.

Mitigation of claims and failures, however, can be achieved by fully integrated geotechnical investigation, design, and construction quality assurance conducted by especially qualified professionals. Integration, rather than departmentalization of these services, ensures a continuity of purpose and philosophy that effectively reduces the risks associated with unanticipated subsurface conditions and design and construction deficiencies. It is also extremely important that owners and prime design professionals recognize that cost savings that reduce the quality of geotechnical services may purchase liabilities several orders of magnitude greater than their initial "savings."

7.2 Soil and Rock Classifications

All soils are initially the products of chemical alteration or mechanical disintegration of bedrock

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that has been exposed to weathering processes. Soil constituents may have been subsequently modified by transportation processes such as water, wind, and ice and by inclusion and decomposition of organic matter. Consequently, soil deposits may be given a geologic as well as a constitutive classification.

Rock types are broadly classified by their mode of formation into igneous, metamorphic, and sedimentary deposits. The supporting ability (quality) assigned to rock for design or analysis should reflect the degree of alteration of the rock minerals due to weathering, the frequency of discontinuities within the rock mass, and the susceptibility of the rock to deterioration upon exposure.

7.2.1 Geologic Classification of Soils

The classification of a soil deposit with respect to its mode of deposition and geologic history is an important step in understanding the variation in soil type and the maximum stresses imposed on the deposit since deposition. (A geologic classification that identifies the mode of deposition of soil deposits is shown in Table 7.1.) The geologic history of a soil deposit may also provide valuable information on the rate of deposition, the amount of erosion, and the tectonic forces that may have acted on the deposit subsequent to deposition.

Geological and agronomic soil maps and detailed reports are issued by the U.S. Department of Agriculture (www.usda.gov), U.S. Geological Survey (www.usgs.gov), and corresponding state offices. Old surveys are useful for locating original shore lines, stream courses, and surface-grade changes.

Table 7.1 Geologic Classification of Soil Deposits

Classification	Mode of Formation
Aeolian	
Dune	Wind deposition (coastal and desert)
Loess	Deposition during glacial periods
Alluvial	
Alluvium	River and stream deposition
Lacustrine	Lake waters, including glacial lakes
Floodplain	Floodwaters
Colluvial	
Colluvium	Downslope soil movement
Talus	Downslope movement of rock debris
Glacial	
Ground moraine	Deposited and consolidated by glaciers
Terminal moraine	Scour and transport at ice front
Outwash	Glacier melt waters
Marine	
Beach or bar	Wave deposition
Estuarine	River estuary deposition
Lagoonal	Deposition in lagoons
Salt marsh	Deposition in sheltered tidal zones
Residual	
Residual soil	Complete alteration by in situ weathering
Saprolite	Incomplete but intense alteration and leaching
Laterite	Complex alteration in tropical environment
Decomposed rock	Advanced alteration within parent rock

7.2.2 Unified Soil Classification System

This is the most widely used of the various constitutive classification systems and correlates soil type with generalized soil behavior. All soils are classified as coarse-grained (50% of the particles >0.074 mm), fine-grained (50% of the particles <0.074 mm), or predominantly organic (see Table 7.2).

Coarse-grained soils are categorized by their particle size into boulders (particles larger than 8 in), cobbles (3 to 8 in), gravel, and sand. For sands (S) and gravels (G), grain-size distribution is identified as either poorly graded (P) or well-graded (W), as indicated by the group symbol in Table 7.2. The presence of fine-grained soil fractions (under 50%), such as silt and clay, is indicated by the symbols M and C, respectively. Sands may also be classified as coarse (larger than No. 10 sieve), medium (smaller than No. 10 but larger than No. 40), or fine

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Table 7.2 Unified Soil Classification Including Identification and Description[a]

Major Division	Group Symbol	Typical Name	Field Identification Procedures[b]	Laboratory Classification Criteria[c]	
A. Coarse-grained soils (more than half of material larger than No. 200 sieve)[d]					
1. Gravels (more than half of coarse fraction larger than No. 4 sieve)[e]					
Clean gravels (little or no fines)	GW	Well-graded gravels, gravel-sand mixtures, little or no fines	Wide range in grain sizes and substantial amounts of all intermediate particle sizes	$D_{60}/D_{10} > 4$ $1 < D_{30}/D_{10}D_{60} < 3$ D_{10}, D_{30}, D_{60} = sizes corresponding to 10, 30, and 60% on grain-size curve	
	GP	Poorly graded gravels or gravel-sand mixtures, little or no fines	Predominantly one size, or a range of sizes with some intermediate sizes missing	Not meeting all gradation requirements for GW	
Gravels with fines (appreciable amount of fines)	GM	Silty gravels, gravel-sand-silt mixtures	Nonplastic fines or fines with low plasticity (see ML soils)	Atterberg limits below A line or PI < 4	Soils above A line with 4 < PI < 7 are borderline cases, require use of dual symbols
	GC	Clayey gravels, gravel-sand-clay mixtures	Plastic fines (see CL soils)	Atterberg limits above A line with PI > 7	
2. Sands (more than half of coarse fraction smaller than No. 4 sieve)[e]					
Clean sands (little or no fines)	SW	Well-graded sands, gravelly sands, little or no fines	Wide range in grain sizes and substantial amounts of all intermediate particle sizes	$D_{60}/D_{10} > 6$ $1 < D_{30}/D_{10}D_{60} < 3$	
	SP	Poorly graded sands or gravel-sands, little or no fines	Predominantly one size, or a range of sizes with some intermediate sizes missing	Not meeting all gradation requirements for SW	
Sands with fines (appreciable amount of fines)	SM	Silty sands, sand-silt mixtures	Nonplastic fines or fines with low plasticity (see ML soils)	Atterberg limits below A line or PI < 4	Soils with Atterberg limits above A line while 4 < PI < 7 are borderline cases; require use of dual symbols
	SC	Clayey sands, sand-clay mixtures	Plastic fines (see CL soils)	Atterberg limits above A line with PI > 7	

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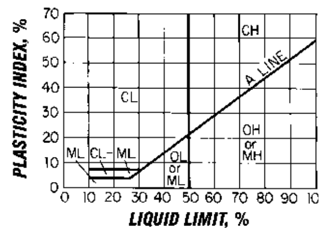
Information required for describing coarse-grained soils:

For undisturbed soils, add information on stratification, degree of compactness, cementation, moisture conditions, and drainage characteristics. Give typical name; indicate approximate percentage of sand and gravel; maximum size, angularity, surface condition, and hardness of the coarse grains; local or geological name and other pertinent descriptive information; and symbol in parentheses. Example: *Salty sand*, gravelly; about 20% hard, angular gravel particles, ½ in maximum size; rounded and subangular sand grains, coarse to fine; about 15% nonplastic fines with low dry strength; well compacted and moist in place; alluvial sand; (SM).

B. Fine-grained soils (more than half of material larger than No. 200 sieve)[d]

Major Division	Group Symbol	Typical Name	Identification Procedures[f]			Laboratory Classification Criteria[c]
			Dry Strength (Crushing Characteristics)	Dilatancy (Reaction to Shaking)	Toughness (Consistency Near PL)	
Silts and clays with liquid limit less than 50	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts with slight plasticity	None to slight	Quick to slow	None	
	CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	Medium to high	None to very slow	Medium	
	OL	Organic silts and organic silty clays of low plasticity	Slight to medium	Slow	Slight	
Silts and clays with liquid limit more than 50	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts	Slight to medium	Slow to none	Slight to medium	Plasticity chart laboratory classifications of fine-grained soils compares them at equal liquid limit. Toughness and dry strength increase with increasing plasticity index (PI)
	CH	Inorganic clays of high plasticity, fat clays	None to very high	None	High	
	CH	Organic clays of medium to high plasticity	Medium to high	None to very slow	Slight to medium	

(*Table continued*)

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Table 7.2 *(Continued)*

C. Highly organic soils		
Pt	Peat and other highly organic soils	Readily identified by color, odor, spongy feel, and often by fibrous texture

Field identification procedures for fine-grained soils or fractions:[g]

Dilatancy (Reaction to Shaking)	Dry Strength (Crushing Characteristics)	Toughness (Consistency Near PL)
After removing particles larger than No. 40 sieve, prepare a pat of moist soil with a volume of about ½ in^3. Add enough water if necessary to make the soil soft but not sticky. Place the pat in the open palm of one hand and shake horizontally, striking vigorously against the other hand several times. A positive reaction consists of the appearance of water on the surface of the pat, which changes to a livery consistency and becomes glossy. When the sample is squeezed between the fingers, the water and gloss disappear from the surface, the pat stiffens, and finally it cracks or crumbles. The rapidity of appearance of water during shaking and of its disappearance during squeezing assist in identifying the character of the fines in a soil. Very fine clean sands give the quickest and most distinct reaction, whereas a plastic clay has no reaction. Inorganic silts, such as a typical rock flour, show a moderately quick reaction.	After removing particles larger than No. 40 sieve, mold a pat of soil to the consistency of putty, adding water if necessary. Allow the pat to dry completely by oven, sun, or air drying, then test its strength by breaking and crumbling between the fingers. This strength is a measure of character and quantity of the colloidal fraction contained in the soil. The dry strength increases with increasing plasticity. High dry strength is characteristic of clays of the CH group. A typical inorganic silt possesses only very slight dry strength. Silty fine sands and silts have about the same slight dry strength but can be distinguished by the feel when powdering the dried specimen. Fine sand feels gritty, whereas a typical silt has the smooth feel of flour.	After particles larger than the No. 40 sieve are removed, a specimen of soil about ½ in^3 in size is molded to the consistency of putty. If it is too dry, water must be added. If it is too sticky, the specimen should be spread out in a thin layer and allowed to lose some moisture by evaporation. Then, the specimen is rolled out by hand on a smooth surface or between the palms into a thread about ⅛ in in diameter. The thread is then folded and rerolled repeatedly. During this manipulation, the moisture content is gradually reduced and the specimen stiffens, finally loses its plasticity, and crumbles when the plastic limit (PL) is reached. After the thread crumbles, the pieces should be lumped together and a slight kneading action continued until the lump crumbles. The tougher the thread near the PL and the stiffer the lump when it finally crumbles, the more potent is the colloidal clay fraction in the soil. Weakness of the thread at the PL and quick loss of coherence of the lump below the PL indicate either organic clay of low plasticity or materials such as kaolin-type clays and organic clays that occur below the *A* line. Highly organic clays have a very weak and spongy feel at PL.

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Information required for describing fine-grained soils:

For undisturbed soils, add information on structure, stratification, consistency in undisturbed and remolded states, moisture, and drainage conditions. Give typical name; indicate degree and character of plasticity; amount and maximum size of coarse grains; color in wet conditions, odor, if any; local or geological name and other pertinent descriptive information; and symbol in parentheses. Example: *Clayey silt*, brown; slightly plastic; small percentage of fine sand; numerous vertical root holes; firm and dry in place; loess; (ML)

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[a]Adapted from recommendations of U.S. Army Corps of Engineers and U.S. Bureau of Reclamation. All sieve sizes United States standard.

[b]Excluding particles larger than 3 in and basing fractions on estimated weights.

[c]Use grain-size curve in identifying the fractions as given under field identification.

For coarse-grained soils, determine percentage of gravel and sand from grain-size curve. Depending on percentage of fines (fractions smaller than No. 200 sieve), coarse-grained soils are classified as follows:

Less than 5% fines:	GW, GP, SW, SP
More than 12% fines:	GM, GC, SM, SC
5% to 12% fines:	Borderline cases requiring use of dual symbols

Soils possessing characteristics of two groups are designated by cobinations of group symbols; for example, GW-GC indicates a well-graded, gravel-sand mixture with clay binder.

[d]The No. 200 sieve size is about the smallest particle visible to the naked eye.

[e]For visual classification, the $\frac{1}{4}$ in size may be used as equivalent to the No. 4 sieve size.

[f]Applicable to fractions smaller than No. 40 sieve.

[g]These procedures are to be performed on the minus 40-sieve-size particles (about $\frac{1}{64}$ in).

For field classification purposes, screening is not intended. Simply remove by hand the coarse particles that interfere with the tests.

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(smaller than No. 40). Because properties of these soils are usually significantly influenced by relative density D_r, rating of the in situ density and D_r is an important consideration (see Art. 7.4).

Fine-grained soils are classified by their liquid limit and plasticity index as organic clays OH or silts OL, inorganic clays CH or CL, or silts or sandy silts MH or ML, as shown in Table 7.2. For the silts and organic soils, the symbols H and L denote a high and low potential compressibility rating; for clays, they denote a high and low plasticity. Typically, the consistency of cohesive soils is classified from pocket penetrometer or Torvane tests on soil samples. These index tests are convenient for relative comparisons but do not provide design strength values and should not be used as property values for design or analysis. The consistency ratings are expressed as follows:

Soft—under 0.25 tons/ft^2

Firm—0.25 to 0.50 tons/ft^2

Stiff—0.50 to 1.0 tons/ft^2

Very stiff—1.0 to 2.0 tons/ft^2

Hard—more than 2.0 tons/ft^2

7.2.3 Rock Classification

Rock, obtained from core samples, is commonly characterized by its type, degree of alteration (weathering), and continuity of the core. (Where outcrop observations are possible, rock structure may be mapped.) Rock-quality classifications are typically based on the results of compressive strength tests or the condition of the core samples, or both. Rock types typical of igneous deposits include basalt, granite, diorite, rhyolite, and andesite. Typical metamorphic rocks include schist, gneiss, quartzite, slate, and marble. Rocks typical of sedimentary deposits include shale, sandstone, conglomerate, and limestone.

Rock structure and degree of fracturing usually control the behavior of a rock mass that has been significantly altered by weathering processes. It is necessary to characterize both regional and local structural features that may influence design of foundations, excavations, and underground openings in rock. Information from geologic publications and maps are useful for defining regional trends relative to the orientation of bedding, major joint systems, faults, and so on.

Rock-quality indices determined from inspection of rock cores include the fracture frequency (*FF*) and rock-quality designation (*RQD*). *FF* is the number of naturally occurring fractures per foot of core run, whereas *RQD* is the cumulative length of naturally separated core pieces, 4 in or more in dimension, expressed as a percentage of the length of core run. The rock-quality rating also may be based on the velocity index obtained from laboratory and in situ seismic-wave-propagation tests. The velocity index is given by $(V_s/V_l)^2$, where V_s and V_l represent seismic-wave velocities from in situ and laboratory core measurements, respectively. Proposed *RQD* and velocity index rock-quality classifications and in situ deformability correlations are in Table 7.3. A relative-strength rating of the quality rock cores representative of the intact elements of the rock mass, proposed by Deere and Miller, is based on the uniaxial compressive (*UC*) strength of the core and its tangent modulus at one-half of the *UC*.

(D. U. Deere and R. P. Miller, "Classification and Index Properties for Intact Rock," Technical Report AFWL-TR-65-116, Airforce Special Weapons Center, Kirtland Airforce Base, New Mexico, 1966.)

Inasmuch as some rocks tend to disintegrate rapidly (slake) upon exposure to the atmosphere, the potential for slaking should be rated from laboratory tests. These tests include emersion in water, Los Angeles abrasion, repeated wetting and drying, and other special tests, such as a

Table 7.3 Rock-Quality Classification and Deformability Correlation

Classification	RQD	Velocity Index	Deformability E_d/E_t*
Very poor	0–25	0–0.20	Under 0.20
Poor	25–50	0.20–0.40	Under 0.20
Fair	50–75	0.40–0.60	0.20–0.50
Good	75–90	0.60–0.80	0.50–0.80
Excellent	90–100	0.80–1.00	0.80–1.00

*E_d = in situ deformation modulus of rock mass; E_t = tangent modulus at 50% of *UC* strength of core specimens.

Source: Deere, Patton and Cording, "Breakage of Rock," *Proceedings*, 8th Symposium on Rock Mechanics, American Institute of Mining and Metallurgical Engineers, Minneapolis, Minn.

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slaking-durability test. Alteration of rock minerals due to weathering processes is often associated with reduction in rock hardness and increase in porosity and discoloration. In an advanced stage of weathering, the rock may contain soil-like seams, be easily abraded (friable), readily broken, and may (but will not necessarily) exhibit a reduced *RQD* or *FF*. Rating of the degree of rock alteration when logging core specimens is a valuable aid in assessing rock quality.

7.3 Physical Properties of Soils

Basic soil properties and parameters can be subdivided into physical, index, and engineering categories. Physical soil properties include density, particle size and distribution, specific gravity, and water content.

The **water content** w of a soil sample represents the weight of free water contained in the sample expressed as a percentage of its dry weight.

The **degree of saturation** S of the sample is the ratio, expressed as percentage, of the volume of free water contained in a sample to its total volume of voids V_v.

Porosity n, which is a measure of the relative amount of voids, is the ratio of void volume to the total volume V of soil:

$$n = \frac{V_v}{V} \tag{7.1}$$

The ratio of V_v to the volume occupied by the soil particles V_s defines the **void ratio** e. Given e, the degree of saturation may be computed from

$$S = \frac{wG_s}{e} \tag{7.2}$$

where G_s represents the specific gravity of the soil particles. For most inorganic soils, G_s is usually in the range of 2.67 ± 0.05.

The dry unit weight γ_d of a soil specimen with any degree of saturation may be calculated from

$$\gamma_d = \frac{\gamma_w G_s S}{1 + wG_s} \tag{7.3}$$

where γ_w is the unit weight of water and is usually taken as 62.4 lb/ft^3 for fresh water and 64.0 lb/ft^3 for seawater.

The particle-size distribution (gradation) of soils can be determined by mechanical (sieve) analysis and combined with hydrometer analysis if the sample contains a significant amount of particles finer than 0.074 mm (No. 200 sieve). The soil particle gradation in combination with the maximum, minimum, and in situ density of cohesionless soils can provide useful correlations with engineering properties (see Arts. 7.4 and 7.52).

7.4 Index Parameters for Soils

Index parameters of cohesive soils include liquid limit, plastic limit, shrinkage limits, and activity. Such parameters are useful for classifying cohesive soils and providing correlations with engineering soil properties.

The **liquid limit** of cohesive soils represents a near liquid state, that is, an undrained shear strength about 0.01 lb/ft^2. The water content at which the soil ceases to exhibit plastic behavior is termed the **plastic limit**. The **shrinkage limit** represents the water content at which no further volume change occurs with a reduction in water content. The most useful classification and correlation parameters are the plasticity index I_p, the liquidity index I_l, the shrinkage index I_s, and the activity A_c. These parameters are defined in Table 7.4.

Relative density D_r of cohesionless soils may be expressed in terms of void ratio e or unit dry weight γ_d:

$$D_r = \frac{e_{\max} - e_o}{e_{\max} - e_{\min}} \tag{7.4a}$$

$$D_r = \frac{1/\gamma_{\min} - 1/\gamma_d}{1/\gamma_{\min} - 1/\gamma_{\max}} \tag{7.4b}$$

D_r provides cohesionless soil property and parameter correlations, including friction angle, permeability, compressibility, small-strain shear modulus, cyclic shear strength, and so on.

In situ field tests such as the Standard Penetration Test (SPT), static cone penetrometer, pressuremeter, and dilatometer can also be used to determine the index properties of cohensionless and cohensive soils.

(H. Y. Fang, "Foundation Engineering Handbook," 2nd ed., Van Nostrand Reinhold, New York.)

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Table 7.4 Soil Indices

Index	Definition*	Correlation
Plasticity	$I_p = W_l - W_p$	Strength, compressibility, compactibility, and so forth
Liquidity	$I_l = \dfrac{W_n - W_p}{I_p}$	Compressibility and stress rate
Shrinkage	$I_s = W_p - W_s$	Shrinkage potential
Activity	$A_c = \dfrac{I_p}{\mu}$	Swell potential, and so on

*W_l = liquid limit; W_p = plastic limit; W_n = moisture content, %; W_s = shrinkage limit; μ = percent of soil finer than 0.002 mm (clay size).

7.5 Engineering Properties of Soils

Engineering soil properties and parameters describe the behavior of soil under induced stress and environmental changes. Of interest to most geotechnical applications are the strength, deformability, and permeability of in situ and compacted soils. ASTM promulgates standard test procedures for soil properties and parameters.

7.5.1 Shear Strength of Cohesive Soils

The undrained shear strength c_u, of cohesive soils under static loading can be determined by several types of laboratory tests, including uniaxial compression, triaxial compression (*TC*) or extension (*TE*), simple shear, direct shear, and torsion shear. The objective of soil laboratory testing is to replicate the field stress, loading and drainage conditions with regard to magnitude, rate and orientation. All laboratory strength testing require extreme care in securing, transporting and preparing the test sample. The triaxial test is the most versatile but yet the most complex strength test to perform. Triaxial tests involve application of a controlled confining pressure σ_3 and axial stress σ_1 to a soil specimen. σ_3 may be held constant and σ_1 increased to failure (*TC* tests), or σ_1 may be held constant while σ_3 is decreased to failure (*TE* tests). Specimens may be sheared in a drained or undrained condition.

The unconsolidated-undrained (*UU*) triaxial compression test is appropriate and commonly used for determining the c_u of relatively good-quality samples. For soils that do not exhibit changes in soil structure under elevated consolidation pressures, consolidated-undrained (*CU*) tests following the SHANSEP testing approach mitigate the effects of sample disturbance.

(C. C. Ladd and R. Foott, "New Design Procedures for Stability of Soft Clays," *ASCE Journal of Geotechnical Engineering Division*, vol. 99, no. GT7, 1974, www.asce.org.)

For cohesive soils exhibiting a normal clay behavior, a relationship between the normalized undrained shear strength c_u/σ'_{vo} and the overconsolidation ratio *OCR* can be defined independently of the water content of the test specimen by

$$\frac{c_u}{\sigma'_{vo}} = K(OCR)^n \tag{7.5}$$

where c_u is normalized by the preshear vertical effective stress, the effective overburden pressure σ'_{vo}, or the consolidation pressure σ'_{lc} triaxial test conditions. *OCR* is the ratio of preconsolidation pressure to overburden pressure. The parameter K represents the c_u/σ'_{vo} of the soil in a normally consolidated state, and n primarily depends on the type of shear test. For *CU* triaxial compression tests, K is approximately 0.32 ± 0.02 and is lowest for low plasticity soils and is a maximum for soils with plasticity index I_p over 40%. The exponent n is usually within the range of 0.70 ± 0.05 and tends to be highest for *OCR* less than about 4.

In situ vane shear tests also are often used to provide c_u measurements in soft to firm clays. Tests are commonly made on both the undisturbed and remolded soil to investigate the **sensitivity**, the ratio of the undisturbed to remolded soil strength. This test is not applicable in sand or silts or where

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hard inclusions (nodules, shell, gravel, and so forth) may be present. (See also Art. 7.6.3.)

The Standard Penetration Test, cone penetrometer, pressuremeter, and dilatometer provide guidance on engineering properties of soils. Similar to laboratory tests ASTM procedures have been developed for several of these tests. Each test, similar to the vane shear test, has advantages and disadvantages, as well as being limited to certain soil types.

Drained shear strength of cohesive soils is important in design and control of construction embankments on soft ground as well as in other evaluations involving effective-stress analyses. Conventionally, drained shear strength τ_f is expressed by the Mohr-Coulomb failure criteria as:

$$\tau_f = c' + \sigma_n' \tan \phi' \tag{7.6}$$

The c' and ϕ' parameters represent the effective cohesion and effective friction angle of the soil, respectively. σ_n' is the effective stress normal to the plane of shear failure and can be expressed in terms of total stress σ_n as $(\sigma_n - u_e)$, where u_e is the excess pore-water pressure at failure. u_e is induced by changes in the principal stresses $(\Delta\sigma_1, \Delta\sigma_3)$. For saturated soils, it is expressed in terms of the pore-water-pressure parameter A_f at failure as:

$$u = \Delta\sigma_3 + A_f(\Delta\sigma_1 - \Delta\sigma_3)_f \tag{7.7}$$

The effective-stress parameters c', ϕ', and A_f are readily determined by *CU* triaxial shear tests employing pore-water-pressure measurements or, excepting A_f, by consolidated-drained (CD) tests.

After large movements along preformed failure planes, cohesive soils exhibit a significantly reduced (residual) shear strength. The corresponding effective friction angle ϕ_r' is dependent on I_p. For many cohesive soils, ϕ_r' is also a function of σ_n'. The ϕ_r' parameter is applied in analysis of the stability of soils where prior movements (slides) have occurred.

Cyclic loading with complete stress reversals decreases the shearing resistance of saturated cohesive soils by inducing a progressive buildup in pore-water pressure. The amount of degradation depends primarily on the intensity of the cyclic shear stress, the number of load cycles, the stress history of the soil, and the type of cyclic test used. The strength degradation potential can be determined by postcyclic, *UU* tests.

7.5.2 Shear Strength of Cohesionless Soils

The shear strength of cohesionless soils under static loading can be interpreted from results of drained or undrained *TC* tests incorporating pore-pressure measurements. The effective angle of internal friction ϕ' can also be expressed by Eq. (7.6), except that c' is usually interpreted as zero. For cohesionless soils, ϕ' is dependent on density or void ratio, gradation, grain shape, and grain mineralogy. Markedly stress-dependent, ϕ' decreases with increasing σ_n', the effective stress normal to the plane of shear failure.

In situ cone penetration tests in sands may be used to estimate ϕ' from cone resistance q_c records. One approach relates the limiting q_c values directly to ϕ'. Where q_c increases approximately linearly with depth, ϕ' can also be interpreted from the slope of the curve for $q_c - \sigma_{vo}$ vs. ϕ_{vo}', where σ_{vo} = total vertical stress, $\sigma_{vo}' = \sigma_{vo} - u$, and u = pore-water pressure. The third approach is to interpret the relative density D_r from q_c and then relate ϕ' to D_r as a function of the gradation and grain shape of the sand.

Relative density provides good correlation with ϕ' for a given gradation, grain shape, and normal stress range. A widely used correlation is shown in Fig. 7.1. D_r can be interpreted from standard penetration resistance tests (Fig. 7.12) and cone penetration resistance tests (see Arts. 7.6.2 and 7.6.3) or calculated from the results of in situ or maximum and minimum density tests. The most difficult property to determine in the relative density equation is e_0, the in-situ void ratio.

Dense sands typically exhibit a reduction in shearing resistance at strains greater than those required to develop the peak resistance. At relatively large strains, the stress-strain curves of loose and dense sands converge. The void ratio at which there is no volume change during shear is called the critical void ratio. A volume increase during shear (dilatancy) of saturated, dense, cohesionless soils produces negative pore-water pressures and a temporary increase in shearing resistance. Subsequent dissipation of negative pore-water pressure accounts for the "relaxation effect" sometimes observed after piles have been driven into dense, fine sands.

Saturated, cohesionless soils subject to cyclic loads exhibit a significant reduction in strength if cyclic loading is applied at periods smaller than the time required to achieve significant dissipation of pore pressure. Should the number of load cycles N_c

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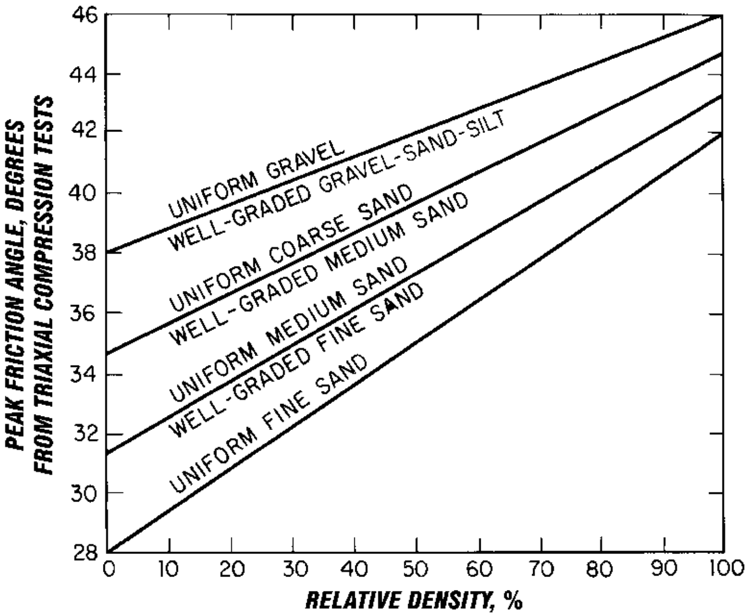

Fig. 7.1 Chart for determining friction angles for sands. (*After J. H. Schmertmann.*)

be sufficient to generate pore pressures that approach the confining pressure within a soil zone, excessive deformations and eventually failure (liquefaction) is induced. For a given confining pressure and cyclic stress level, the number of cycles required to induce initial liquefaction N_{c1} increases with an increase in relative density D_r. Cyclic shear strength is commonly investigated by cyclic triaxial tests and occasionally by cyclic, direct, simple-shear tests.

7.5.3 State of Stress of Soils

Assessment of the vertical σ'_{vo} and horizontal σ'_{ho} effective stresses within a soil deposit and the maximum effective stresses imposed on the deposit since deposition σ'_{vm} is a general requirement for characterization of soil behavior. The ratio $\sigma'_{vm}/\sigma'_{vo}$ is termed the **overconsolidation ratio** (OCR). Another useful parameter is the ratio of $\sigma'_{ho}/\sigma'_{vo}$, which is called the **coefficient of earth pressure at rest** (K_o).

For a simple gravitation piezometric profile, the effective overburden stress σ'_{vo} is directly related to the depth of groundwater below the surface and the effective unit weight of the soil strata. Groundwater conditions, however, may be characterized by irregular piezometric profiles that cannot be modeled by a simple gravitational system. For these conditions, **sealed piezometer measurements** are required to assess σ'_{vo}.

Maximum Past Consolidation Stress ▪ The maximum past consolidation stress σ'_{vm} of a soil deposit may reflect stresses imposed prior to geologic erosion or during periods of significantly lower groundwater, as well as desiccation effects and effects of human activity (excavations). The maximum past consolidation stress is conventionally interpreted from consolidation (oedometer) tests on undisturbed samples.

Normalized-shear-strength concepts provide an alternate method for estimating *OCR* from good-quality *UU* compression tests. In the absence of site-specific data relating c_u/σ'_{vo} and *OCR*, a form of Eq. (7.5) may be applied to estimate *OCR*. In this interpretation, σ'_{vo} represents the effective overburden pressure at the depth of the *UU*-test sample. A very approximate estimate of σ'_{vm} can also be obtained for cohesive soils from relationships proposed between liquidity index and effective vertical stress ("Design Manual—Soil

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Mechanics, Foundations, and Earth Structures," NAVDOCKS DM-7, U.S. Navy). For coarse-grained soil deposits, it is difficult to characterize σ'_{vm} reliably from either in situ or laboratory tests because of an extreme sensitivity to disturbance.

The coefficient of earth pressure at rest K_o can be determined in the laboratory from "no-lateral strain" *TC* tests on undisturbed soil samples or from consolidation tests conducted in specially constructed oedometers. Interpretation of K_o from in situ CPT, PMT, and dilatometer tests has also been proposed. In view of the significant impact of sample disturbance on laboratory results and the empirical nature of in situ test interpretations, the following correlations of K_o with friction angle ϕ' and OCR are useful. For both coarse- and fine-grained soils:

$$K_o = (1 - \sin\phi')OCR^m \tag{7.8}$$

A value for m of 0.5 has been proposed for overconsolidated cohesionless soils, whereas for cohesive soils it is proposed that m be estimated in terms of the plasticity index I_p as $0.581 I_p^{-0.12}$.

7.5.4 Deformability of Fine-Grained Soils

Deformations of fine-grained soils can be classified as those that result from volume change, (elastic) distortion without volume change, or a combination of these causes. Volume change may be a one-dimensional or, in the presence of imposed shear stresses, a three-dimensional mechanism and may occur immediately or be time-dependent. **Immediate deformations** are realized without volume change during undrained loading of saturated soils and as a reduction of air voids (volume change) within unsaturated soils.

The rate of volume change of saturated, fine-grained soils during loading or unloading is controlled by the rate of pore-fluid drainage from or into the stressed soil zone. The compression phase of delayed volume change associated with pore-pressure dissipation under a constant load is termed **primary consolidation**. Upon completion of primary consolidation, some soils (particularly those with a significant organic content) continue to decrease in volume at a decreasing rate. This response is usually approximated as a straight line for a plot of log time vs. compression and is termed **secondary compression**.

As the imposed shear stresses become a substantial fraction of the undrained shear strength of the soil, time-dependent deformations may occur under constant load and volume conditions. This phenomenon is termed **creep deformation**. Failure by creep may occur if safety factors are insufficient to maintain imposed shear stresses below the creep threshold of the soil. (Also see Art. 7.10.)

One-dimensional volume-change parameters are conveniently interpreted from consolidation (oedometer) tests. A typical curve for log consolidation pressure vs. volumetric strain ε_v (Fig. 7.2) demonstrates interpretation of the strain-referenced compression index C'_c, recompression index C'_r, and swelling index C'_s. The secondary compression index C'_a represents the slope of the near-linear portion of the volumetric strain vs. log-time curve following primary consolidation (Fig. 7.2*b*). The parameters C'_c, C'_r, and C'_a be roughly estimated from soil-index properties.

Deformation moduli representing three-dimensional deformation can be interpreted from the stress-strain curves of laboratory shear tests for application to either volume change or elastic deformation problems.

("Design Manual—Soil Mechanics, Foundations, and Earth Structures," NAVDOCKS DM-7, U.S. Navy; T. W. Lambe and R. V. Whitman, "Soil Mechanics," John Wiley & Sons, Inc., New York, www.wiley.com.)

7.5.5 Deformability of Coarse-Grained Soils

Deformation of most coarse-grained soils occurs almost exclusively by volume change at a rate essentially equivalent to the rate of stress change. Deformation moduli are markedly nonlinear with respect to stress change and dependent on the initial state of soil stress. Some coarse-grained soils exhibit a delayed volume-change phenomenon known as **friction lag**. This response is analogous to the secondary compression of fine-grained soils and can account for a significant amount of the compression of coarse-grained soils composed of weak or sharp-grained particles.

The laboratory approach previously described for derivation of drained deformation parameters for fine-grained soils has a limited application for coarse-grained soils because of the difficulty in

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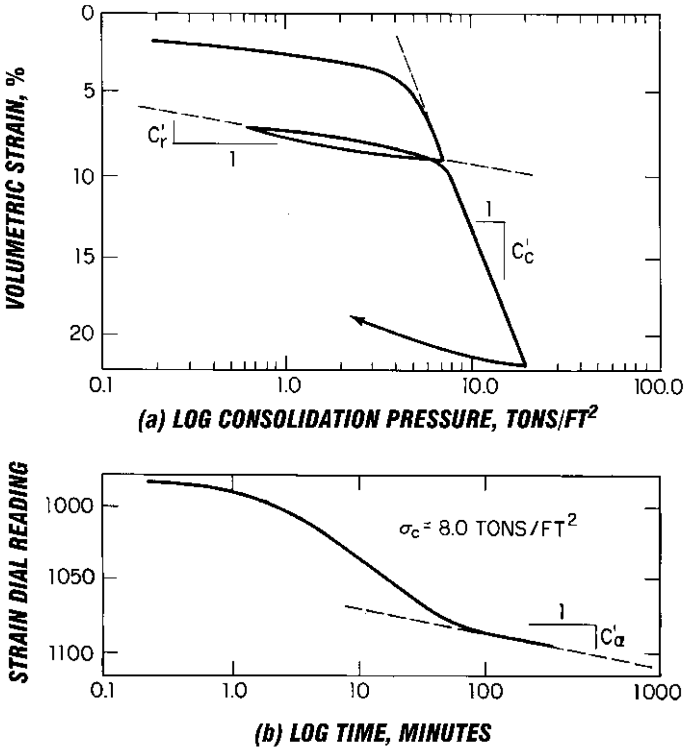

Fig. 7.2 Typical curves plotted from data obtained in consolidation tests.

obtaining reasonably undisturbed samples. Tests may be carried out on reinstituted samples but should be used with caution since the soil fabric, *aging*, and stress history cannot be adequately simulated in the laboratory. As a consequence, in situ testing techniques are often the preferred investigation and testing approach to the characterization of cohesionless soil properties (see Art. 7.6.3).

The Static Cone Penetration Test (CPT) ▪ The CPT is one of the most useful in situ tests for investigating the deformability of cohesionless soils. The secant modulus E'_s, tons/ft^2, of sands has been related to cone resistance q_c by correlations of small-scale plate load tests and load tests on footings. The relationship is given by Eq. (7.9*a*). The empirical correlation coefficient α in Eq. (7.9*a*) is influenced by the relative density, grain characteristics, and stress history of the soil (see Art. 7.6.3).

$$E'_s = \alpha q_c \tag{7.9a}$$

The α parameter has been reported to range between 1.5 and 3 for sands and can be expressed in terms of relative density D_r as $2(1 + D_r^2)$. α may also be derived from correlations between q_c and standard penetration resistance N by assuming that q_c/N for mechanical cones or $q_c/N + 1$ for electronic type cone tips is about 6 for sandy gravel, 5 for gravelly sand, 4 for clean sand, and 3 for sandy silt. However, it should be recognized that E'_s characterizations from q_c or N are empirical and can provide erroneous characterizations. Therefore, the validity of these relationships should be confirmed by local correlations. Cone penetration test soundings should be conducted in accordance with ASTM D-3441 (Briaud, J. L. and Miran J. (1992). "The Cone Penetrometer Test", Federal Highway Administration, FHWA Report No. SA-91-043, Washington D.C; See also Art. 7.13)

Load-Bearing Test ▪ One of the earliest methods for evaluating the in situ deformability

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of coarse-grained soils is the small-scale *load-bearing test*. Data developed from these tests have been used to provide a scaling factor to express the settlement ρ of a full-size footing from the settlement ρ_1 of a 1-ft^2 plate. This factor ρ/ρ_1 is given as a function of the width B of the full-size bearing plate as:

$$\frac{\rho}{\rho_1} = \left(\frac{2B}{1+B}\right)^2 \tag{7.10}$$

From an elastic half-space solution, E'_s can be expressed from results of a plate load test in terms of the ratio of bearing pressure to plate settlement k_v as:

$$E'_s = \frac{k_v(1-\mu^2)\pi/4}{4B/(1+B)^2} \tag{7.9b}$$

μ represents Poisson's ratio, usually considered to range between 0.30 and 0.40. Equation (7.9*b*) assumes that ρ_1 is derived from a rigid, 1-ft-diameter circular plate and that B is the equivalent diameter of the bearing area of a full-scale footing. Empirical formulations such as Eq. (7.10) may be significantly in error because of the limited footing-size range used and the large scatter of the data base. Furthermore, consideration is not given to variations in the characteristics and stress history of the bearing soils.

Pressuremeter tests (**PMTs**) in soils and soft rocks have been used to characterize E'_s from radial pressure vs. volume-change data developed by expanding a cylindrical probe in a drill hole (see Art. 7.6.3). Because cohesionless soils are sensitive to comparatively small degrees of soil disturbance, proper access-hole preparation is critical.

(K. Terzaghi and R. B. Peck, "Soil Mechanics and Engineering Practice," John Wiley & Sons, Inc., New York; H. Y. Fang, "Foundation Engineering Handbook," 2nd ed., Van Nostrand Reinhold, New York; Briaud, J. L. (1989). "The Pressuremeter Test For Highway Applications," Federal Highway Administration Publication No. FHWA-IP-89-008, Washington D.C.)

7.5.6 California Bearing Ratio (CBR)

This ratio is often used as a measure of the quality or strength of a soil that will underlie a pavement, for determining the thickness of the pavement, its base, and other layers.

$$CBR = \frac{F}{F_0} \tag{7.11}$$

where F = force per unit area required to penetrate a soil mass with a 3-in^2 circular piston (about 2 in in diameter) at the rate of 0.05 in/min

F_0 = force per unit area required for corresponding penetration of a standard material

Typically, the ratio is determined at 0.10 in penetration, although other penetrations sometimes are used. An excellent base course has a CBR of 100%. A compacted soil may have a CBR of 50%, whereas a weaker soil may have a CBR of 10.

Tests to determine CBR may be performed in the laboratory or the field. ASTM standard tests are available for each case: "Standard Test Method for CBR (California Bearing Ratio) for Laboratory Compacted Soils," D1883, and "Standard Test Method for CBR (California Bearing Ratio) of Soils in Place," D4429 (www.astm.org).

One criticism of the method is that it does not simulate the shearing forces that develop in supporting materials underlying a flexible pavement.

7.5.7 Soil Permeability

The coefficient of permeability k is a measure of the rate of flow of water through saturated soil under a given hydraulic gradient i, cm/cm, and is defined in accordance with Darcy's law as:

$$V = kiA \tag{7.12}$$

where V = rate of flow, cm^3/s.

A = cross-sectional area of soil conveying flow, cm^2

k is dependent on the grain-size distribution, void ratio, and soil fabric and typically may vary from as much as 10 cm/s for gravel to less than 10^{-7} cm/s for clays. For typical soil deposits, k for horizontal flow is greater than k for vertical flow, often by an order of magnitude.

Soil-permeability measurements can be conducted in tests under falling or constant head, either in the laboratory or the field. Large-scale

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pumping (drawdown) tests also may be conducted in the field to provide a significantly larger scale measurement of formation permeability. Correlations of k with soil gradation and relative density or void ratio have been developed for a variety of coarse-grained materials. General correlations of k with soil index and physical properties are less reliable for fine-grained soils because other factors than porosity may control.

(T. W. Lambe and R. V. Whitman, "Soil Mechanics," John Wiley & Sons, Inc., New York, www.wiley.com.)

7.6 Site Investigations

The objective of most geotechnical site investigations is to obtain information on the site and subsurface conditions that is required for design and construction of engineered facilities and for evaluation and mitigation of geologic hazards, such as landslides, subsidence, and liquefaction. The site investigation is part of a fully integrated process that includes:

1. Synthesis of available data
2. Field and laboratory investigations
3. Characterization of site stratigraphy and soil properties
4. Engineering analyses
5. Formulation of design and construction criteria or engineering evaluations

7.6.1 Planning and Scope

In the planning stage of a site investigation, all pertinent topographical, geologic, and geotechnical information available should be reviewed and assessed. In urban areas, the development history of the site should be studied and evaluated. It is particularly important to provide or require that a qualified engineer direct and witness all field operations.

The scope of the geotechnical site investigation varies with the type of project but typically includes topographic and location surveys, exploratory "drilling and sampling, in situ testing and groundwater monitoring. Frequently the investigation is supplemented by test pits, geophysical tests, air photos and remote sensing".

7.6.2 Exploratory Borings

Typical boring methods employed for geotechnical exploration consist of rotary drilling, auger drilling, percussion drilling, or any combination of these. Deep soil borings (greater than about 100 ft) are usually conducted by rotary-drilling techniques recirculating a weighted drilling fluid to maintain borehole stability. Auger drilling, with hollow-stem augers to facilitate sampling, is a widely used and economical method for conducting short- to intermediate-length borings. Most of the drill rigs are truck-mounted and have a rockcoring capability. A wide variety of drilling machines are available to provide access to the most difficult projects.

With percussion drilling, a casing is usually driven to advance the boring. Water circulation or driven, clean-out spoons are often used to remove the soil (cuttings) in the casing. This method is employed for difficult-access locations where relatively light and portable drilling equipment is required. A rotary drill designed for rock coring is often included.

Soil Samples ▪ These are usually obtained by driving a split-barrel sampler or by hydraulically or mechanically advancing a thin-wall (Shelby) tube sampler. Driven samplers, usually 2 in outside diameter (OD), are advanced 18 in by a 140-lb hammer dropped 30 in (ASTM D1586). The number of blows required to drive the last 12 in of penetration constitutes the **standard penetration resistance** (SPT) value. The **Shelby tube sampler**, used for undisturbed sampling, is typically a 12- to 16-gage seamless steel tube and is nominally 3 in OD (ASTM D1587). In soils that are soft or otherwise difficult to sample, a stationary piston sampler is used to advance a Shelby tube either hydraulically (pump pressure) or by the down-crowd system of the drill.

Rotary core drilling is typically used to obtain core samples of rock and hard, cohesive soils that cannot be penetrated by conventional sampling techniques. Typically, rock cores are obtained with diamond bits that yield core-sample diameters from $\frac{7}{8}$ (AX) to $2\frac{1}{8}$ (NX). For hard clays and soft rocks, a 3- to 6-in OD undisturbed sample can also be obtained by rotary drilling with a **Dennison** or **Pitcher sampler**.

Test Boring Records (Logs) ▪ These typically identify the depths and material classification

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of the various strata encountered, the sample location and penetration resistance, rock-core interval and recovery, groundwater levels encountered during and after drilling. Special subsurface conditions should be noted on the log, for example, changes in drilling resistance, hole caving, voids, and obstructions. General information required includes the location of the boring, surface elevation, drilling procedures, sampler and core barrel types, and other information relevant to interpretation of the boring log.

Groundwater Monitoring ▪ Monitoring groundwater levels is an integral part of boring and sampling operations. Groundwater measurements during and at least 12 h after drilling are usually required. Standpipes are often installed in test borings to provide longer-term observations; they are typically small-diameter pipes perforated in the bottom few feet of casing.

If irregular piezometric profiles are suspected, piezometers may be set and sealed so as to measure hydrostatic heads within selected strata. Piezometers may consist of watertight ½ to ¾-in OD standpipes or plastic tubing attached to porous ceramic or plastic tips. Piezometers with electronic or pneumatic pressure sensors have the advant-age of quick response and automated data acquisition. However, it is not possible to conduct in situ permeability tests with these closed-system piezometers.

7.6.3 In Situ Testing Soils

In situ tests can be used under a variety of circumstances to enhance profile definition, to provide data on soil properties, and to obtain parameters for empirical analysis and design applications.

Quasi-static and dynamic cone penetration tests (**CPTs**) quite effectively enhance profile definition by providing a continuous record of penetration resistance. Quasi-static cone penetration resistance is also correlated with the relative density, *OCR*, friction angle, and compressibility of coarse-grained soils and the undrained shear strength of cohesive soils. Empirical foundation design parameters are also provided by the CPT.

The standard CPT in the United States consists of advancing a 10-cm^2, 60° cone at a rate between 1.5 and 2.5 cm/s and recording the resistance to cone penetration (ASTM D3441). A friction sleeve may also be incorporated to measure frictional resistance during penetration. The cone may be incrementally (mechanical penetrometer) or continuously (electronic penetrometer) advanced.

Dynamic cones are available in a variety of sizes, but in the United States, they typically have a 2-in upset diameter with a 60° apex. They are driven by blows of a 140-lb hammer dropped 30 in. Automatically driven cone penetrometers are widely used in western Europe and are portable and easy to operate.

Pressuremeter tests (**PMTs**) provide an in situ interpretation of soil compressibility and undrained shear strength. Pressuremeters have also been used to provide parameters for foundation design.

The PMT is conducted by inserting a probe containing an expandable membrane into a drill hole and then applying a hydraulic pressure to radially expand the membrane against the soil, to measure its volume change under pressure. The resulting curve for volume change vs. pressure is the basis for interpretation of soil properties.

Vane shear tests provide in situ measurements of the undrained shear strength of soft to firm clays, usually by rotating a four-bladed vane and measuring the torsional resistance T. Undrained shear strength is then calculated by dividing T by the cylindrical side and end areas inscribing the vane. Account must be taken of torque rod friction (if unsleeved), which can be determined by calibration tests (ASTM D2573). Vane tests are typically run in conjunction with borings, but in soft clays the vane may be advanced without a predrilled hole.

Other in situ tests occasionally used to provide soil-property data include plate load tests (PLTs), borehole shear (BHS) tests, and dilatometer tests. The PLT technique may be useful for providing data on the compressibility of soils and rocks. The BHS may be useful for characterizing effective-shear-strength parameters for relatively free-draining soils as well as total-stress (undrained) shear-strength parameters for fine-grained soils. Dilatometer tests provide a technique for investigating the horizontal effective stress σ'_{ho} and soil compressibility. Some tests use small-diameter probes to measure pore-pressure response, acoustical emissions, bulk density, and moisture content during penetration.

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Prototype load testing as part of the geotechnical investigation represents a variation of in situ testing. It may include pile load tests, earth load tests to investigate settlement and stability, and tests on small-scale or full-size shallow foundation elements. Feasibility of construction can also be evaluated at this time by test excavations, indicator pile driving, drilled shaft excavation, rock rippability trials, dewatering tests, and so on.

(H. Y. Fang, "Foundation Engineering Handbook," 2nd ed., Van Nostrand Reinhold, New York.)

7.6.4 Geophysical Investigations

Geophysical measurements are often valuable in evaluation of continuity of soil and rock strata between boring locations. Under some circumstances, such data can reduce the number of borings required. Certain of these measurements can also provide data for interpreting soil and rock properties. The techniques often used in engineering applications are as follows:

Seismic-wave-propagation techniques include seismic refraction, seismic rejection, and direct wave-transmission measurements. Refraction techniques measure the travel time of seismic waves generated from a single-pulse energy source to detectors (geophones) located at various distances from the source. The principle of seismic refraction surveying is based on refraction of the seismic waves at boundaries of layers with different acoustical impedances. This technique is illustrated in Fig. 7.3.

Compression P wave velocities are interpreted to define velocity profiles that may be correlated with stratigraphy and the depth to rock. The P wave velocity may also help identify type of soil. However, in saturated soils the velocity measured represents wave transmission through water-filled voids. This velocity is about 4800 ft/s regardless of the soil type. Low-cost single- and dual-channel seismographs are available for routine engineering applications.

Seismic reflection involves measuring the times required for a seismic wave induced at the surface to return to the surface after reflection from the interfaces of strata that have different acoustical impedances. Unlike refraction techniques, which usually record only the first arrivals of the seismic waves, wave trains are concurrently recorded by several detectors at different positions so as to provide a pictorial representation of formation structure. This type of survey can be conducted in both marine and terrestrial environments and usually incorporates comparatively expensive multiple-channel recording systems.

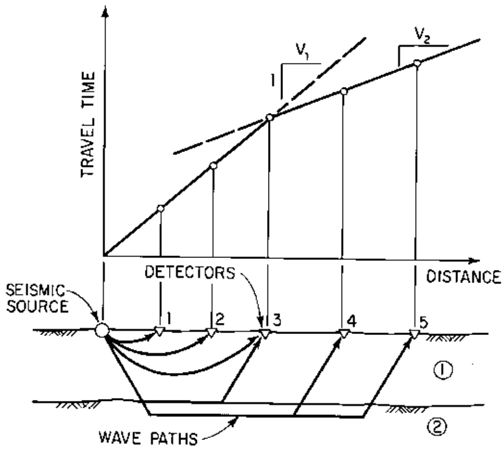

Fig. 7.3 Illustration of seismic refraction concept.

Direct seismic-wave-transmission techniques include measurements of the arrival times of P waves and shear S waves after they have traveled between a seismic source and geophones placed at similar elevations in adjacent drill holes. By measuring the precise distances between source and detectors, both S and P wave velocities can be measured for a given soil or rock interval if the hole spacing is chosen to ensure a direct wave-transmission path.

Alternatively, geophones can be placed at different depths in a drill hole to measure seismic waves propagated down from a surface source near the drill hole. The detectors and source locations can also be reversed to provide up-hole instead of down-hole wave propagation. Although this method does not provide as precise a measure of interval velocity as the cross-hole technique, it is substantially less costly.

Direct wave-transmission techniques are usually conducted so as to maximize S wave energy generation and recognition by polarization of the energy input. S wave interpretations allow calculation of the small-strain shear modulus G_{max} required for dynamic response analysis. Poisson's ratio can also be determined if both P and S wave velocities can be recorded.

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Resistivity and conductance investigation techniques relate to the proposition that stratigraphic details can be derived from differences in the electrical resistance or conductivity of individual strata. Resistivity techniques for engineering purposes usually apply the Wenner method of investigation, which involves four equally spaced steel electrodes (pins). The current is introduced through the two end pins, and the associated potential drop is measured across the two center pins. The apparent resistivity ρ is then calculated as a function of current I, potential difference V, and pin spacing a as:

$$\rho = \frac{2\pi a V}{I} \tag{7.13}$$

To investigate stratigraphic changes, tests are run at successively greater pin spacings. Interpretations are made by analyzing accumulative or discrete-interval resistivity profiles or by theoretical curve-matching procedures.

A conductivity technique for identifying subsurface anomalies and stratigraphy involves measuring the transient decay of a magnetic field with the source (dipole transmitter) in contact with the surface. The depth of apparent conductivity measurement depends on the spacing and orientation of the transmitter and receiver loops.

Both resistivity and conductivity interpretations are influenced by groundwater chemistry. This characteristic has been utilized to map the extent of some groundwater pollutant plumes by conductivity techniques.

Other geophysical methods with more limited engineering applications include gravity and magnetic field measurements. Surveys using these techniques can be airborne, shipborne, or ground-based. Microgravity surveys have been useful in detecting subsurface solution features in carbonate rocks.

Aerial surveys are appropriate where large areas are to be explored. Analyses of conventional aerial stereoscopic photographs; thermal and false-color, infrared imagery; multispectral satellite imagery; or side-looking aerial radar can disclose the surface topography and drainage, linear features that reflect geologic structure, type of surface soil and often the type of underlying rock. These techniques are particularly useful in locating filled-in sinkholes in karst regions, which are often characterized by closely spaced, slight surface depressions.

(M. B. Dobrin, "Introduction to Geophysical Prospecting," McGraw-Hill Book Company, New York, books.mcgraw-hill.com.)

7.7 Hazardous Site and Foundation Conditions

There are a variety of natural hazards of potential concern in site development and foundation design. Frequently, these hazards are overlooked or not given proper attention, particularly in areas where associated failures have been infrequent.

7.7.1 Solution-Prone Formations

Significant areas in the eastern and midwestern United States are underlain by formations (carbonate and evaporate rocks) susceptible to dissolution. Subsurface voids created by dissolution range from open jointing to huge caverns. These features have caused catastrophic failures and detrimental settlements of structures as a result of ground loss or surface subsidence.

Special investigations designed to identify rock-solution hazards include geologic reconnaissance, air photo interpretation, and geophysical (resistivity, microgravity, and so on) surveys. To mitigate these hazards, careful attention should be given to:

1. Site drainage to minimize infiltration of surface waters near structures
2. Limitation of excavations to maximize the thickness of soil overburden
3. Continuous foundation systems designed to accommodate a partial loss of support beneath the foundation system
4. Deep foundations socketed into rock and designed solely for socket bond resistance

It is prudent to conduct special **proof testing** of the bearing materials during construction in solution-prone formations. Proof testing often consists of soundings continuously recording the penetration resistance through the overburden and the rate of percussion drilling in the rock. Suspect zones are thus identified and can be improved by excavation and replacement or by in situ grouting.

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7.7.2 Expansive Soils

Soils with a medium to high potential for causing structural damage on expansion or shrinkage are found primarily throughout the Great Plains and Gulf Coastal Plain Physiographic Provinces. Heave or settlement of *active* soils occurs because of a change in soil moisture in response to climatic changes, construction conditions, changes in surface cover, and other conditions that influence the groundwater and evapotransportation regimes. Differential foundation movements are brought about by differential moisture changes in the bearing soils. Figure 7.4 presents a method for classifying the volume-change potential of clay as a function of activity.

Investigations in areas containing potentially expansive soils typically include laboratory swell tests. Infrequently, soil suction measurements are made to provide quantitative evaluations of volume-change potential. Special attention during the field investigation should be given to evaluation of the groundwater regime and to delineation of the depth of active moisture changes.

Common design procedures for preventing structural damage include mitigation of moisture changes, removal or modification of expansive material and deep foundation support. Horizontal and vertical moisture barriers have been utilized to minimize moisture losses due to evaporation or infiltration and to cut off subsurface groundwater flow into the area of construction. Excavation of potentially active materials and replacement with inert material or with excavated soil modified by the addition of lime have proved feasible where excavation quantities are not excessive.

Deep foundations (typically drilled shafts) have been used to bypass the active zone and to resist or minimize uplift forces that may develop on the shaft. Associated grade beams are usually constructed to prevent development of uplift forces.

("Engineering and Design of Foundations on Expansive Soils," U.S. Department of the Army, 1981. L. D. Johnson, "Predicting Potential Heave and Heave with Time and Swelling Foundation Soils," Technical Report S-78-7, U.S. Army Engineers Waterways Experiment Station, Vicksburg, Miss., 1978.)

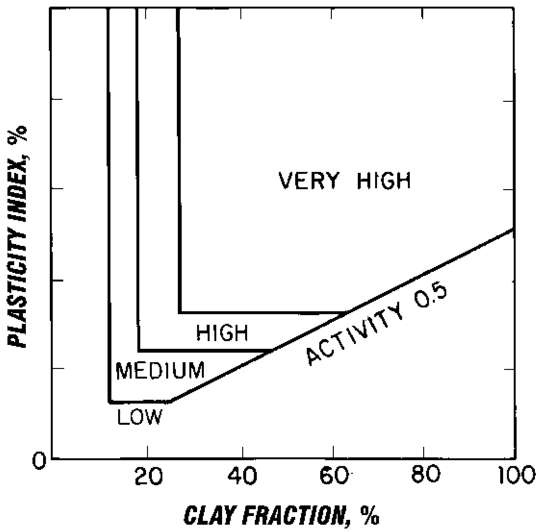

Fig. 7.4 Chart for rating volume-change potential of expansive soils.

7.7.3 Landslide Hazards

Landslides are usually associated with areas of significant topographic relief that are characterized by relatively weak sedimentary rocks (shales, siltstones, and so forth) or by relatively impervious soil deposits containing interbedded water-bearing strata. Under these circumstances, slides that have occurred in the geologic past, whether or not currently active, represent a significant risk for hillside site development. In general hillside development in potential landslide areas is a most hazardous undertaking. If there are alternatives, one of those should be adopted.

Detailed geologic studies are required to evaluate slide potential and should emphasize detection of old slide areas. Procedures that tend to stabilize an active slide or to provide for the continued stability of an old slide zone include:

1. Excavation at the head of the sliding mass to reduce the driving force
2. Subsurface drainage to depress piezometric levels along potential sliding surface
3. Buttressing at the toe of the potential sliding mass to provide a force-resisting slide movement

Within the realm of economic feasibility, the reliability of these or any other procedures to

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stabilize active or old slides involving a significant sliding mass are generally of a relatively low order.

On hillsides where prior slides have not been identified, care should be taken to reduce the sliding potential of superimposed fills by removing weak or potentially unstable surficial materials, benching and keying the fill into competent materials, and (most importantly) installing effective subsurface drainage systems. Excavations that result in steepening of existing slopes are potentially detrimental and should not be employed. Direction and collection of surface runoff so as to prevent slope erosion and infiltration are recommended. ("Landslides Investigation and Mitigation," Transportation Research Board Special Report 247 National Academy Press 1996)

7.7.4 Liquefaction of Soils

Relatively loose saturated cohesionless soils may become unstable under cyclic shear loading such as that imposed by earthquake motions. A simplified method of analysis of the *liquefaction potential* of cohesionless soils has been proposed for predicting the ratio of the horizontal shear stress τ_{av} to the effective overburden pressure σ'_{vo} imposed by an earthquake. (τ_{av} represents a uniform cyclic-stress representation of the irregular time history of shear stress induced by the design earthquake.) This field stress ratio is a function of the maximum horizontal ground-surface acceleration a_{max}, the acceleration of gravity g, a stress-reduction factor r_d, and total vertical stress σ_{vo} and approximated as

$$\frac{\tau_{av}}{\sigma'_{vo}} = 0.65\frac{a_{max}}{g}\frac{\sigma_{vo}}{\sigma'_{vo}}r_d \qquad (7.14)$$

r_d varies from 1.0 at the ground surface to 0.9 for a depth of 30 ft. (H. B. Seed and I. M. Idriss, "A Simplified Procedure for Evaluating Soil Liquefaction Potential," Report EERC 70-9, Earthquake Engineering Research Center, University of California, Berkeley, 1970.)

Stress ratios that produce liquefaction may be characterized from correlations with field observations (Fig. 7.5). The relevant soil properties are represented by their corrected penetration resistance

$$N_1 = (1 - 1.25\log\sigma'_{vo})N \qquad (7.15)$$

where σ'_{vo} is in units of tons/ft^2. The stress ratio causing liquefaction should be increased about 25% for earthquakes with Richter magnitude 6 or lower. (H. B. Seed, "Evaluation of Soil Liquefaction Effects on Level Ground during Earthquakes," *Symposium on Liquefaction Problems and Geotechnical Engineering*,

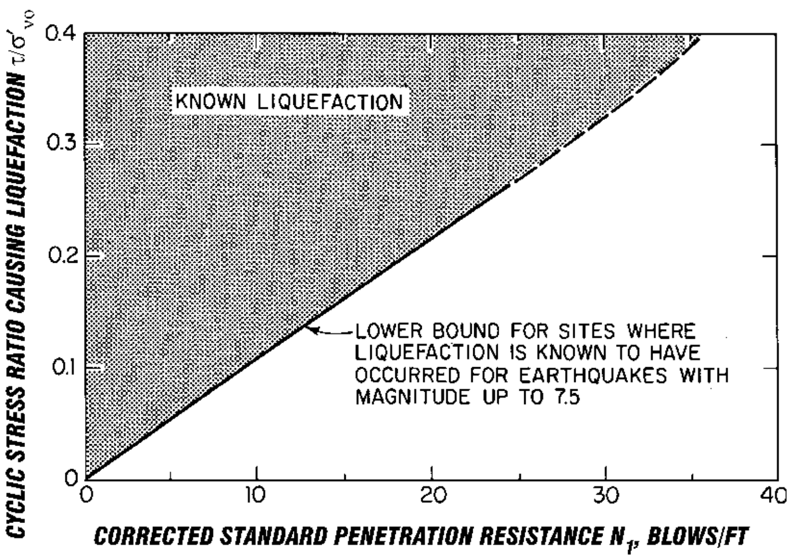

Fig. 7.5 Chart correlates cyclic-stress ratios that produce soil liquefaction with standard penetration resistance. (*After H. B. Seed.*)

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ASCE National Convention, Philadelphia, Pa., 1976. ("Design Guidance: Geotechnical Earthquake Engineering For Highways," Federal Highway Administration, Publication No. FHWA-SA-97-076, May 1997)

Significantly, more elaborate, dynamic finite-element procedures have been proposed to evaluate soil liquefaction and degradation of undrained shear strength as well as generation and dissipation of pore-water pressure in soils as a result of cyclic loading. Since stress increases accompany dissipation of pore-water pressures, settlements due to cyclic loading can also be predicted. Such residual settlements can be important even though liquefaction has not been induced.

(P. B. Schnabel, J. Lysmer, and H. B. Seed, "A Computer Program for Earthquake Response Analysis of Horizontally Layered Sites," Report EERC 72-12, Earthquake Engineering Research Center, University of California, Berkeley, 1972; H. B. Seed, P. P. Martin, and J. Lysmer, "Pore-Water Pressure Changes During Soil Liquefaction," *ASCE Journal of Geotechnical Engineering Division*, vol. 102, no. GT4, 1975; K. L. Lee and A. Albaisa, "Earthquake-Induced Settlements in Saturated Sands," *ASCE Journal of Geotechnical Engineering Division*, vol. 100, no. GT4, 1974, www.asce.org.)

7.8 Types of Footings

Spread (individual) footings (Fig. 7.6) are the most economical shallow foundation types but are more susceptible to differential settlement. They usually support single concentrated loads, such as those imposed by columns.

Shallow Foundations

Shallow foundation systems can be classified as spread footings, wall and continuous (strip) footings, and mat (raft) foundations. Variations are combined footings, cantilevered (strapped) footings, two-way strip (grid) footings, and discontinuous (punched) mat foundations.

Combined footings (Fig. 7.7) are used where the bearing areas of closely spaced columns overlap. Cantilever footings (Fig. 7.8) are designed to accommodate eccentric loads.

Continuous wall and strip footings (Fig. 7.9) can be designed to redistribute bearing-stress concentrations and associated differential settlements in the event of variable bearing conditions or localized ground loss beneath footings.

Mat foundations have the greatest facility for load distribution and for redistribution of subgrade stress concentrations caused by localized anomalous bearing conditions. Mats may be constant section, ribbed, waffled, or arched. Buoyancy mats are used on compressible soil sites in combination with basements or subbasements, to create a permanent unloading effect, thereby reducing the net stress change in the foundation soils.

(M. J. Tomlinson, "Foundation Design and Construction," John Wiley & Sons, Inc., New York (www.wiley.com); J. E. Bowles, "Foundation

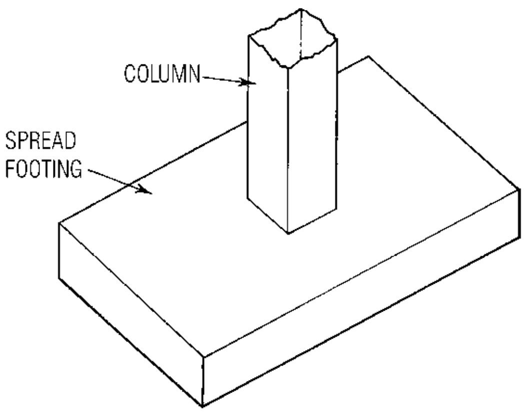

Fig. 7.6 Spread footing.

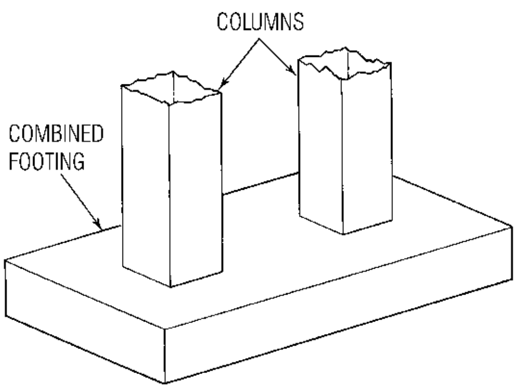

Fig. 7.7 Combined footing.

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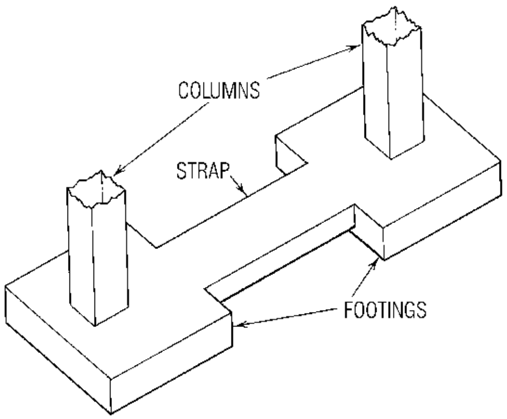

Fig. 7.8 Cantilever footing.

Analysis and Design," McGraw-Hill Book Company, New York. (books.mcgraw-hill.com) "Spread Footings for Highway Bridges," Federal Highway Administration, Publication No. FHWA-RD-86-185 October, 1987)

7.9 Approach to Foundation Analysis

Shallow-foundation analysis and formulation of geotechnical design provisions are generally approached in the following steps:

1. Establish project objectives and design or evaluation conditions.
2. Characterize site stratigraphy and soil rock properties.
3. Evaluate load-bearing fill support or subsoil-improvement techniques, if applicable.
4. Identify bearing levels; select and proportion candidate foundation systems.
5. Conduct performance, constructibility, and economic feasibility analyses.
6. Repeat steps 3 through 5 as required to satisfy the design objectives and conditions.

The scope and detail of the analyses vary according to the project objectives.

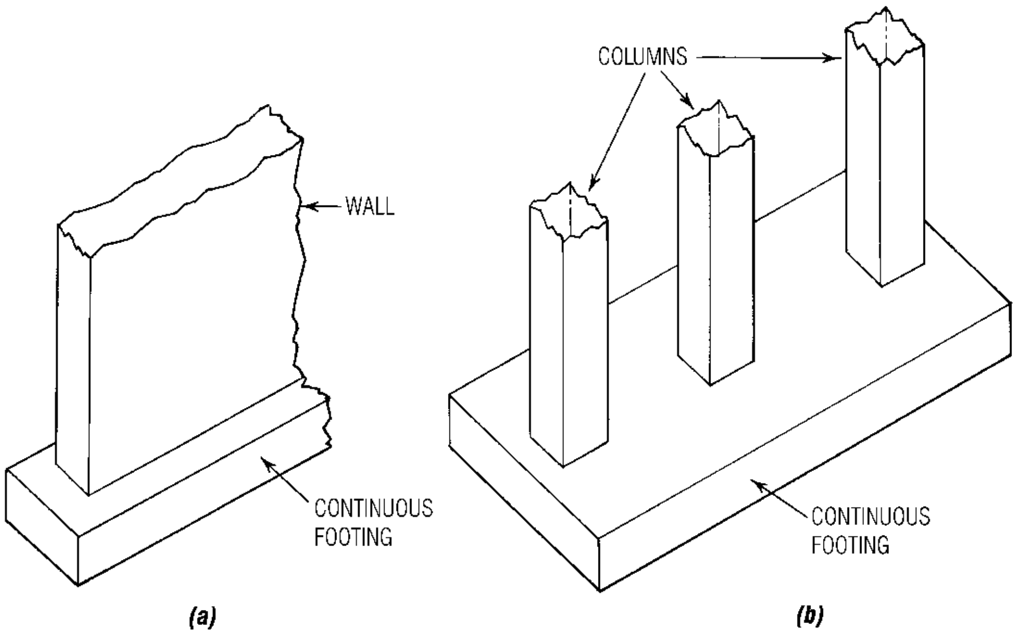

Fig. 7.9 Continuous footings for (*a*) a wall, (*b*) several columns.

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Project objectives to be quantified are essentially the intent of the project assignment and the specific scope of associated work. The conditions that control geotechnical evaluation or design work include criteria for loads and grades, facility operating requirements and tolerances, construction schedules, and economic and environmental constraints. Failure to provide a clear definition of relevant objectives and design conditions can result in significant delays, extra costs, and, under some circumstances, unsafe designs.

During development of design conditions for structural foundations, tolerances for total and differential settlements are commonly established as a function of the ability of a structure to tolerate movement. Suggested structure tolerances in terms of angular distortion are in Table 7.5. Angular distortion represents the differential vertical movement between two points divided by the horizontal distance between the points.

Development of *design profiles* for foundation analysis ideally involves a synthesis of geologic and geotechnical data relevant to site stratigraphy and soil and rock properties. This usually requires site investigations (see Arts. 7.6.1 to 7.6.4) and in situ or laboratory testing, or both, of representative soil and rock samples (see Arts. 7.3 to 7.5.6).

Table 7.5 Limiting Angular Distortions*

Structural Response	Angular Distortion
Cracking of panel and brick walls	1/100
Structural damage to columns and beams	1/150
Impaired overhead crane operation	1/300
First cracking of panel walls	1/300
Limit for reinforced concrete frame	1/400
Limit for wall cracking	1/500
Limit for diagonally braced frames	1/600
Limit for settlement-sensitive machines	1/750

* Limits represent the maximum distortions that can be safely accommodated.
Source: After L. Bjerrum, European Conference on Soil Mechanics and Foundation Engineering, Wiesbaden, Germany, vol. 2, 1963.

To establish and proportion candidate foundation systems, consideration must first be given to identification of feasible bearing levels. The *depth of the foundation* must also be sufficient to protect exposed elements against frost heave and to provide sufficient confinement to produce a factor of safety not less than 2.5 (preferably 3.0) against shear failure of the bearing soils. Frost penetration has been correlated with a **freezing index**, which equals the number of days with temperature below 32° F multiplied by $T - 32$, where $T =$ average daily temperature. Such correlations can be applied in the absence of local codes or experience. Generally, footing depths below final grade should be a minimum of 2.0 to 2.5 ft.

For marginal bearing conditions, consideration should be given to improvement of the quality of potential bearing strata. Soil-improvement techniques include excavation and replacement or overlaying of unsuitable subsoils by *load-bearing fills*, *preloading* of compressible subsoils, *soil densification*, *soil reinforcement*, and *grouting techniques*. Densification methods include high energy surface impact (dynamic compaction), on grade vibratory compaction, and subsurface vibratory compaction by vibro-compaction techniques. Soil reinforcement methods include: stone columns, soil mixing, mechanically stabilized earth and soil nailing.

“Ground Improvement Technical Summaries,” Federal Highway Administration, Publication No. FHWA-SA-98-086, December 1999.

The choice of an appropriate soil improvement technique is highly dependent on performance requirements (settlement), site and subsurface conditions, time and space constraints and cost.

Assessment of the suitability of candidate foundation systems requires evaluation of the safety factor against both catastrophic failure and excessive deformation under sustained and transient design loads. Catastrophic-failure assessment must consider overstress and creep of the bearing soils as well as lateral displacement of the foundation. Evaluation of the probable settlement behavior requires analysis of the stresses imposed within the soil and, with the use of appropriate soil parameters, prediction of foundation settlements. Typically, *settlement analyses* provide estimates of total and differential settlement at strategic locations within the foundation area and may include time-rate predictions of settlement. Usually, the suitability of shallow foundation systems is

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governed by the systems' load-settlement response rather than bearing capacity.

7.10 Foundation-Stability Analysis

The maximum load that can be sustained by shallow foundation elements at incipient failure (*bearing capacity*) is a function of the cohesion and friction angle of bearing soils as well as the width B and shape of the foundation. The **net bearing capacity** per unit area q_u of a long footing is conventionally expressed as:

$$q_u = \alpha_f c_u N_c + \sigma'_{vo} N_q + \beta_f \gamma B N_\gamma \tag{7.16}$$

where α_f = 1.0 for strip footings and 1.3 for circular and square footings

c_u = undrained shear strength of soil

σ'_{vo} = effective vertical shear stress in soil at level of bottom of footing

β_f = 0.5 for strip footings, 0.4 for square footings, and 0.6 for circular footings

γ = unit weight of soil

B = width of footing for square and rectangular footings and radius of footing for circular footings

N_c, N_q, N_γ = bearing-capacity factors, functions of angle of internal friction ϕ (Fig. 7.10)

For undrained (rapid) loading of cohesive soils, $\phi = 0$ and Eq. (7.16) reduces to

$$q_u = N'_c c_u \tag{7.17}$$

where $N'_c = \alpha_f N_c$. For drained (slow) loading of cohesive soils, ϕ and c_u are defined in terms of effective friction angle ϕ' and effective stress c'_u.

Modifications of Eq. (7.16) are also available to predict the bearing capacity of layered soil and for eccentric loading.

Rarely, however, does q_u control foundation design when the safety factor is within the range of 2.5 to 3. (Should creep or local yield be induced, excessive settlements may occur. This consideration is particularly important when selecting a safety factor for foundations on soft to firm clays with medium to high plasticity.)

Equation (7.16) is based on an infinitely long strip footing and should be corrected for other shapes. Correction factors by which the bearing-capacity factors should be multiplied are given in Table 7.6, in which L = footing length.

The derivation of Eq. (7.16) presumes the soils to be homogeneous throughout the stressed zone, which is seldom the case. Consequently, adjustments may be required for departures from homo-

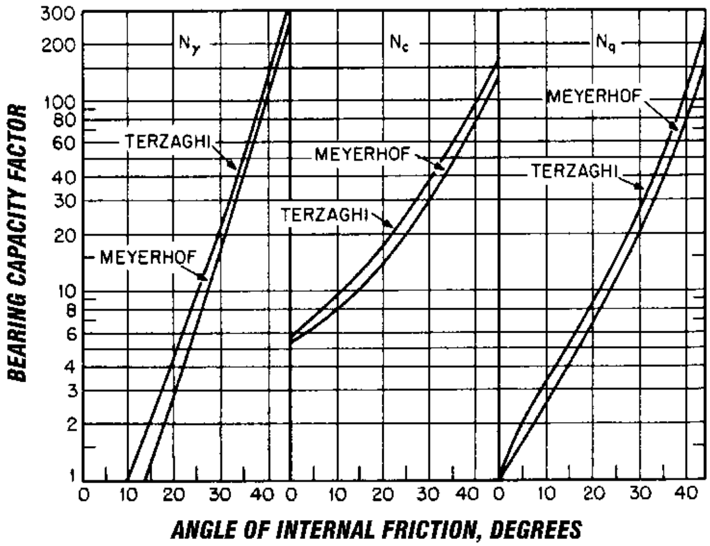

Fig. 7.10 Bearing capacity factors for use in Eq. (7.16) as determined by Terzaghi and Meyerhof.

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Table 7.6 Shape Corrections for Bearing-Capacity Factors of Shallow Foundations*

	Correction Factor		
Shape of Foundation	N_c	N_q	N_γ
Rectangle†	$1 + (B/L)\,(N_q/N_c)$	$1 + (B/L)\tan\phi$	$1 - 0.4(B/L)$
Circle and square	$1 + (N_q/N_c)$	$1 + \tan\phi$	0.60

* After E. E. De Beer, as modified by A. S. Vesic. See H. Y. Fang, "Foundation Engineering Handbook," Van Nostrand Reinhold, 2d ed., New York.

† No correction factor is needed for long strip foundations.

geneity. In sands, if there is a moderate variation in strength, it is safe to use Eq. (7.16), but with bearing-capacity factors representing a weighted average strength.

For strongly varied soil profiles or interlayered sands and clays, the bearing capacity of each layer should be determined. This should be done by assuming the foundation bears on each layer successively but at the contact pressure for the depth below the bottom of the foundation of the top of the layer.

Eccentric loading can have a significant impact on selection of the bearing value for foundation design. The conventional approach is to proportion the foundation to maintain the resultant force within its middle third. The footing is assumed to be rigid and the bearing pressure is assumed to vary linearly as shown by Fig. 7.11*b*. If the resultant lies outside the middle third of the footing, it is assumed that there is bearing over only a portion of the footing, as shown in Fig. 7.11*d*. For the conventional case (Fig. 7.11*a*), the maximum and minimum bearing pressures are:

$$q_m = \frac{P}{BL}\left(1 \pm \frac{6e}{B}\right) \tag{7.18}$$

where B = width of rectangular footing

L = length of rectangular footing

e = eccentricity of loading

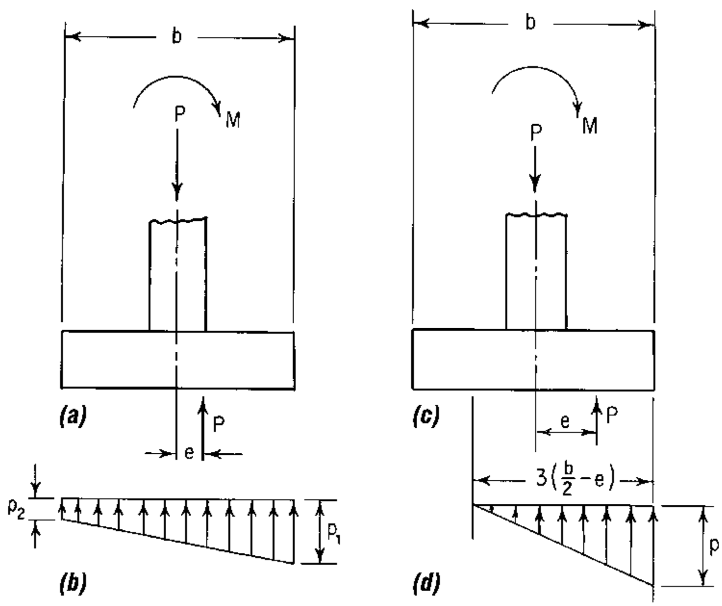

Fig. 7.11 Footings subjected to overturning.

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For the other case (Fig. 7.11*c*), the soil pressure ranges from 0 to a maximum of:

$$q_m = \frac{2P}{3L(B/2 - e)} \tag{7.19}$$

For square or rectangular footings subject to overturning about two principal axes and for unsymmetrical footings, the loading eccentricities e_1 and e_2 are determined about the two principal axes. For the case where the full bearing area of the footings is engaged, q_m is given in terms of the distances from the principal axes c_1 and c_2, the radius of gyration of the footing area about the principal axes r_1 and r_2, and the area of the footing A as:

$$q_m = \frac{P}{A}\left(1 + \frac{e_1 c_1}{r_1^2} + \frac{e_2 c_2}{r_2^2}\right) \tag{7.20}$$

For the case where only a portion of the footing is bearing, the maximum pressure may be approximated by trial and error.

For all cases of *sustained eccentric loading*, the maximum (edge) pressures should not exceed the shear strength of the soil and also the factor of safety should be at least 1.5 (preferably 2.0) against overturning.

The foregoing analyses, except for completely rigid foundation elements, are a very conservative approximation. Because most mat foundations and large footings are not completely rigid, their deformation during eccentric loading acts to produce a more uniform distribution of bearing pressures than would occur under a rigid footing and to reduce maximum contact stresses.

In the event of *transient eccentric loading*, experience has shown that footings can sustain maximum edge pressures significantly greater than the shear strength of the soil. Consequently, some building codes conservatively allow increases in sustained-load bearing values of 30% for transient loads. Reduced safety factors have also been used in conjunction with transient loading. For cases where significant cost savings can be realized, finite-element analyses that model soil-structure interaction can provide a more realistic evaluation of an eccentrically loaded foundation.

Allowable Bearing Pressures ▪ Approximate allowable soil bearing pressures, without tests, for various soil and rocks are given in Table 7.7 for normal conditions. These basic bearing pressures should be used for preliminary

Table 7.7 Allowable Bearing Pressures for Soils

Soil Material	Pressure, tons/ft^2	Notes
Unweathered sound rock	60	No adverse seam structure
Medium rock	40	
Intermediate rock	20	
Weathered, seamy, or porous rock	2 to 8	
Hardpan	12	Well cemented
Hardpan	8	Poorly cemented
Gravel soils	10	Compact, well graded
Gravel soils	8	Compact with more than 10% gravel
Gravel soils	6	Loose, poorly graded
Gravel soils	4	Loose, mostly sand
Sand soils	3 to 6	Dense
Fine sand	2 to 4	Dense
Clay soils	5	Hard
Clay soils	2	Medium stiff
Silt soils	3	Dense
Silt soils	1½	Medium dense
Compacted fills		Compacted to 90% to 95% of maximum density (ASTM D1557)
Fills and soft soils	2 to 4	By field or laboratory test only

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design only. Final design values should be based on the results of a thorough subsurface investigation and the results of engineering analysis of potential failure and deformation limit states.

Resistance to Horizontal Forces ▪ The horizontal resistance of shallow foundations is mobilized by a combination of the passive soil resistance on the vertical projection of the embedded foundation and the friction between the foundation base and the bearing soil and rock. The soil pressure mobilized at full passive resistance, however, requires lateral movements greater than can be sustained by some foundations. Consequently, a soil resistance between the at-rest and passive-pressure cases should be determined on the basis of the allowable lateral deformations of the foundation.

The frictional resistance f to horizontal translation is conventionally estimated as a function of the sustained, real, load-bearing stresses q_d from

$$f = q_d \tan \delta \tag{7.21}$$

where δ is the friction angle between the foundation and bearing soils. δ may be taken as equivalent to the internal-friction angle ϕ' of the subgrade soils. In the case of cohesive soils, $f = c_u$. Again, some relative movement must be realized to develop f, but this movement is less than that required for passive-pressure development.

If a factor of safety against translation of at least 1.5 is not realized with friction and passive soil/rock pressure, footings should be keyed to increase sliding resistance or tied to engage additional resistance. Building basement and shear walls are also commonly used to sustain horizontal loading.

7.11 Stress Distribution Under Footings

Stress changes imposed in bearing soils by earth and foundation loads or by excavations are conventionally predicted from elastic half-space theory as a function of the foundation shape and the position of the desired stress profile. Elastic solutions available may take into account foundation rigidity, depth of the compressible zone, superposition of stress from adjacent loads, layered profiles, and moduli that increase linearly with depth.

For most applications, stresses may be computed by the pressure-bulb concept with the methods of either Boussinesq or Westergaard. For thick deposits, use the Boussinesq distribution shown in Fig. 7.12*a*; for thinly stratified soils, use the Westergaard approach shown in Fig. 7.12*b*. These charts indicate the stresses q beneath a single foundation unit that applies a pressure at its base of q_o.

Most facilities, however, involve not only multiple foundation units of different sizes, but also floor slabs, perhaps fills, and other elements that contribute to the induced stresses. The stresses used for settlement calculation should include the overlapping and contributory stresses that may arise from these multiple loads.

7.12 Settlement Analyses of Cohesive Soils

Settlement of foundations supported on cohesive soils is usually represented as the sum of the primary one-dimensional consolidation ρ_c, immediate ρ_i, and secondary ρ_s settlement components. Settlement due to primary consolidation is conventionally predicted for n soil layers by Eq. (7.22) and (7.23). For normally consolidated soils,

$$\rho_c = \sum_{i=1}^{n} H_i \left(C_c' \log \frac{\sigma_v}{\sigma_{vo}'} \right) \tag{7.22}$$

where H_i = thickness of ith soil layer

C_c' = strain referenced compression index for ith soil layer (Art. 7.5.4)

σ_v = sum of average σ_{vo}' and average imposed vertical-stress change $\Delta\sigma_v$ in ith soil layer

σ_{vo}' = initial effective overburden pressure at middle of ith layer (Art 7.5.3)

For overconsolidated soils with $\sigma_v > \sigma_{vm}'$,

$$\rho_c = \sum_{i=1}^{n} H_i \left(C_r' \log \frac{\sigma_{vm}'}{\sigma_{vo}'} + C_c' \log \frac{\sigma_v}{\sigma_{vm}'} \right) \tag{7.23}$$

where C_r' = strain referenced recompression index of ith soil layer (Art. 7.5.4)

σ_{vm}' = preconsolidation (maximum past consolidation) pressure at middle of ith layer (Art. 7.5.3)

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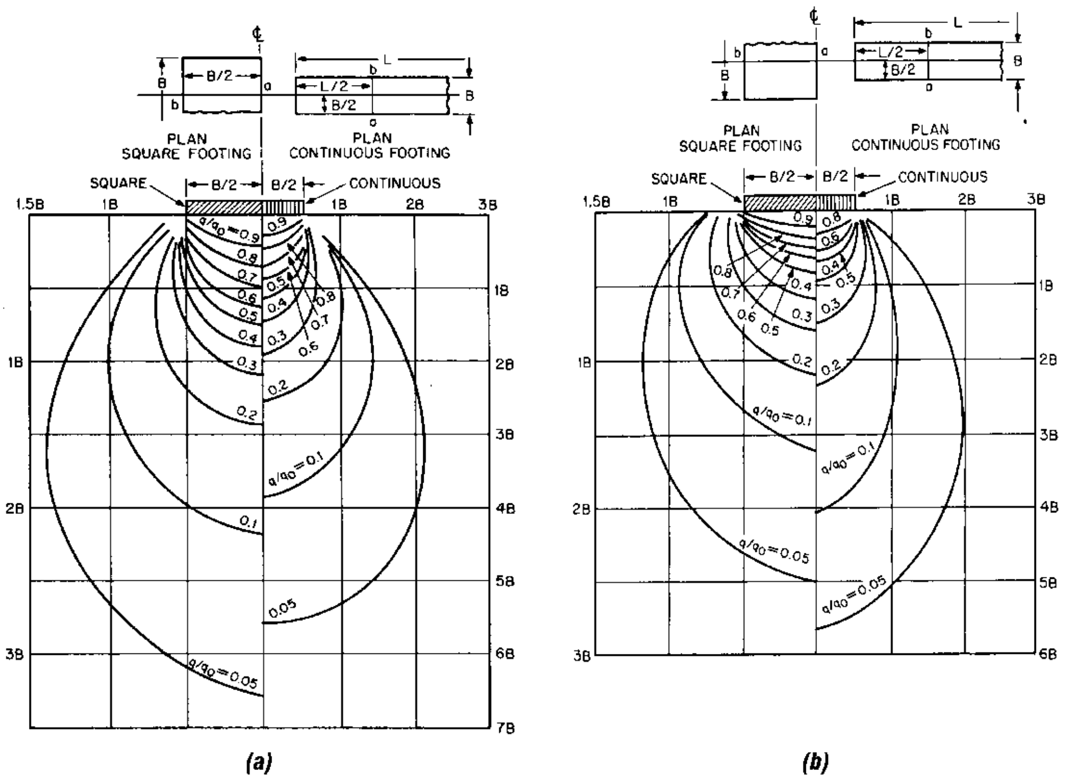

Fig. 7.12 Stress distribution under a square footing with side B and under a continuous footing with width B, as determined by equations of (*a*) Boussinesq and (*b*) Westergaard.

The maximum thickness of the compressible soil zone contributing significant settlement can be taken to be equivalent to the depth where $\Delta\sigma_v = 0.1\sigma'_{vo}$.

Equation (7.22) can also be applied to overconsolidated soils if σ_v is less than σ'_{vm} and C'_r is substituted for C'_c.

Inasmuch as Eqs. (7.22) and (7.23) represent one-dimensional compression, they may provide rather poor predictions for cases of three-dimensional loading. Consequently, corrections to ρ_c have been derived for cases of three-dimensional loading. These corrections are approximate but represent an improved approach when loading conditions deviate significantly from the one-dimensional case. (A. W. Skempton and L. Bjerrum, "A Contribution to Settlement Analysis of Foundations on Clay," *Geotechnique*, vol. 7, 1957.)

The stress-path method of settlement analysis attempts to simulate field loading conditions by conducting triaxial tests so as to track the sequential stress changes of an average point or points beneath the foundation. The strains associated with each drained and undrained load increment are summed and directly applied to the settlement calculation. Deformation moduli can also be derived from stress-path tests and used in three-dimensional deformation analysis.

Three-dimensional settlement analyses using elastic solutions have been applied to both drained and undrained conditions. Immediate (elastic) foundation settlements ρ_i, representing the undrained deformation of saturated cohesive soils, can be calculated by discrete analysis [Eq. (7.25)].

$$\rho_i = \sum_{i=1}^{n} H_i \frac{\sigma_1 - \sigma_3}{E_i} \tag{7.24}$$

where $\sigma_1 - \sigma_3$ = change in average deviator stress within each layer influenced by applied

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load. Note that Eq. (7.24) is strictly applicable only for axisymmetrical loading. Drained three-dimensional deformation can be estimated from Eq. (7.24) by substituting the secant modulus E'_s for E (see Art. 7.5.5).

The rate of one-dimensional consolidation can be evaluated with Eq. (7.26) in terms of the degree of consolidation U and the nondimensional time factor T_v. U is defined by

$$U = \frac{\rho_t}{\rho_c} = 1 - \frac{u_t}{u_i} \tag{7.25}$$

where ρ_t = settlement at time t after instantaneous loading

ρ_c = ultimate consolidation settlement

u_t = excess pore-water pressure at time t

u_i = initial pore-water pressure ($t = 0$)

To correct approximately for the assumed instantaneous load application, ρ_t at the end of the loading period can be taken as the settlement calculated for one-half of the load application time. The time t required to achieve U is evaluated as a function of the shortest drainage path within the compressible zone h, the coefficient of consolidation C_v, and the dimensionless time factor T_v from

$$t = T_v \frac{h^2}{C_v} \tag{7.26}$$

Closed-form solutions T_v vs. U are available for a variety of initial pore-pressure distributions. (H. Y. Fang, "Foundation Engineering Handbook," 2nd ed., Van Nostrand Reinhold Company, New York.)

Solutions for constant and linearly increasing u_i are shown in Fig. 7.13. Equation (7.27) presents an approximate solution that can be applied to the constant initial u_i distribution case for $T_v > 0.2$.

$$U = 1 - \frac{8}{\pi^2} e^{-\pi^2 Tv/4} \tag{7.27}$$

where $e = 2.71828$. Numerical solutions for any u_i configuration in a single compressible layer as well as solutions for contiguous clay layers may be derived with finite-difference techniques.

(R. F. Scott, "Principles of Soil Mechanics," Addison-Wesley Publishing Company, Inc., Reading, Mass.)

The coefficient of consolidation C_v should be established based on experience and from site

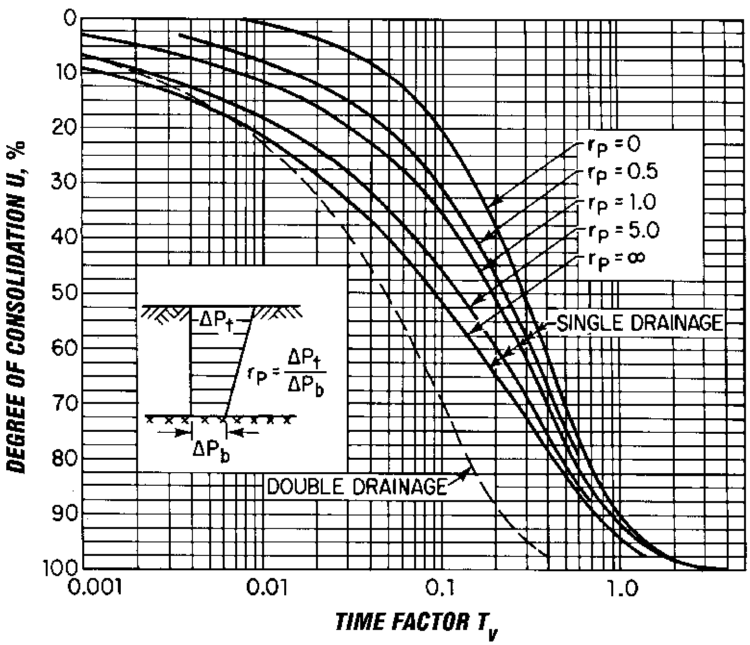

Fig. 7.13 Curves relate degree of consolidation and time factor T_v.

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specific conventional consolidation tests by fitting the curve for time vs. deformation (for an appropriate load increment) to the theoretical solution for constant u_i. For tests of samples drained at top and bottom, C_v may be interpreted from the curve for log time or square root of time vs. strain (or dial reading) as

$$C_v = \frac{T_v H^2}{4t} \tag{7.28}$$

where H = height of sample, in

t = time for 90% consolidation ($\sqrt{t}$ curve) or 50% consolidation (log t curve), days

T_v = 0.197 for 90% consolidation or 0.848 for 50% consolidation

(See T. W. Lambe and R. V. Whitman, "Soil Mechanics," John Wiley & Sons, Inc., New York, www.wiley.com), for curve-fitting procedures.) Larger values of C_v are usually obtained with the $\sqrt{t}$ method and appear to be more representative of field conditions.

Secondary compression settlement ρ_s is assumed, for simplicity, to begin on completion of primary consolidation, at time t_{100} corresponding to 100% primary consolidation. ρ_s is then calculated from Eq. (7.29) for a given period t after t_{100}.

$$\rho_s = \sum_{i=1}^{n} H_i C_\alpha \log \frac{t}{t_{100}} \tag{7.29}$$

H_i represents the thickness of compressible layers and C_α is the coefficient of secondary compression given in terms of volumetric strain (Art. 7.5.4).

The ratio C_α to compression index C_c is nearly constant for a given soil type and is generally within the range of 0.045 ± 0.015. C_α, as determined from consolidation tests (Fig. 7.2), is extremely sensitive to pressure-increment ratios of less than about 0.5 (standard is 1.0). The effect of overconsolidation, either from natural or construction preload sources, is to significantly reduce C_α. This is an important consideration in the application of preloading for soil improvement.

The rate of *consolidation due to radial drainage* is important for the design of vertical *wick drains*. As a rule, drains are installed in compressible soils to reduce the time required for consolidation and to accelerate the associated gain in soil strength. Vertical drains are typically used in conjunction with preloading as a means of improving the supporting ability and stability of the subsoils.

(S. J. Johnson, "Precompression for Improving Foundation Soils," *ASCE Journal of Soil Mechanics and Foundation Engineering Division*, vol. 96, no. SM1, 1970 (www.asce.org); R. D. Holtz and W. D. Kovacs, "An Introduction to Geotechnical Engineering," Prentice-Hall, Inc., Englewood Cliffs, N.J. (www.prenhall.com) "Prefabricated Vertical Drains" Federal Highway Administration, Publication No. FHWA-RD-86-168, 1986)

7.13 Settlement Analysis of Sands

The methods most frequently used to estimate the settlement of foundations supported by relatively free-draining cohesionless soils generally employ empirical correlations between field observations and in situ tests. The primary correlative tests are plate bearing (PLT), cone penetration resistance (CPT), and standard penetration resistance (SPT) (see Art 7.6.3). These methods, however, are developed from data bases that contain a number of variables not considered in the correlations and, therefore, should be applied with caution.

Time rate of settlement in coarse grained soils is extremely rapid. This behavior may be used to the advantage of the designer where both dead and live loads are applied to the foundation. Often vertical deformations which occur during the construction process will have a minimal effect on the completed facility.

Plate Bearing Tests ▪ The most common approach is to scale the results of PLTs to full-size footings in accordance with Eq. (7.10). A less conservative modification of this equation proposed by A. R. S. S. Barazaa is

$$\rho = \left[\frac{2.5B}{1.5 + B}\right]^2 \rho_1 \tag{7.30}$$

where B = footing width, ft

ρ = settlement of full-size bearing plate

ρ_1 = settlement of 1-ft square bearing plate

These equations are not sensitive to the relative density, gradation, and OCR of the soil or to the effects of depth and shape of the footing.

Use of *large-scale load tests* or, ideally, full-scale load tests mitigates many of the difficulties of the preceding approach but is often precluded by costs

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and schedule considerations. Unless relatively uniform soil deposits are encountered, this approach also requires a number of tests, significantly increasing the cost and time requirements. (See J. K. Mitchell and W. S. Gardner, "In-Situ Measurement of Volume-Change Characteristics," ASCE Specialty Conference on In-Situ Measurement of Soil Properties, Raleigh, N.C., 1975.)

Cone Penetrometer Methods ▪ Correlations between quasi-static penetration resistance q_c and observation of the settlement of bearing plates and small footings form the basis of foundation settlement estimates using CPT data. The Buisman-DeBeer method utilizes a one-dimensional compression formulation. A recommended modification of this approach that considers the influence of the relative density of the soil D_r and increased secant modulus E'_s is

$$\rho = \sum_{i=1}^{n} H_i \frac{1.15\sigma'_{vo}}{(1 + D_r^2)q_c} \log \frac{\sigma'_{vo} + \Delta\sigma_v}{\sigma'_{vo}} \tag{7.31}$$

where ρ = estimated footing settlement. The σ'_{vo} and $\Delta\sigma_v$ parameters represent the average effective overburden pressure and vertical stress change for each layer considered below the base of the foundation (see Art 7.12). Equation (7.31) has limitations because no consideration is given to: (1) soil stress history, (2) soil gradation, and (3) three-dimensional compression. Also, Eq. (7.31) incorporates an empirical representation of E'_s, given by Eq. (7.32), and has all the limitations thereof (see Art. 7.5.5).

$$E'_s = 2(1 + D_r^2)q_c \tag{7.32}$$

The foregoing procedures are not applicable to large footings and foundation mats. From field observations relating foundation width B, meters, to ρ/B, the upper limit for ρ/B, for $B > 13.5$ m, is given, in percent, approximately by

$$\frac{\rho}{B} = 0.194 - 0.115 \log \frac{B}{10} \tag{7.33}$$

For the same data base, the best fit of the average ρ/B measurements ranges from about 0.09% ($B = 20$ m) to 0.06% ($B = 80$ m).

Standard Penetration Resistance Methods ▪ A variety of methods have been proposed to relate foundation settlement to standard penetration resistance N. An approach proposed by I. Alpan and G. G. Meyerhof appears reasonable and has the advantage of simplicity. Settlement S, in, is computed for $B < 4$ ft from

$$S = \frac{8q}{N'} \tag{7.34a}$$

and for $B \geq 4$ ft from

$$S = \frac{12q}{N'}\left(\frac{2B}{1 + B}\right)^2 \tag{7.34b}$$

where q = bearing capacity of soil, tons/ft^2
B = footing width, ft

N' is given approximately by Eq. (7.34c) for $\sigma'_{vo} \leq$ 40 psi.

$$N' = \frac{50N}{\sigma'_{vo} + 10} \tag{7.34c}$$

and represents N (blows per foot) normalized for $\sigma'_{vo} = 40$ psi (see Fig. 7.14).

(G. G Meyerhof, "Shallow Foundations," *ASCE Journal of Soil Mechanics and Foundation Engineering Division*, vol. 91, no. SM92, 1965; W. G. Holtz and H. J. Gibbs, "Shear Strength of Pervious Gravelly Soils," *Proceedings ASCE*, paper 867, 1956 (www.asce.org); R. B. Peck, W. E. Hanson, and T. H. Thornburn, "Foundation Engineering," John Wiley & Sons, Inc., New York, www.wiley.com.)

Laboratory Test Methods ▪ The limitations in developing representative deformation parameters from reconstituted samples were described in Art. 7.5.5. A possible exception may be for the settlement analyses of foundations supported by compacted fill. Under these circumstances, consolidation tests and stress-path, triaxial shear tests on the fill materials may be appropriate for providing the parameters for application of settlement analyses described for cohesive soils.

(D. J. D'Appolonia, E. D'Appolonia, and R. F. Brisette, "Settlement of Spread Footings on Sand," *ASCE Journal of Soil Mechanics and Foundation Engineering Division*, vol. 94, no. SM3, 1968, www.asce.org.)

Deep Foundations

Subsurface conditions, structural requirements, site location and features, and economics generally dictate the type of foundation to be employed for a given structure.

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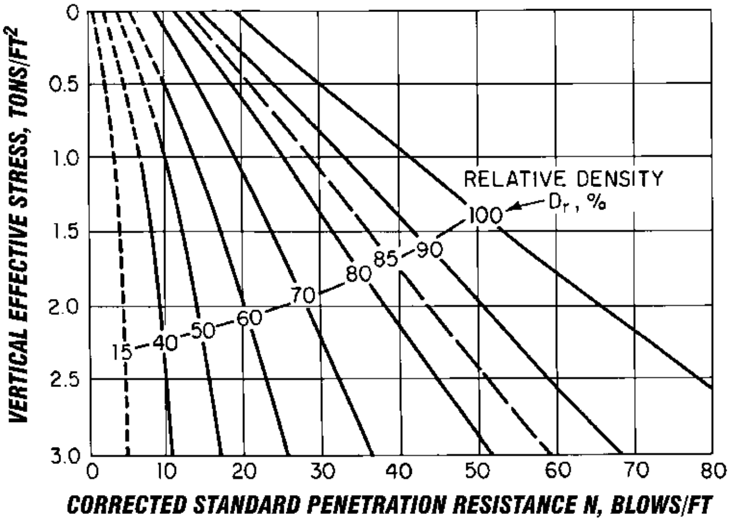

Fig. 7.14 Curves relate relative density to standard penetration resistance and effective vertical stress.

Deep foundations, such as piles, drilled shafts, and caissons, should be considered when:

Shallow foundations are inadequate and structural loads need to be transmitted to deeper, more competent soil or rock

Loads exert uplift or lateral forces on the foundations

Structures are required to be supported over water

Functionality of the structure does not allow for differential settlements

Future adjacent excavations are expected

7.14 Application of Piles

Pile foundations are commonly installed for bridges, buildings, towers, tanks, and offshore structures. Piles are of two major types: prefabricated and installed with a pile-driving hammer, or cast-in-place. In some cases, a pile may incorporate both prefabricated and cast-in-place elements. Driven piles may be made of wood, concrete, steel, or a combination of these materials. Cast-in-place piles are made of concrete that is placed into an auger-drilled hole in the ground. When the diameter of a drilled or augured cast-in-place pile exceeds about 24 in, it is then generally classified as a drilled shaft, bored pile, or caisson (Arts. 7.15.2, 7.2, and 7.23).

The load-carrying capacity and behavior of a single pile is governed by the lesser of the structural strength of the pile shaft and the strength and deformation properties of the supporting soils. When the latter governs, piles derive their capacity from soil resistance along their shaft and under their toe. The contribution of each of these two components is largely dependent on subsurface conditions and pile type, shape, and method of installation. Piles in sand or clay deposits with shaft resistance predominant are commonly known as **friction piles**. Piles with toe resistance primary are known as **end-bearing piles**. In reality, however, most piles have both shaft and toe resistance, albeit to varying degrees. The sum of the ultimate resistance values of both shaft and toe is termed the pile **capacity**, which when divided by an appropriate safety factor yields the **allowable load** at the pile head.

The capacity of a laterally loaded pile is usually defined in terms of a limiting lateral deflection of the pile head. The ratio of the ultimate lateral load defining structural or soil failure to the associated lateral design load represents the safety factor of the pile under lateral load.

Piles are rarely utilized singly, but are typically installed in groups. The behavior of a pile in a group differs from that of the single pile. Often, the

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group effect dictates the overall behavior of the pile foundation system.

The following articles provide a general knowledge of pile design, analysis, construction, and testing methods. For major projects, it is advisable that the expertise of a geotechnical engineer with substantial experience with deep-foundations design, construction, and verification methods be employed.

7.15 Pile Types

Piles that cause a significant displacement of soil during installation are termed **displacement piles**. For example, closed-end steel pipes and precast concrete piles are displacement piles, whereas open-end pipes and H piles are generally **limited-displacement piles**. They may plug during driving and cause significant soil displacement. Auger-cast piles are generally considered **nondisplacement piles** since the soil is removed and replaced with concrete during pile installation.

Piles are usually classified according to their method of installation and type of material. Preformed driven piles may be made of concrete, steel, timber, or a combination of these materials.

7.15.1 Precast Concrete Piles

Reinforced or prestressed to resist handling and pile-driving stresses, precast concrete piles are usually constructed in a casting yard and transported to the jobsite. Pretensioned piles (commonly known as **prestressed piles**) are formed in very long casting beds, with dividers inserted to produce individual pile sections. Precast piles come in a variety of cross sections; for example, square, round, octagonal. They may be manufactured full length or in sections that are spliced during installation. They are suitable for use as friction piles for driving in sand or clay or as end-bearing piles for driving through soft soils to firm strata.

Prestressed concrete piles usually have solid sections between 10 and 30 in square. Frequently, piles larger than 24 in square and more than 100 ft long are cast with a hollow core to reduce pile weight and facilitate handling.

Splicing of precast concrete piles should generally be avoided. When it is necessary to extend pile length, however, any of several splicing methods may be used. Splicing can be accomplished, for instance, by installing dowel bars of sufficient length and then injecting grout or epoxy to bond them and the upper and lower pile sections. Oversize grouted sleeves may also be used. Alternatives to these bonding processes include welding of steel plates or pipes cast at pile ends. Some specialized systems employ mechanical jointing techniques using pins to make the connection. These mechanical splices reduce field splice time, but the connector must be incorporated in the pile sections at the time of casting.

All of the preceding methods transfer some tension through the splice. There are, however, systems, usually involving external sleeves (or *cans*), that do not transfer tensile forces; this is a possible advantage for long piles in which tension stresses would not be high, but these systems are not applicable to piles subject to uplift loading. For prestressed piles, since the tendons require bond-development length, the jointed ends of the pile sections should also be reinforced with steel bars to transfer the tensile forces across the spliced area.

Prestressed piles also may be posttensioned. Such piles are mostly cylindrical (typically up to 66-in diameter and 6-in wall thickness) and are centrifugally cast in sections and assembled to form the required length before driving. Stressing is achieved with the pile sections placed end to end by threading steel cables through precast ducts and then applying tension to the cables with hydraulic devices. Piles up to 200 fit long have been thus assembled and driven.

Advantages of precast concrete piles include their ability to carry high axial and inclined loads and to resist large bending moments. Also, concrete piles can be used as structural columns when extended above ground level. Disadvantages include the extra care required during handling and installation, difficulties in extending and cutting off piles to required lengths, and possible transportation difficulties. Special machines, however, are available for pile cutting, such as saws and hydraulic crushing systems. Care, however, is necessary during all stages of pile casting, handling, transportation, and installation to avoid damaging the piles.

Precast concrete piles are generally installed with pile-driving hammers. For this purpose, pile heads should always be protected with cushioning material. Usually, sheets of plywood are used. Other precautions should also be taken to protect piles during and after driving. When driving is expected

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to be through hard soil layers or into rock, pile toes should generally be fitted with steel shoes for reinforcement and protection from damage. When piles are driven into soils and groundwater containing destructive chemicals, special cement additives or coatings should be used to protect concrete piles. Seawater may also cause damage to concrete piles by chemical reactions or mechanical forces.

("Recommended Practice for Design, Manufacture, and Installation of Prestressed Concrete Piling," Prestressed Concrete Institute, 209 W. Jackson Blvd., Chicago, IL 60604, www.pci.org.)

7.15.2 Cast-in-Place Concrete Piles

These are produced by forming holes in the ground and then filling them with concrete. A steel cage may be used for reinforcement. There are many methods for forming the holes, such as driving of a closed-end steel pipe, with or without a mandrel. Alternatively, holes may be formed with drills or continuous-flight augers. Two common methods of construction are (1) a hole is excavated by drilling before placement of concrete to form a **bored pile**, and (2) a hole is formed with a continuous-flight auger (CFA) and grout is injected into the hole under pressure through the toe of the hollow auger stem during auger withdrawal. A modification of the CFA method is used to create a mixed-in-place concrete pile in clean granular sand. There are numerous other procedures used in constructing cast-in-place concrete piles, most of which are proprietary systems.

Advantages of cast-in-place concrete piles include: relatively low cost, fast execution, ease of adaptation to different lengths, capability for soil sampling during construction at each pile location, possibility of penetrating undesirable hard layers, high load-carrying capacity of large-size piles, and low vibration and noise levels during installation. Construction time is less than that needed for precast piles inasmuch as cast-in-place piles can be formed in place to required lengths and without having to wait for curing time before installation.

Pile foundations are normally employed where subsurface conditions are likely to be unfavorable for spread footings or mats. If cast-in-place concrete piles are used, such conditions may create concerns about the structural integrity, bearing capacity, and general performance of the pile foundation. The reason for this is that the constructed shape and structural integrity of such piles depend on subsurface conditions, concrete quality and method of placement, quality of work, and design and construction practices, all of which require tight control. Structural deficiencies may result from degraded or debonded concrete, necking, or inclusions or voids. Unlike pile driving, where the installation process itself constitutes a crude qualitative pile-capacity test and hammer-pile-soil behavior may be evaluated from measurements made during driving, methods for evaluating cast-in-place piles during construction are generally not available. Good installation procedures and inspection are critical to the success of uncased augured or drilled piles.

("Drilled Shafts: Construction Procedures and Design Methods," 1999, by Michael W. O'Neil and Lymon C. Reese, Report No. FHWA-IF-99-025, Federal Highway Administration, 400 7th Street SW, Washington, D.C. 20590 (www.fhwa.gov) various publications of The International Association of Foundation Drilling (ADSC), P.O. Box 280379, Dallas, TX 75228.)

7.15.3 Steel Piles

Structural steel H and pipe sections are often used as piles. Pipe piles may be driven open- or closed-end. After being driven, they may be filled with concrete. Common sizes of pipe piles range from 8 to 48 in in diameter. A special type of pipe pile is the Monotube, which has a longitudinally fluted wall, may be of constant section or tapered, and may be filled with concrete after being driven. Closed-end pipes have the advantage that they can be visually inspected after driving. Open-end pipes have the advantage that penetration of hard layers can be assisted by drilling through the open end.

H-piles may be rolled or built-up steel sections with wide flanges. Pile toes may be reinforced with special shoes for driving through soils with obstructions, such as boulders, or for driving to rock. If splicing is necessary, steel pile lengths may be connected with complete-penetration welds or commercially available special fittings. H piles, being low-displacement piles, are advantageous in situations where ground heave and lateral movement must be kept to a minimum.

Steel piles have the advantages of being rugged, strong, and easy to handle. They can be driven through hard layers. They can carry high compressive loads and withstand tensile loading. Because

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of the relative ease of splicing and cutting to length, steel piles are advantageous for use in sites where the depth of the bearing layer varies. Disadvantages of steel piles include small cross-sectional area and susceptibility to corrosion, which can cause a significant reduction in load-carrying capacity. Measures that may be taken when pile corrosion is anticipated include the use of larger pile sections than otherwise needed, use of surface-coating materials, or cathodic protection. In these cases, the pipes are usually encased in or filled with concrete.

Specifications pertaining to steel pipe piles are given in "Specification for Welded and Seamless Steel Pipe Piles," ASTM A252 (www.astm.org). For dimensions and section properties of H piles, see "HP Shapes" in "Manual of Steel Construction," American Institute of Steel Construction, One East Wacker Drive, Suite 3100, Chicago, IL 60601-2001.

7.15.4 Timber Piles

Any of a variety of wood species but usually southern pine or douglas fir, and occasionally red or white oak, can be used as piles. Kept below the groundwater table, timber piles can serve in a preserved state for a long time. Untreated piles that extend above the water table, however, may be exposed to damaging marine organisms and decay. Such damage may be prevented or delayed and service life prolonged by treating timber piles with preservatives. Preservative treatment should match the type of wood.

Timber piles are commonly available in lengths of up to 75 ft. They should be as straight as possible and should have a relatively uniform taper. Timber piles are usually used to carry light to moderate loads or in marine construction as dolphins and fender systems.

Advantages of timber piles include their relatively low cost, high strength-to-weight ratio, and ease of handling. They can be cut to length after driving relatively easily. Their naturally tapered shape (about 1 in in diameter per 10 ft of length) is advantageous in situations where pile capacities derive mostly from shaft resistance. Disadvantages include their susceptibility to damage during hard driving and difficulty in splicing.

Timber piles should be driven with care to avoid damage. Hammers with high impact velocities should not be used. Protective accessories should be utilized, when hard driving is expected, especially at the head and toe of the pile.

Specifications relevant to timber piles are contained in "Standard Specifications for Round Timber Piles," ASTM D25; "Establishing Design Stresses for Round Timber Piles," ASTM D2899 (www.astm.org); and "Preservative Treatment by Pressure Processes," AWPA C3, American Wood Preservers Association (www.awpa.com). Information on timber piles also may be obtained from the National Timber Piling Council, Inc., 446 Park Ave., Rye, NY 10580.

7.15.5 Composite Piles

This type of pile includes those made of more than one major material or pile type, such as thick-walled, concrete-filled, steel pipe piles, precast concrete piles with steel (pipe or H section) extensions, and timber piles with cast-in-place concrete extensions.

7.15.6 Selection of Pile Type

The choice of an appropriate pile type for a particular application is essential for satisfactory foundation functioning. Factors that must be considered in the selection process include subsurface conditions, nature and magnitude of loads, local experience, availability of materials and experienced labor, applicable codes, and cost. Pile drivability, strength, and serviceability should also be taken into account. Figure 7.15 presents general

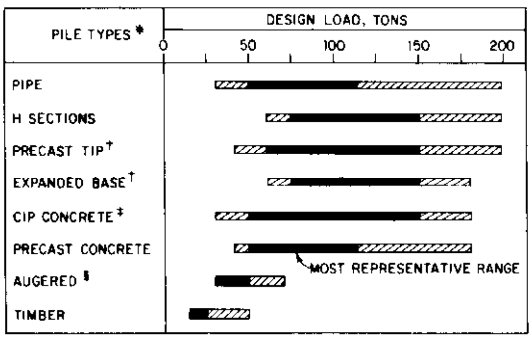

Fig. 7.15 Approximate ranges of design loads for vertical piles in axial compression.

*For shaft diameters not exceeding 18 in.
† Primary end bearing.
‡ Permanent shells only.
§ Uncased only.

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guidelines and approximate ranges of design loads for vertical piles in axial compression. Actual loads that can be carried by a given pile in a particular situation should be assessed in accordance with the general methods and procedures presented in the preceding and those described in more specialized geotechnical engineering books.

7.16 Pile-Driving Equipment

Installation of piles by driving is a specialized field of construction usually performed by experienced contractors with dedicated equipment. The basic components of a pile-driving system are shown in Figure 7.16 and described in the following. All components of the driving system have some effect on the pile-driving process. The overall stability and capacity of the pile-driving crane should be assessed for all stages of loading conditions, including pile pickup and driving.

Lead ▪ The functions of the lead (also known as leader or guide) are to guide the hammer, maintain pile alignment, and preserve axial alignment between the hammer and pile. For proper functioning, leads should have sufficient strength and be straight and well greased to allow free hammer travel.

There are four main types of leads: swinging, fixed, semifixed, and offshore. Depending on the relative positions of the crane and the pile, pile size, and other factors, a specific type of lead may have to be employed. Swinging leads are the simplest, lightest in weight, and most versatile. They do not, however, provide much fixity for prevention of lateral pile movement during driving. Fixed leads maintain the position of the pile during driving and facilitate driving a pile at an inclined angle. However, they are the most expensive type of lead. Semifixed leads have some of the advantages and disadvantages of the swinging and fixed leads. Offshore leads are used mostly in offshore construction to drive large-size steel piles and on land or near shore when a template is used to hold the pile in place. Their use for inclined piles is limited by the pile flexural strength.

Pile Cap (Helmet) ▪ The pile cap (also referred to as helmet) is a boxlike steel element inserted in the lead between the hammer and pile (Fig. 7.17). The function of the cap is to house both hammer and pile cushions and maintain axial alignment between hammer and pile. The size of the cap needed depends on the pile size and the jaw-opening size of the lead. In some cases, an adaptor is inserted under the cap to accommodate various pile sizes, assuring that the hammer and

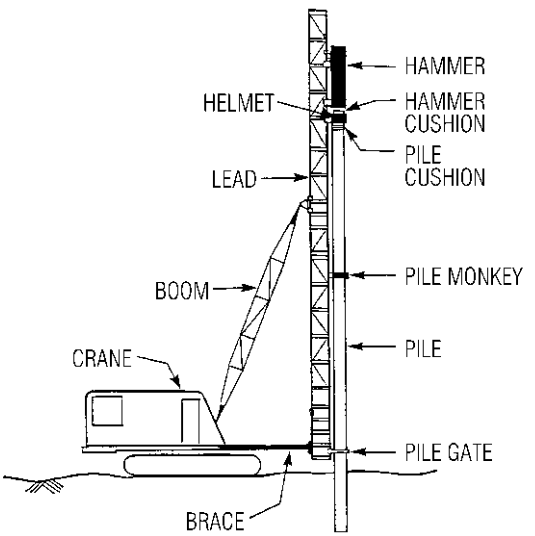

Fig. 7.16 Basic components of a pile-driving rig. (*From "The Performance of Pile Driving Systems—Inspection Manual," FHWA/RD-86/160, Federal High-way Administration.*)

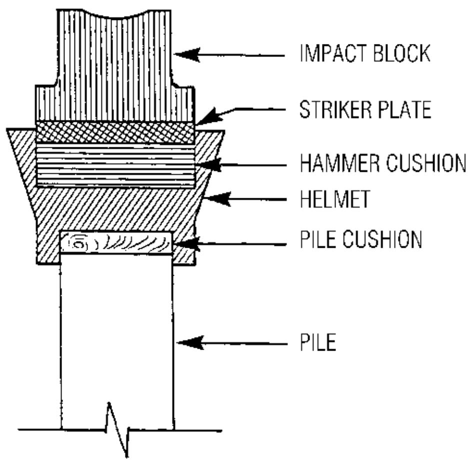

Fig. 7.17 Pile helmet and adjoining parts.

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pile are concentrically aligned. A poor seating of the pile in the cap can cause pile damage and buckling due to localized stresses and eccentric loading at the pile top.

Cushions ▪ Hammers, except for some hydraulic hammers, include a cushion in the hammer (Fig. 7.17). The function of the hammer cushion is to attenuate the hammer impact forces and protect both the pile and hammer from damaging driving stresses. Normally, a steel striker plate, typically 3 in thick, is placed on top of the cushion to insure uniform cushion compression. Most cushions are produced by specialized manufacturers and consist of materials such as phenolic or nylon laminate sheets.

For driving precast concrete piles, a pile cushion is also placed at the pile top (Fig. 7.17). The most common material is plywood. It is placed in layers with total thickness between 4 and 12 in. In some cases, hardwood boards may be used (with the grain perpendicular to the pile axis) as pile cushions. Specifications often require that a fresh pile cushion be used at the start of the driving of a pile. The wood used should be dry. The pile cushion should be changed when significantly compressed or when signs of burning are evident. Cushions (hammer and pile) should be durable and reasonably able to maintain their properties. When a change is necessary during driving, the driving log should record this. The measured resistance to driving immediately thereafter should be discounted, especially if the pile is being driven close to its capacity, inasmuch as a fresh cushion will compress significantly more from a hammer blow than would an already compressed one. Hence, measurements of pile movement per blow will be different.

With the aid of a computer analytical program, such as one based on the wave equation, it is possible to design a cushion system for a particular hammer and pile that allows maximum energy transfer with minimum risk of pile damage.

Hammer ▪ This provides the energy needed for pile installation. Basically, an impact pile-driving hammer consists of a striking part, called the ram, and a means of imposing impacts in rapid succession to the pile.

Hammers are commonly rated by the amount of potential energy per blow. This energy basically is the product of ram weight and drop height (stroke). To a contractor, a hammer is a mass-production machine; hammers with higher efficiency are generally more productive and can achieve higher pile capacity. To an engineer, a hammer is an instrument that is used to measure the quality of the end product, the driven pile. Implicit assumptions regarding hammer performance are included in common pile evaluation procedures. Hammers with low energy transfer are the source of poor installation. Hence, pile designers, constructors, and inspectors should be familiar with operating principles and performance characteristics of the various hammer types. Following are brief discussions of the major types of impact pile-driving hammers.

Impact pile-driving hammers rely on a falling mass to create forces much greater than their weight. Usually, strokes range between 3 and 10 ft.

These hammers are classified by the mode used in operating the hammer; that is, the means used to raise the ram after impact for a new blow. There are two major modes: external combustion and internal combustion. Hammers of each type may be single or double acting. For single-acting hammers (Fig. 7.18), power is only needed to raise the ram, whereas the fall is entirely by gravity. Double-acting hammers also apply power to assist the ram during downward travel. Thus these hammers deliver more blows per minute than single-acting hammers; however, their efficiency may be lower, since the power source supplies part of the impact energy.

External-combustion hammers (ECHs) rely on a power source external to the hammer for their operation. One type is the drop hammer, which is raised by a hoist line from the crane supporting the pile and leads and then dropped to fall under the action of gravity to impact the pile. Main advantages of drop hammers are relatively low cost and maintenance and the ability to vary the stroke easily. Disadvantages include reduction of the effectiveness of the drop due to the cable and winch assembly required for the operation, slow operation, and hammer efficiency dependence on operator's skills. (The operator must allow the cable to go slack when the hammer is raised to drop height.) Consequently, use of drop hammers is generally limited to small projects involving lightly loaded piles or sheetpiles.

For some pile drivers, hydraulic pressure is used to raise the ram. The hammers, known as air/steam

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Fig. 7.18 Single-acting, external-combustion hammer driving a precast concrete pile.

or hydraulic hammers, may be single or double acting. Action starts with introduction of the motive fluid (steam, compressed air, or hydraulic fluid) under the piston in the hammer chamber to lift the ram. When the ram attains a prescribed height, flow of the motive fluid is discontinued and the ram "coasts" against gravity up to the full stroke. At top of stroke, the pressure is vented and the ram falls under gravity. For double-acting hammers, the pressure is redirected to act on top of the piston and push the ram downward during its fall. Many hydraulic hammers are equipped with two stroke heights for more flexibility.

The next cycle starts after impact, and the start should be carefully controlled. If pressure is introduced against the ram too early, it will slow down the ram excessively and reduce the energy available to the pile. Known as **preadmission**, this is not desirable due to its adverse effect on energy transfer. For some hammers, the ram, immediately preceding impact, activates a valve to allow the motive fluid to enter the cylinder to start the next cycle. For most hydraulic hammers, the ram position is detected by proximity switches and the next cycle is electronically controlled.

Main advantages of external combustion hammers include their higher rate of operation than drop hammers, long track record of performance and reliability, and their relatively simple design. Disadvantages include the need to have additional equipment on site, such as boilers and compressors, that would not be needed with another type of hammer. Also disadvantageous is their relatively high weight, which requires equipment with large lifting capacity.

Diesel hammers are internal-combustion hammers (ICHs). The power needed for hammer operation comes from fuel combustion inside the hammer, therefore eliminating the need for an outside power source. Basic components of a diesel hammer include the ram, cylinder, impact block, and fuel distribution system. Hammer operation is started by lifting the ram with one of the hoist lines from the crane or a hydraulic jack to a preset height. A tripping mechanism then releases the ram, allowing it to fall under gravity. During its descent, the ram closes cylinder exhaust ports, as a result of which gases in the combustion chamber are compressed. At some point before impact, the ram activates a fuel pump to introduce into the combustion chamber a prescribed amount of fuel in either liquid or atomized form. The amount of fuel depends on the fuel pump setting.

For liquid-injection hammers, the impact of the ram on the impact block atomizes the fuel. Under the high pressure, ignition and combustion result. For atomized-fuel-injection hammers, ignition occurs when the pressure reaches a certain threshold before impact. The ram impact and the explosive force of the fuel drive the pile into the ground while the explosion and pile reaction throw the ram upward past the exhaust ports, exhausting the combustion gases and drawing in fresh air for the next cycle. With an open-end diesel (OED), shown in Fig. 7.19, the ram continues to travel upward until arrested by gravity. Then the next cycle starts. The distance that the ram travels upward (stroke) depends on the amount of fuel introduced into the chamber (fuel-pump setting), cushions, pile stiffness, and soil resistance. In the case of closed-end diesel hammers (CEDs), the top of the cylinder is closed, creating an air-pressure, or bounce, chamber. Upward movement of the ram compresses the

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Fig. 7.19 Open-end, single-acting diesel hammer driving a pile.

air in the bounce chamber and thus stores energy. The pressure shortens ram stroke and the stored energy accelerates the ram downward.

The energy rating of diesel hammers is commonly appraised by observing the ram stroke (or bounce-chamber pressure for closed-end hammers). This is an important indication but can be misleading; for example, when the hammer becomes very hot during prolonged driving. Because the fuel then ignites too early, the ram expends more energy to compress the gases and less energy is available for transmission into the pile. The high pressure still causes a relatively high stroke. This condition is commonly referred to as preignition. In contrast, short ram strokes may be caused by lack of fuel or improper fuel type, lack of compression in the chamber due to worn piston rings, excessive ram friction, pile stiffness, or lack of soil resistance.

Internal-combustion hammers are advantageous because they are entirely self-contained. They are relatively lightweight and thus permit use of smaller cranes than those required for external-combustion hammers. Also, stroke adjustment to soil resistance with internal-combustion hammers is advantageous in controlling dynamic stresses during driving of concrete piles. Among disadvantages is stroke dependence on the hammer-pile-soil system, relatively low blow rate, and potential cessation of operation when easy driving is encountered.

Table 7.8 presents the characteristics of impact pile-driving hammers. The hammers are listed by rated energy in ascending order. The table indicates for each model the type of hammer: ECH, external combustion hammer, or OED, open-end diesel hammer; manufacturer; model number; ram weight; and equivalent stroke. Note, however, that new models of hammers become available at frequent intervals.

Vibratory hammers drive or extract piles by applying rapidly alternating forces to the pile. The forces are created by eccentric weights (eccentrics) rotating around horizontal axes. The weights are placed in pairs so that horizontal centrifugal forces cancel each other, leaving only vertical-force components. These vertical forces shake piles up and down and cause vertical pile penetration under the weight of the hammer. The vibration may be either low frequency (less than 50 Hz) or high frequency (more than 100 Hz).

The main parameters that define the characteristics of a vibratory hammer are amplitude produced, power consumption, frequency (vibrations per minute), and driving force (resultant vertical force of the rotating eccentrics). Vibratory hammers offer the advantages of fast penetration, limited noise, minimal shock waves induced in the ground, and usually high penetration efficiency in cohesionless soils. A disadvantage is limited penetration capacity under hard driving conditions and in clay soils. Also, there is limited experience in correlating pile capacity with driving energy and penetration rate. This type of hammer is often used to install non-load-bearing piles, such as retaining sheetpiles.

("Vibratory Pile Driving," J. D. Smart, Ph.D. Thesis, University of Illinois, Urbana, 1969; "Driveability and Load Transfer Characteristics of Vibro-Driven Piles," D. Wong, Ph.D Thesis, University of Houston, Texas, 1988; various publications of the

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Table 7.8 Impact Pile-Driving Hammers

Rated Energy kip-ft	Manufacturer	Hammer Model	Ram Weight, kips	Equivalent Stroke, ft	Hammer Type	Rated Energy kip-ft	Manufacturer	Hammer Model	Ram Weight, kips	Equivalent Stroke, ft	Hammer Type	Rated Energy kip-ft	Manufacturer	Hammer Model	Ram Weight, kips	Equivalent Stroke, ft	Hammer Type
1.00	MKT	No. 5	0.20	5.00	ECH	22.13	IHC Hydh	SC 30	3.64	6.08	ECH	32.50	VULCAN	VUL 010	10.00	3.25	ECH
2.50	MKT	No. 6	0.40	6.25	ECH	22.40	MKT	DE 30	2.80	8.00	OED	32.55	FAIRCHLD	F-32	10.85	3.00	ECH
4.15	MKT	No. 7	0.80	5.19	ECH	22.50	FEC	FEC 1200	2.75	8.18	OED	32.90	VULCAN	VUL 100C	10.00	3.29	ECH
7.26	VULCAN	VUL 30C	3.00	2.42	ECH	22.50	ICE	30 S	3.00	7.50	OED	33.00	MKT 33	DE333020	3.30	10.00	OED
7.26	VULCAN	VUL 02	3.00	2.42	ECH	22.99	BERMINGH	B23	2.80	8.21	CED	33.18	UDDCOMB	H4H	8.80	3.77	ECH
8.10	LINKBELT	LB 180	1.73	4.68	CED	22.99	BERMINGH	B23 5	2.80	8.21	CED	34.72	BANUT	6 Tonnes	13.23	2.62	ECH
8.13	ICE	180	1.73	4.70	CED	23.12	ICE	422	4.00	5.78	CED	34.72	Pilemer	DKH-4	8.82	3.95	ECH
8.23	DELMAG	D 5	1.10	7.48	OED	23.14	BANUT	4 Tonnes	8.82	2.62	ECH	35.37	JUNTTAN	HHK 4	8.82	4.01	ECH
8.65	DAWSON	HPH 1200	2.29	3.77	ECH	23.59	DELMAG	D 12	2.75	8.58	OED	35.40	BERMINGH	B250 5	3.00	11.80	OED
8.75	MKT	9B3	1.60	5.47	ECH	23.80	MKT	DA35B SA	2.80	8.50	OED	35.98	VULCAN	VUL 140C	14.00	2.57	ECH
10.50	DELMAG	D 6-32	1.32	7.94	OED	23.80	MKT	DE30B	2.80	8.50	OED	37.38	CONMACO	C 115	11.50	3.25	ECH
13.11	MKT	10B3	3.00	4.37	ECH	24.38	RAYMOND	R 0	7.50	3.25	ECH	37.52	MKT	S 14	14.00	2.68	ECH
14.20	MKT	C5-Air	5.00	2.84	ECH	24.40	MKT	C826 Stm	8.00	3.05	ECH	37.72	ICE	115	11.50	3.28	ECH
15.00	CONMACO	C 50	5.00	3.00	ECH	24.48	RAYMOND	R 80CH	8.00	3.06	ECH	38.20	MKT	DA 55B	5.00	7.64	CED
15.00	RAYMOND	R 1	5.00	3.00	ECH	24.48	RAYMOND	R 80C	8.00	3.06	ECH	38.69	MENCK	MHF3-5	11.02	3.51	ECH
15.00	VULCAN	VUL 01	5.00	3.00	ECH	24.48	VULCAN	VUL 80C	8.00	3.06	ECH	38.69	MENCK	MHF5-5	11.02	3.51	ECH
15.02	LINKBELT	LB 312	3.86	3.89	CED	24.76	MENCK	MHF3-3	7.05	3.51	ECH	39.00	VULCAN	VUL 012	12.00	3.25	ECH
15.10	VULCAN	VUL 50C	5.00	3.02	ECH	24.88	UDDCOMB	H3H	6.60	3.77	ECH	39.25	DELMAG	D 16-32	3.52	11.15	OED
16.00	MKT	DE 20	2.00	8.00	OED	26.00	ICE	32S	3.00	8.67	OED	40.00	CONMACO	C 80E5	8.00	5.00	ECH
16.20	MKT	C5-Steam	5.00	3.24	ECH	28.14	MITSUB.	MH 15	3.31	8.50	OED	40.00	ICE	40-S	4.00	10.00	OED
16.25	MKT	S-5	5.00	3.25	ECH	28.31	DELMAG	D 15	3.30	8.58	OED	40.00	MKT	DA55B SA	5.00	8.00	OED
17.32	DAWSON	HPH 2400	4.19	4.13	ECH	28.92	BANUT	5 Tonnes	11.02	2.62	ECH	40.00	MKT 40	DE333020	4.00	10.00	OED
17.34	BANUT	3 Tonnes	6.61	2.62	ECH	29.25	BERMINGH	B225	3.00	9.75	OED	40.00	VULCAN	VUL 508	8.00	5.00	ECH
17.60	DELMAG	D 8-22	1.76	10.00	OED	29.48	IHC Hydh	SC 40	5.51	5.35	ECH	40.31	BERMINGH	B300	3.75	10.75	OED
18.00	BERMINGH	B200	2.00	9.00	OED	30.36	IHC Hydh	S 40	5.51	5.51	ECH	40.31	BERMINGH	B300 M	3.75	10.75	OED
18.20	LINKBELT	LB 440	4.00	4.55	CED	30.37	ICE	520	5.07	5.99	CED	40.49	BANUT	7 Tonnes	15.43	2.62	ECH
18.56	ICE	440	4.00	4.64	CED	30.41	HERA	1500	3.37	9.02	OED	40.61	DELMAG	D 22	4.91	8.27	OED
19.15	MKT	11B3	5.00	3.83	ECH	30.72	MKT	DA 45	4.00	7.68	CED	40.62	ICE	640	6.00	6.77	CED
19.18	VULCAN	VUL 65C	6.50	2.95	ECH	30.80	MKT	MS-350	7.72	3.99	ECH	40.63	RAYMOND	R 3/0	12.50	3.25	ECH
19.50	CONMACO	C 65	6.50	3.00	ECH	30.96	MENCK	MHF3-4	8.82	3.51	ECH	41.47	UDDCOMB	H5H	11.00	3.77	ECH
19.50	RAYMOND	R 65C	6.50	3.00	ECH	31.33	DELMAG	D 12-32	2.82	11.11	OED	42.00	CONMACO	C 140	14.00	3.00	ECH
19.50	RAYMOND	R 1S	6.50	3.00	ECH	32.00	MKT	DE 40	4.00	8.00	OED	42.00	ICE	42-S	4.09	10.27	OED
19.50	RAYMOND	R 65CH	6.50	3.00	ECH	32.50	CONMACO	C 100	10.00	3.25	ECH	42.00	VULCAN	VUL 014	14.00	3.00	ECH
19.50	VULCAN	VUL 06	6.50	3.00	ECH	32.50	CONMACO	C565	6.50	5.00	ECH	42.40	DELMAG	D 19-32	4.00	10.60	OED
19.57	VULCAN	VUL 65CA	6.50	3.01	ECH	32.50	HPSI	650	6.50	5.00	ECH	42.50	MKT	DE 50B	5.00	8.50	OED
20.00	MKT 20	DE333020	2.00	10.00	OED	32.50	MKT	S 10	10.00	3.25	ECH	43.01	MITSUB.	M 23	5.06	8.50	OED
21.00	MKT	DA 35B	2.80	7.50	CED	32.50	RAYMOND	R 2/0	10.00	3.25	ECH	43.20	BERMINGH	B400 4.8	4.80	9.00	OED
21.20	MKT	C826 Air	8.00	2.65	ECH	32.50	VULCAN	VUL 506	6.50	5.00	ECH	43.37	BSP	HH 5	11.02	3.94	ECH

(Table continued)

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Table 7.8 (*Continued*)

Rated Energy kip-ft	Manufacturer	Hammer Model	Ram Weight, kips	Equivalent Stroke, ft	Hammer Type	Rated Energy kip-ft	Manufacturer	Hammer Model	Ram Weight, kips	Equivalent Stroke, ft	Hammer Type	Rated Energy kip-ft	Manufacturer	Hammer Model	Ram Weight, kips	Equivalent Stroke, ft	Hammer Type
43.40	Pilemer	DKH-5	11.02	3.95	ECH	53.05	JUNTTAN	HHK 6	13.23	4.01	ECH	69.50	BSP	HH 8	17.64	3.94	ECH
44.00	HPSI	110	11.00	4.00	ECH	53.75	BERMINGH	B400	5.00	10.75	OED	69.65	MENCK	MHF5-9	19.84	3.51	ECH
44.00	MKT	MS 500	11.00	4.00	ECH	54.17	MENCK	MHF3-7	15.43	3.51	ECH	70.00	MKT 70	DE70/50B	7.00	10.00	OED
44.23	JUNTTAN	HHK 5	11.03	4.01	ECH	54.17	MENCK	MHF5-7	15.43	3.51	ECH	70.96	HERA	3500	7.87	9.02	OED
44.24	IHC Hydh	SC60	7.72	5.73	ECH	55.99	FEC	FEC 2800	6.16	9.09	OED	72.18	KOBE	K 35	7.72	9.35	OED
44.26	IHC Hydh	S. 60	13.23	3.35	ECH	56.77	HERA	2800	6.29	9.02	OED	72.60	ICE	1070	10.00	7.26	CED
45.00	BERMINGH	B400 5.0	5.00	9.00	OED	56.88	RAYMOND	R 5/0	17.50	3.25	ECH	73.00	FEC	FEC 3400	7.48	9.76	OED
45.00	FAIRCHLD	F-45	15.00	3.00	ECH	57.50	CONMACO	C 115E5	11.50	5.00	ECH	73.66	DELMAG	D 30-32	6.60	11.16	OED
45.07	MENCK	MRBS 500	11.02	4.09	ECH	58.90	IHC Hydh	SC 80	11.24	5.24	ECH	73.66	DELMAG	D 30-23	6.60	11.16	OED
45.35	KOBE	K22-Est	4.85	9.35	OED	59.00	BERMINGH	B400 5	5.00	11.80	OED	75.00	RAYMOND	R 30X	30.00	2.50	ECH
46.43	MENCK	MHF5-6	13.23	3.51	ECH	59.50	MKT	DE 70B	7.00	8.50	OED	77.39	MENCK	MHF5-10	22.04	3.51	ECH
46.43	MENCK	MHF3-6	13.23	3.51	ECH	59.60	DELMAG	D 30	6.60	9.03	OED	77.42	IHC Hydh	SC 110	15.21	5.09	ECH
46.84	MITSUB.	MH 25	5.51	8.50	OED	60.00	CONMACO	C 200	20.00	3.00	ECH	77.88	BERMINGH	B450 5	6.60	11.80	OED
47.20	BERMINGH	B350 5	4.00	11.80	OED	60.00	HPSI	1500	15.00	4.00	ECH	78.17	BSP	HH 9	19.84	3.94	ECH
48.50	DELMAG	D 22-13	4.85	10.00	OED	60.00	ICE	60S	7.00	8.57	OED	80.00	HPSI	2000	20.00	4.00	ECH
48.50	DELMAG	D 22-02	4.85	10.00	OED	60.00	MKT	S 20	20.00	3.00	ECH	80.00	ICE	80S	8.00	10.00	OED
48.75	CONMACO	C 160	16.25	3.00	ECH	60.00	VULCAN	VUL 320	20.00	3.00	ECH	80.41	MITSU8.	M 43	9.46	8.50	OED
48.75	RAYMOND	R 4/0	15.00	3.25	ECH	60.00	VULCAN	VUL 512	12.00	5.00	ECH	81.25	RAYMOND	R 8/0	25.00	3.25	ECH
48.75	RAYMOND	R 150C	15.00	3.25	ECH	60.00	VULCAN	VUL 020	20.00	3.00	ECH	83.82	DELMAG	D 36-13	7.93	10.57	OED
48.75	VULCAN	VUL 016	16.25	3.00	ECH	60.76	Pilemer	DKH-7	15.43	3.95	ECH	83.82	DELMAG	D 36-02	7.93	10.57	OED
49.18	MENCK	MH 68	7.72	6.37	ECH	60.78	BSP	HH 7	15.43	3.94	ECH	83.82	DELMAG	D 36	7.93	10.57	OED
49.76	UDDCOMB	H6H	13.20	3.77	ECH	61.49	DELMAG	D 25-32	5.51	11.16	OED	85.13	MENCK	MHF5-11	24.25	3.51	ECH
50.00	CONMACO	C 100E5	10.00	5.00	ECH	61.71	MITSUB.	M 33	7.26	8.50	OED	85.43	MITSUB.	MH 45	10.05	8.50	OED
50.00	FEC	FEC 2500	5.50	9.09	OED	61.91	JUNTTAN	HHK 7	15.44	4.01	ECH	86.80	Pilemer	DKH-10	22.04	3.95	ECH
50.00	HPSI	1000	10.00	5.00	ECH	61.91	MENCK	MHF5-8	17.64	3.51	ECH	86.88	UDDCOMB	H10H	22.05	3.94	ECH
50.00	MKT 50	DE70/50B	5.00	10.00	OED	62.50	CONMACO	C 125E5	12.50	5.00	ECH	88.00	BERMINGH	B550 C	11.00	8.00	OED
50.00	VULCAN	VUL 510	10.00	5.00	ECH	63.03	FEC	FEC 3000	6.60	9.55	OED	88.42	JUNTTAN	HHK 10	22.05	4.01	ECH
50.20	VULCAN	VUL 200C	20.00	2.51	ECH	64.00	ICE	160	16.00	4.00	ECH	88.50	DELMAG	D 36-32	7.93	11.16	OED
50.69	HERA	2500	5.62	9.02	OED	65.62	MITSUB.	MH 35	7.72	8.50	OED	88.50	DELMAG	D 36-23	7.93	11.16	OED
51.26	DELMAG	D 22-23	4.85	10.57	OED	66.00	DELMAG	D 30-02	6.60	10.00	OED	90.00	HPSI	225	22.50	4.00	ECH
51.52	KOBE	K 25	5.51	9.35	OED	66.00	DELMAG	D 30-13	6.60	10.00	OED	90.00	ICE	90-S	9.00	10.00	OED
51.63	ICE	660	7.57	6.82	CED	66.36	IHC Hydh	S90	9.92	6.69	ECH	90.00	VULCAN	VUL 330	30.00	3.00	ECH
51.63	LINKBELT	LB 660	7.57	6.82	CED	67.77	MENCK	MRBS 750	16.53	4.10	ECH	90.00	VULCAN	VUL 030	30.00	3.00	ECH
51.65	IHC Hydh	S70	7.72	6.69	ECH	69.34	UDDCOMB	H8H	17.60	3.94	ECH	90.44	DELMAG	D 44	9.50	9.52	OED
51.78	CONMACO	160 **	17.26	3.00	ECH	69.43	MENCK	MH 96	11.02	6.30	ECH	92.04	BERMINGH	B500 5	7.80	11.80	OED

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92.75	KOBE	K 45	9.92	9.35	OED	138.87	Pilemer	DKH-16	35.27	3.95	ECH	262.11	MENCK	MRBS250	63.93	4.10	ECH
92.87	MENCK	MHF5-12	26.45	3.51	ECH	141.12	MENCK	MH 195	22.05	6.40	ECH	289.55	MENCK	MHU 400	50.71	5.71	ECH
93.28	MENCK	MRBS 850	18.96	4.92	ECH	147.18	IHC Hydh	S 200	22.00	6.69	ECH	294.60	IHC Hydh	S 400	44.30	6.65	ECH
93.28	MENCK	MRBS 800	18.98	4.92	ECH	147.38	MENCK	MHUT 200	26.46	5.57	ECH	295.12	MENCK	MHUT 400	52.70	5.60	ECH
95.54	BSP	HH 11	24.25	3.94	ECH	149.60	MITSUB.	MH 80B	17.60	8.50	OED	300.00	VULCAN	VUL 3100	100.00	3.00	ECH
96.53	DELMAG	D 46-13	10.14	9.52	OED	150.00	CONMACO	C 5300	30.00	5.00	ECH	300.00	VULCAN	VUL 560	62.50	4.80	ECH
100.00	CONMACO	C 5200	20.00	5.00	ECH	150.00	MKT 150	DE110150	15.00	10.00	OED	325.36	MENCK	MRBS300	66.13	4.92	ECH
100.00	ICE	100	10.00	10.00	OED	150.00	RAYMOND	R 60X	60.00	2.50	ECH	347.16	BSP	HA 40	88.18	3.94	ECH
100.00	RAYMOND	R 40X	40.00	2.50	ECH	150.00	VULCAN	VUL 530	30.00	5.00	ECH	350.00	CONMACO	C 5700	70.00	5.00	ECH
100.00	VULCAN	VUL 520	20.00	5.00	ECH	151.00	IHC Hydh	SC 200	30.20	5.00	ECH	368.30	IHC Hydh	S 500	55.30	6.66	ECH
101.37	HERA	5000	11.24	9.02	OED	152.06	HERA	7500	16.85	9.02	OED	368.55	MENCK	MHUT 500	59.54	6.19	ECH
103.31	IHC Hydh	SC 150	24.25	4.26	ECH	152.45	DELMAG	D 62-02	13.66	11.16	OED	433.64	MENCK	MHU 600	77.16	5.62	ECH
104.80	MENCK	MH 145	16.53	6.34	ECH	152.45	DELMAG	D 62-22	13.66	11.16	OED	498.94	MENCK	MRBS460	101.41	4.92	ECH
106.10	JUNTTAN	HHK 12	26.46	4.01	ECH	152.45	DELMAG	D 62-12	13.66	11.16	OED	500.00	VULCAN	VUL 5100	100.00	5.00	ECH
106.20	BERMINGH	B550 5	9.00	11.80	OED	154.32	HPSI	3005	30.86	5.00	ECH	510.00	CONMACO	C 6850	85.00	6.00	ECH
107.18	DELMAG	D 46-02	10.14	10.57	OED	154.69	MENCK	MHF10-20	44.07	3.51	ECH	513.34	MENCK	MRBS390	86.86	5.91	ECH
107.18	DELMAG	D 46	10.14	10.57	OED	170.00	ICE	205S	20.00	8.50	OED	516.13	MENCK	MHUT700	92.83	5.56	ECH
107.18	DELMAG	D 46-23	10.14	10.57	OED	173.58	BSP	HH 20S	44.09	3.94	ECH	542.33	MENCK	MRBS500	110.23	4.92	ECH
110.00	MKT 110	DE110150	11.00	10.00	OED	173.60	Pilemer	DKH-20	44.10	3.95	ECH	589.85	IHC Hydh	S 800	81.57	7.23	ECH
112.84	Pilemer	DKH-13	28.66	3.95	ECH	173.58	KOBE	KB 80	17.64	9.84	OED	619.18	MENCK	MHUT700	92.83	6.67	ECH
113.16	DELMAG	D 46-32	10.14	11.16	OED	176.32	HPSI	3505	35.26	5.00	ECH	631.40	MENCK	MRBS700	154.00	4.10	ECH
113.60	VULCAN	VUL 400C	40.00	2.84	ECH	176.97	IHC Hydh	SC 250	39.24	4.51	ECH	736.91	MENCK	MHUT100	132.30	5.57	ECH
115.57	HERA	5700	12.81	9.02	OED	178.42	HERA	8800	19.78	9.02	OED	737.26	IHCHydh	S 1000	101.41	7.27	ECH
116.04	MENCK	MHF10-15	33.06	3.51	ECH	179.16	VULCAN	VUL 600C	60.00	2.99	ECH	737.70	MENCK	MHU 1000	126.97	5.81	ECH
120.00	ICE	120S	12.00	10.00	OED	180.00	VULCAN	VUL 360	60.00	3.00	ECH	750.00	VULCAN	VUL 5150	150.00	5.00	ECH
120.00	VULCAN	VUL 340	40.00	3.00	ECH	180.00	VULCAN	VUL 060	60.00	3.00	ECH	759.23	MENCK	MRBS600	132.27	5.74	ECH
120.00	VULCAN	VUL 040	40.00	3.00	ECH	184.64	IHC Hydh	S 250	27.60	6.69	ECH	867.74	MENCK	MRBS800	176.37	4.92	ECH
121.59	BSP	HH 14	30.86	3.94	ECH	186.24	DELMAG	D 80-12	17.62	10.57	OED	954.53	MENCK	MRBS880	194.01	4.92	ECH
123.43	MENCK	MRBS110	24.25	5.09	ECH	189.81	MENCK	MRBS180	38.58	4.92	ECH	1177.80	IHC Hydh	S 1600	156.00	7.55	ECH
123.79	JUNTTAN	HHK 14	30.87	4.01	ECH	196.64	DELMAG	D 80-23	17.62	11.16	OED	1228.87	MENCK	MHU 1700	207.23	5.93	ECH
124.53	DELMAG	D 55	11.86	10.50	OED	200.00	VULCAN	VUL 540	40.90	4.89	ECH	1547.59	MENCK	MHU 2100	255.80	6.05	ECH
125.70	HERA	6200	13.93	9.02	OED	206.51	IHC Hydh	S 280	29.80	6.93	ECH	1581.83	MENCK	MBS12500	275.58	5.74	ECH
130.18	KOBE	KB 60	13.23	9.84	OED	225.00	CONMACO	C 5450	45.00	5.00	ECH	1694.97	IHC Hydh	S 2300	226.60	7.48	ECH
132.50	ICE	120S-15	15.00	8.83	OED	225.95	MENCK	MRBS250	55.11	4.10	ECH	1800.00	VULCAN	VUL 6300	300.00	6.00	ECH
135.15	MITSUB.	MH 72B	15.90	8.50	OED	225.95	MENCK	MRBS250	55.11	4.10	ECH	2171.65	MENCK	MHU 3000	363.76	5.97	ECH
135.59	MENCK	MRBS150	33.07	4.10	ECH	245.85	DELMAG	D100-13	22.03	11.16	OED	2210.12	IHC Hydh	S 3000	332.00	6.66	ECH
138.87	BSP	HH 16	35.27	3.94	ECH	260.37	BSP	HA 30	66.13	3.94	ECH						

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Deep Foundations Institute, 120 Charolette Place, Englewood Cliffs, NJ 07632).

Other Pile Driving Accessories ▪ In addition to the basic equipment discussed in the preceding, some pile driving requires employment of special accessories, such as an adaptor, follower, mandrel, auger, or water jet.

An **adaptor** is inserted between a pile helmet and pile head to make it possible for one helmet to accommodate different pile sizes.

A **follower** is usually a steel member used to extend a pile temporarily in cases where it is necessary to drive the pile when the top is below ground level or under water. For efficiency in transmitting hammer energy to the pile, stiffness of the follower should be nearly equal to that of the pile. The follower should be integrated into the driving system so that it maintains axial alignment between hammer and pile.

Mandrels are typically used to drive steel shells or thin-wall pipes that are later filled with concrete. A mandrel is a uniform or tapered, round steel device that is inserted into a hollow pile to serve as a rigid core during pile driving.

Water jets or **augers** are sometimes needed to advance a pile tip through some intermediate soil layers. Jet pipes may be integrated into the pile shaft or may be external to the pile. Although possibly advantageous in assisting in pile penetration, jetting may have undesirable effects on pile capacity (compression and particularly uplift) that should be considered by the engineer.

(Department of Transportation Federal Highway Administration, "The Performance of Pile Driving Systems: Inspection Manual," FHWA Report No. FHWA/RD-86/160, National Technical Information Service, Springfield, VA 22161, www.ntis.gov.)

7.17 Vibration and Noise

With every hammer blow, pile driving produces vibration and noise effects that may extend a long distance from the source. Hundreds, or even thousands, of hammer blows are typically needed to drive a single pile. These environmental factors are increasingly becoming an issue concerning pile driving activities, particularly in urban areas. Careful planning and execution of pile driving can limit the potential for real damage, and also for litigation based on human perception.

A significant portion of the pile driving hammer energy radiates from the pile into the surrounding ground and propagates away as seismic stress waves. The nature (transient or steady state) and characteristics (frequency, amplitude, velocity, attenuation, etc.) of these traveling waves depend on the type and size of hammer (impact or vibratory), pile (displacement or non-displacement, impedance, length etc.), and subsurface soil conditions. The resulting motion can adversely affect structures above or below ground surface, underground utilities, sensitive equipment and processes, and annoy the public. Structural damage may range from superficial plaster cracking to failure of structural elements. Experience has shown that damage to structures is not likely to occur at a distance greater than the pile embedded depth, or 50 feet minimum (Wood, 1997). In situations where liquefaction or shakedown settlement of loose granular materials may occur, pile driving vibration effects may extend to more than 1000 feet. The following information should be recorded as part of a survey of the pile driving site and surrounding area: distance to the nearest structure or underground utility, function and condition of nearby structures and facilities, and ground conditions of site and vicinity. The Florida Department of Transportation, for example, requires the monitoring of structures for settlement by recording elevations to 0.001 foot within a distance, in feet, of pile driving operations equal to 0.5 times the square root of the hammer energy, in foot-pounds.

People are much more sensitive to ground vibrations than structures. Since they can become annoyed with vibrations that are only 1/100th of those that might be harmful to most structures, human sensitivity should not be used as a measure of vibration for engineering purposes. Vibration limits to prevent damage and human discomfort are not clearly and firmly established. Pile driving vibrations are typically measured by monitoring ground peak particle velocity (ppv) using specialty equipment. Published limits range from 0.2 inch/second (for historical buildings) to 2 inch/second (for industrial structures); the Florida Department of Transportation typically uses 0.5 inch/second as a limiting value. The prediction of the vibration level which may be induced for a particular com-

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bination of hammer, pile, and soil is fraught with difficulties, nevertheless, the literature contains several equations for computing predicted peak particle velocity (Wiss, 1981). Vibration mitigation measures include: active isolation screening by means of a wave barrier near the pile driving location, passive isolation screening by means of a wave barrier near the target affected structure, and pile driving operation controls (jetting, predrilling, change of piling system, hammer type, pile top cushions, and driving sequence). Wave barriers (trenches or sheet pile wall) are attractive, but they are expensive and difficult to design and implement. Design parameters are available in the technical literature (Wood, 1968).

Noise annoys, frustrates and angers people. During the last 30 years there has been increasing concern with the quality of the environment. Through the Noise Control Act of 1972, the United States Congress directed the Environmental Protection Agency (EPA) to publish information about all identifiable effects of noise and to define acceptable levels which would protect the public health and welfare. Impact pile driving is inherently noisy, perhaps the noisiest of all construction operations. Noise is an environmental issue in populated areas, and should be of concern to those involved in pile driving activities. The usual unit of sound measurement is the decibel (dB), and one decibel is the lowest value a normal human ear can detect. The ear also registers pitch in addition to loudness. It is sensitive to frequencies of 0.01 to 16 kHz and most responsive at about 1 to 5 kHz. Measuring devices take this into account, by filtering components outside the most responsive range, and express results in dB(A). Thus, sound intensity, noise, is typically recorded in dB(A). The scale is logarithmic. Apparent loudness doubles for each 10 decibel increment. Typical values are: noisy factory 90 dB(A), busy street 85 dB(A), radio at full volume 70 dB(A), and normal speech 35 dB(A). Most people become annoyed by steady levels above 55 dB(A). Levels below 80 dB(A) would not cause hearing loss, sustained exposure to levels above 90 dB(A) cause physical and mental discomfort and result in permanent hearing damage.

Sources of noise in pile driving operations include hammer (or related equipment) exhaust, impact of hammer, and noise radiating from the pile itself. At distances of 10 to 100 feet, pile driving generally produce levels between 75 to 115 dB(A). Typically, a distance of about 300 feet would be needed for the noise level to be below the OSHA allowed 8-hour exposure (90 dB(A)), and the sound from the noisiest hammer/pile would have to travel miles before decreasing to below moderately annoying levels. Sound level drops by about 6 dB(A) as the distance doubles from the source.

Acoustic shrouds or curtain enclosures have been successfully employed to reduce the pile driving noise, by 15 to 30 dB(A), to below annoying levels. A reduction of 30 dB(A) would make the noisiest pile driving operation acceptable to most people located farther than 500 feet from the pile driving operation.

Smoke is another environmental factor to be considered in planning and executing a pile driving project, especially in metropolitan areas. Sources of smoke are the exhaust from the hammer itself (diesel hammers), the external combustion equipment such as boiler (steam hammers), compressor (air hammers), or power pak (hydraulic hammers). Some modern internal combustion hammers use environmentally friendly fuels.

(Wood, R. D., "Dynamic Effects of Pile Installations on Adjacent Structures", Synthesis of Highway Practice No. 253, National Cooperative Highway Research Program, National Academy Press, Washington, D.C., 1997, 86 pages; Wood, R. D. (1968), "Screening of elastic waves by trenches", ASCE Journal of the Soil Mechanics and Foundations Division, vol. 94, pp 951–979; Wiss, J. F. (1981), "Construction vibrations; state-of-the-art", ASCE Journal of Geotechnical Engineering, vol. 107, No. GT2, pp 167–181, www. asce.org.)

7.18 Pile-Design Concepts

Methods for evaluating load-carrying capacity and general behavior of piles in a foundation range from simple empirical to techniques that incorporate state-of-the-art analytical and field verification methods. Approaches to pile engineering include (1) precedence, (2) static-load analysis (3) static-load testing, and (4) dynamic-load analytical and testing methods. Regardless of the method selected, the foundation designer should possess full knowledge of the site subsurface conditions. This requires consultations with a geotechnical engineer and possibly a geologist familiar with the

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local area to ensure that a sufficient number of borings and relevant soil and rock tests are performed.

Design by precedent includes application of building code criteria, relevant published data, performance of similar nearby structures, and experience with pile design and construction. Under some circumstances this approach may be acceptable, but it is not highly recommended. Favorable situations include those involving minor and temporary structures where failure would not result in appreciable loss of property or any loss of life and construction sites where long-term experience has been accumulated and documented for a well-defined set of subsurface and loading conditions.

Static-load analysis for design and prediction of pile behavior is widely used by designers who are practitioners of geotechnical engineering. This approach is based on soil-mechanics principles, geotechnical engineering theories, pile characteristics, and assumptions regarding pile-soil interaction. Analysis generally involves evaluations of the load-carrying capacity of a single pile, of pile-group behavior, and of foundation settlement under service conditions. Designs based on this approach alone usually incorporate relatively large factors of safety in determination of allowable working loads. Safety factors are based on the engineer's confidence in parameters obtained from soil exploration and their representation of the whole site, anticipated loads, importance of the structure, and the designer's experience and subjective preferences. Static-load analysis methods are used in preliminary design to calculate required pile lengths for cost estimation and bidding purposes. Final pile design and acceptability are based on additional methods of verification. Pile-group behavior and settlement predictions are, however, usually based entirely on static analyses due to the lack of economical and efficient routine field verification techniques.

Field testing should be performed on a sufficient number of piles to confirm or revise initial design assumptions, verify adequacy of installation equipment and procedures, evaluate the effect of subsurface profile variations, and form the basis of final acceptance. Traditionally, piles were tested with a **static-load test** (by loading in axial compression, uplift, or laterally). The number of such tests that will be performed on a site is limited, however, due to the expense and time required when large number of piles are to be installed.

See Art. 7.19 for a description of static-load analysis and pile testing.

Dynamic-load pile testing and analysis is often used in conjunction with or as an alternative to static testing. Analytical methods utilizing computers and numerical modeling and based on one-dimensional elastic-wave-propagation theories are helpful in selecting driving equipment, assessing pile drivability, estimating pile bearing capacity, and determining required driving criteria, that is, blow count. Dynamic analysis is commonly known as *wave equation analysis of pile driving*. Field dynamic testing yields information on driving-system performance of piles: static axial capacity, driving stresses, structural integrity, pile-soil interaction, and load-movement behavior.

See Art. 7.20 for a description of dynamic analysis and pile testing.

(Manual for "Design and Construction of Driven Pile Foundations," 1996, Federal Highway Administration, Report No. FHWA-H1-96-033, Federal Highway Administration, 400 7th Street SW, Washington, D.C. 20590)

7.19 Static-Analysis and Pile Testing

Static analysis of piles and pile design based on it commonly employ global factors of safety. Use of the load-and-resistance-factors approach is growing, however. Steps involved in static analysis include calculation of the static load-carrying capacity of single piles, evaluation of group behavior, and assessment of foundation settlement. Normally, capacity and settlement are treated separately and either may control the design. Pile drivability is usually treated as a separate item and is not considered in static-load analysis. See also Art. 7.18.

7.19.1 Axial-Load Capacity of Single Piles

Pile capacity Q_u may be taken as the sum of the shaft and toe resistances, Q_{su} and Q_{bu} respectively. The allowable load Q_a may then be determined

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from either Eq. (7.35) or (7.36)

$$Q_a = \frac{Q_{su} + Q_{bu}}{F} \tag{7.35}$$

$$Q_a = \frac{Q_{su}}{F_1} + \frac{Q_{bu}}{F_2} \tag{7.36}$$

where F, F_1, F_2, are safety factors. Typically F for permanent structures is between 2 and 3 but may be larger, depending on the perceived reliability of the analysis and construction as well as the consequences of failure. Equation (7.36) recognizes that the deformations required to fully mobilize Q_{su} and Q_{bu} are not compatible. For example, Q_{su} may be developed at displacements less than 0.25 in, whereas Q_{bu} may be realized at a toe displacement equivalent to 5% to 10% of the pile diameter. Consequently, F_1 may be taken as 1.5 and F_2 as 3.0, if the equivalent single safety factor equals F or larger. (If $Q_{su}/Q_{bu} < 1.0$, F is less than the 2.0 usually considered as a minimum safety factor for permanent structures.)

7.19.2 Shaft Resistance in Cohesive Soils

The ultimate stress $\bar{f}_s$ of axially loaded piles in cohesive soils under compressive loads is conventionally evaluated from the ultimate frictional resistance

$$Q_{su} = A_s\bar{f}_s = A_s\alpha\bar{c}_u \tag{7.37}$$

where c_u = average undrained shear strength of soil in contact with shaft surface

A_s = shaft surface area

α = shear-strength (adhesion) reduction factor

One relationship for selection of α is shown in Fig. 7.20. This and similar relationships are empirical and are derived from correlations of load-test data with the c_u of soil samples tested in the laboratory. Some engineers suggest that $\bar{f}_s$ is influenced by pile length and that a limiting value of 1 ton/ft^2 be set for displacement piles less than 50 ft long and reduced 15% for each 50 ft of additional length. This suggestion is rejected by other engineers on the presumption that it neglects the effects of pile residual stresses in evaluation of the results of static-load tests on piles.

The shaft resistance stress $\bar{f}_s$ for cohesive soils may be evaluated from effective-stress concepts:

$$\bar{f}_s = \beta\sigma'_{vo} \tag{7.38}$$

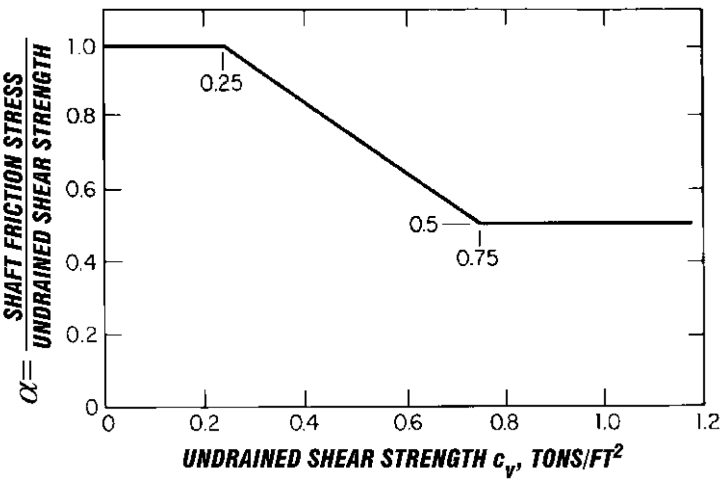

Fig. 7.20 Variation of shear-strength (adhesion) reduction factor α with undrained shear strength. (*After "Recommended Practice for Planning, Designing, and Constructing Fixed Off-Shore Platforms," American Petroleum Institute, Dallas.*)

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where σ'_{vo} = effective overburden pressure of soil

β = function of effective friction angle, stress history, length of pile, and amount of soil displacement induced by pile installation

β usually ranges between 0.22 and 0.35 for intermediate-length displacement piles driven in normally consolidated soils, whereas for piles significantly longer than 100 ft, β may be as small as 0.15. Derivations of β are given by G. G. Meyerhof, "Bearing Capacity and Settlement of Pile Foundations," *ASCE Journal of Geotechnical Engineering Division*, vol. 102, no. GT3, 1976; J. B. Burland, "Shaft Friction of Piles in Clay," *Ground Engineering*, vol. 6, 1973; "Soil Capacity for Supporting Deep Foundation Members in Clay," STP 670, ASTM.

Both the α and β methods have been applied in analysis of n discrete soil layers:

$$Q_{su} = \sum_{i=1}^{n} A_{si}\bar{f}_{si} \tag{7.39}$$

The capacity of friction piles driven in cohesive soils may be significantly influenced by the elapsed time after pile driving and the rate of load application. The frictional capacity Q_s of displacement piles driven in cohesive soils increases with time after driving. For example, the pile capacity after substantial dissipation of pore pressures induced during driving (a typical design assumption) may be three times the capacity measured soon after driving. This behavior must be considered if piles are to be rapidly loaded shortly after driving and when load tests are interpreted.

Some research indicates that frictional capacity for tensile load Q_{ut} may be less than the shaft friction under compression loading Q_{su}. In the absence of load-test data, it is therefore appropriate to take Q_{ut}, as $0.80Q_{su}$ and ignore the weight of the pile. Also, Q_{ut} is fully developed at average pile deformations of about 0.10 to 0.15 in, about one-half those developed in compression. Expanded-base piles develop additional base resistance and can be used to substantially increase uplift resistances.

(V. A. Sowa, "Cast-In-Situ Bored Piles," *Canadian Geotechnical Journal*, vol. 7, 1970; G. G. Meyerhof and J. I. Adams, "The Ultimate Uplift Capacity of Foundations," *Canadian Geotechnical Journal*, vol. 5, no. 4, 1968.)

7.19.3 Shaft Resistance in Cohesionless Soils

The shaft resistance stress $\bar{f}_s$ is a function of the soil-shaft friction angle δ, deg, and an empirical lateral earth-pressure coefficient K:

$$\bar{f}_s = K\bar{\sigma}'_{vo} \tan\delta \leq f_l \tag{7.40}$$

At displacement-pile penetrations of 10 to 20 pile diameters (loose to dense sand), the average skin friction reaches a limiting value f_l. Primarily depending on the relative density and texture of the soil, f_l has been approximated conservatively by using Eq. (7.40) to calculate $\bar{f}_s$. This approach employs the same principles and involves the same limitations discussed in Art. 7.8.2.

For relatively long piles in sand, K is typically taken in the range of 0.7 to 1.0 and δ is taken to be about $\phi - 5$, where ϕ' is the angle of internal friction, deg. For piles less than 50 ft long, K is more likely to be in the range of 1.0 to 2.0 but can be greater than 3.0 for tapered piles.

Empirical procedures have also been used to evaluate $\bar{f}_s$ from in situ tests, such as cone penetration, standard penetration, and relative density tests. Equation (7.41), based on standard penetration tests, as proposed by Meyerhof, is generally conservative and has the advantage of simplicity.

$$\bar{f}_s = \frac{\bar{N}}{50} \tag{7.41}$$

where $\bar{N}$ = average average standard penetration resistance within the embedded length of pile and $\bar{f}_s$ is given in tons/ft^2. (G. G. Meyerhof, "Bearing Capacity and Settlement of Pile Foundations," *ASCE Journal of Geotechnical Engineering Division*, vol. 102, no. GT3, 1976.)

7.19.4 Toe Capacity Load

For piles installed in cohesive soils, the ultimate toe load may be computed from

$$Q_{bu} = A_b q = A_b N_c c_u \tag{7.42}$$

where A_b = end-bearing area of pile

q = bearing capacity of soil

N_c = bearing-capacity factor

c_u = undrained shear strength of soil within zone 1 pile diameter above and 2 diameters below pile tip

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Although theoretical conditions suggest that N_c may vary between about 8 and 12, N_c is usually taken as 9.

For cohesionless soils, the toe resistance stress q is conventionally expressed by Eq. (7.43) in terms of a bearing-capacity factor N_q and the effective overburden pressure at the pile tip σ'_{vo}.

$$q = N_q \sigma'_{vo} \leq q_l \tag{7.43}$$

Some research indicates that, for piles in sands, q, like $\bar{f}_s$, reaches a quasi-constant value q_l after penetrations of the bearing stratum in the range of 10 to 20 pile diameters. Approximately,

$$q_l = 0.5 N_q \tan \phi \tag{7.44}$$

where ϕ is the friction angle of the bearing soils below the critical depth. Values of N_q applicable to piles are given in Fig. 7.21. Empirical correlations of CPT data with q and q_l have also been applied to predict successfully end-bearing capacity of piles in sand. (G. G. Meyerhof, "Bearing Capacity and Settlement of Pile Foundations," *ASCE Journal of Geotechnical Engineering Division*, vol. 102, no. GT3, 1976.)

7.19.5 Pile Settlement

Prediction of pile settlement to confirm allowable loads requires separation of the pile load into shaft friction and end-bearing components. Since q and $\bar{f}_s$ at working loads and at ultimate loads are different, this separation can only be qualitatively evaluated from ultimate-load analyses. A variety of methods for settlement analysis of single piles have been proposed, many of which are empirical or semiempirical and incorporate elements of elastic solutions.

(H. Y. Fang, "Foundation Engineering Handbook," 2nd ed., Van Nostrand Reinhold, New York.)

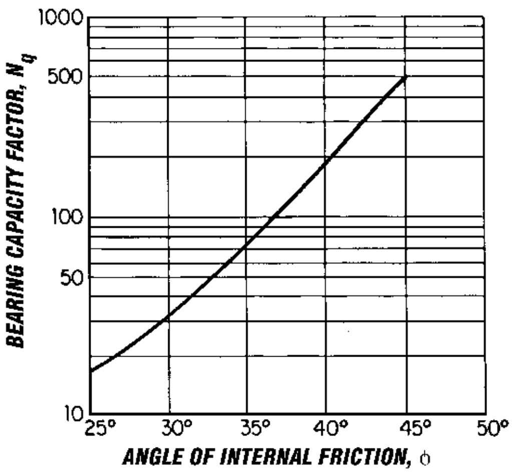

Fig. 7.21 Bearing-capacity factor for granular soils related to angle of internal friction.

7.19.6 Groups of Piles

A pile group may consist of a cluster of piles or several piles in a row. The response of an individual pile in a pile group, where the piles are situated close to one another, may be influenced by the response and geometry of neighboring piles. Piles in such groups interact with one another through the surrounding soil, resulting in what is called the pile-soil-pile interaction, or group effect. The efficiency of a pile group (η_g) is defined as the ratio of the actual capacity of the group to the summation of the capacities of the individual piles in the group when tested as single piles. The pile-soil-pile interaction has two components: pile installation, and loading effects. Analytical models developed to analyze the pile-soil-pile interaction by considering strain superposition in the soil mass neglect the effect of installation and the alteration of the failure zone around an individual pile by those of neighboring piles.

In loose sand, the group efficiency in compression exceeds unity, with the highest values occurring at a pile center to center spacing(s) to diameter or width (d) ratio (s/d) of 2. Generally, higher efficiencies occur with an increase in the number of piles in the group. However, in dense sand, efficiency may be either greater or less than unity, although the trend is toward $\eta_g > 1$. An efficiency smaller than one is probably due to dilatancy and would generally be expected for bored or partially jetted piles. Conventional practice for the analysis of pile groups in sand has been based on assigning a conservative upper bound for η_g of unity for driven piles and 0.67 for bored piles.

Piles in clay always yield values of group efficiencies less than unity with a distinctive trend toward block failure in square groups with an (s/d) ratio of less than 2. Historically, the geotechnical practice was based on a value of η_g of unity for pile

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groups in clay, provided that block failure does not occur and that sufficient time has elapsed between installation and the first application of load to permit excess pore pressure to dissipate.

Several efficiency formulas have been published in the literature. These formulas are mostly based on relating the group efficiency to the spacing between the piles and generally yield efficiency values of less than unity, regardless of the pile/soil conditions. A major apparent shortcoming in most of the efficiency formulas is that they do not account for the characteristics of the soil in contact with the pile group. Comparison of different efficiency formulas show considerable difference in their results. There is no comprehensive mathematical model available for computing the efficiency of a pile group. Any general group efficiency formula that only considers the planar geometry of the pile group should be considered with caution. Soil characteristics, time-dependent effects, cap contact, order of pile driving, and the increase of lateral pressure influence the efficiency of a pile group.

The pile group efficiency formula developed by Sayed and Bakeer (1992) accounts for the three-dimensional geometry of the pile group, soil strength and time-dependent change, and type of embedding soil (cohesive and cohensionless). For a typical configuration of pile group, the group efficiency η_g is expressed as:

$$\eta_g = 1 - (1 - \eta_s' \cdot K) \cdot \rho \qquad (7.45)$$

where η_s' = geometric efficiency

K = group interaction factor, and

ρ = friction factor

This equation is particularly applicable for computing the efficiency of pile groups where a considerable percentage of the load is carried through shaft resistance. For an end bearing pile group, the term ρ becomes practically equal to zero, and accordingly, the formula yields a value of η_g of one. The formula does not account for the contribution of pile cap resistance to the overall bearing capacity of the pile group (neglected due to the potential of erosion or loss of support from settlement of the soil).

For a pile group arranged in a rectangular or square array, the geometric efficiency η_s' is defined as $\eta_s' = P_g/\Sigma P_p$, where P_g = the perimeter of the pile group; and ΣP_p is the summation of the perimeters of the individual piles in the group. Generally, η_s' increases with an increase in the pile spacing-to-diameter (width) ratio s/d, and its typical values are between 0.6 and 2.5.

The factor K is a function of the method of pile installation, pile spacing, and soil type. It is also used to model the change in soil strength due to pile driving (e.g., compaction in cohesionless soils or remolding in cohesive soils). The value of K may range from 0.4 to 9, where higher values are expected in dense cohesionless or stiff cohesive soils and smaller values are expected in loose or soft soils. A value of 1 is obtained for piles driven in soft clay and a value greater than 1 is expected for sands. The appropriate value of K is determined according to the relative density of the sand or the consistency of the clay. For example, a value of 2 to 3 is appropriate in medium-dense sand. These values were back-calculated from the results of several load tests on pile groups.

The friction factor ρ is defined as Q_{su}/Q_u, where Q_{su} and Q_u are the ultimate shaft resistance and total capacity of a single pile, respectively. This factor can be used to introduce the effect of time into the analysis when it is important to assess the short-term as well as the long-term efficiencies of a pile group. This is achieved by considering the gain or loss in the shear strength of the soil in the calculation of Q_{su} and Q_u. Typical values of ρ may range from zero for end-bearing piles to one for friction piles, with typical values of greater than 0.60 for friction or floating type foundations.

Pile dynamic measurements and related analysis (i.e., PDA and CAPWAP) made at the End Of Initial Driving (EOID) and the Beginning Of Restrike (BOR) can provide estimates of the friction factor ρ for the short-term and/or long-term conditions, respectively. For bored piles in cohesive soils, some remolding and possibly lateral stress relief usually occur during construction. It is suggested that the friction factor ρ be determined from a total stress analysis to calculate the short-term efficiency. The long-term efficiency should be based on an effective stress approach. Two types of triaxial tests, unconsolidated-undrained (UU) and consolidated-undrained with pore-pressure measurement (CU), can be used to provide the required strength parameters for the analysis. Moreover, high-strain dynamic tests can be performed on bored piles to provide similar information on the friction factor ρ, analogous to that obtained for driven piles. Some geotechnical

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engineers may prefer to use a total stress approach to compute the ultimate load capacity for both the short and long-term capacities. One can still compute the friction factor ρ for the short-term and/or long-term conditions provided that adequate information is available regarding the thixotropic gain of strength with time.

The three-dimensional modeling of pile groups incorporating the efficiency formula can be performed using the Florida-Pier (FLPIER) computer program developed by the Florida Department of Transportation (FDOT) at the University of Florida. FLPIER considers both axial and lateral pile-soil interaction and group effects. Pile-soil-pile interaction effects are considered through p-y multipliers which are assigned for each row within the group for lateral loading and group efficiency η_g for axial loading. FHWA's computer program COM624 is also available for modeling the lateral pile-soil interaction.

(S. M. Sayed and R. M. Bakeer (1992), "Efficiency Formula For Pile Groups," *Journal of Geotechnical Engineering*, ASCE, 118 No. 2, pages 278–299; S. T. Wang and L. C. Reese, L. C. (1993) "COM624P–Laterally Loaded Pile Analysis Program for the Microcomputer, Version 2.0," *FHWA Office of Technology Applications*, Publication No. FHWA-SA-91-048, Washington, D.C. 20590; Florida Department of Transportation FDOT (1995) "User's Manual for Florida Pier Program", www.dot.state.fl.us)

A very approximate analysis of group settlement, applicable to friction piles, models the pile group as a raft of equivalent plan dimensions situated at a depth below the surface equal to two-thirds the pile length. Subsequently, conventional settlement analyses are employed (see Arts. 7.12 and 7.13).

Negative Skin Friction, Dragload, and Downdrag ▪ Influenced by consolidation induced by placement of fill and/or lowering of the water table, soils along the upper portion of a pile will tend to compress and move down relative to the pile. In the process, load is transferred to the pile through negative skin friction. The permanent load (dead load) on the pile and the dragload imposed by the negative skin friction are transferred to the lower portion of the pile and resisted by means of positive shaft resistance and by toe resistance. A point of equilibrium, called the neutral plane, exists where the negative skin friction changes over into positive shaft resistance. This is where there is no relative movement between the pile and the soil, which means that if the neutral plane is located in non-settling soil, then, the pile does not settle. If on the other hand, the soil experiences settlement at the level of the neutral plane, the pile will settle the same amount, i.e., be subjected to downdrag. Dragload is of concern if the sum of the dragload and the dead load exceeds the structural strength of the pile. The following must be considered:

1. Sum of dead plus live loads is smaller than the pile capacity divided by an appropriate factor of safety. The dragload is not included with these loads.
2. Sum of dead load and dragload is smaller than the structural strength with an appropriate factor of safety. The live load is not included because live load and drag load can not coexist.
3. The settlement of the pile (pile group) is smaller than a limiting value. The live load and dragload are not included in this analysis.

A procedure for construing the neutral plane and determining pile allowable load is illustrated in Fig. 7.22. The diagrams assume that above the neutral plane, the unit negative skin friction, q_n, and positive shaft resistance, r_s, are equal, which is an assumption on the safe side. A key factor is the estimate of the pile toe resistance, R_t. If the pile toe resistance is small, the neutral plane lies higher than when the toe resistance is large. Further, if the pile toe is located in a non-settling soil, the pile settlement will be negligible and only a function of the pile toe penetration necessary to mobilize the pile and bearing resistance. The maximum negative skin friction that can be developed on a single pile can be calculated with Eq. (7.38) with β factors for clay of 0.20 to 0.25, for silt of 0.25 to 0.35, and for sand of 0.35 to 0.50.

A very approximate method of pile-group analysis calculates the upper limit of group drag load Q_{gd} from

$$Q_{gd} = A_F \gamma_F H_F + PHc_u \tag{7.46}$$

H_F, γ_F, and A_F represent the thickness, unit weight, and area of fill contained within the group. P, H, and c_u are the circumference of the group, the thickness of the consolidating soil layers penetrated by the piles, and their undrained shear strength, respect-

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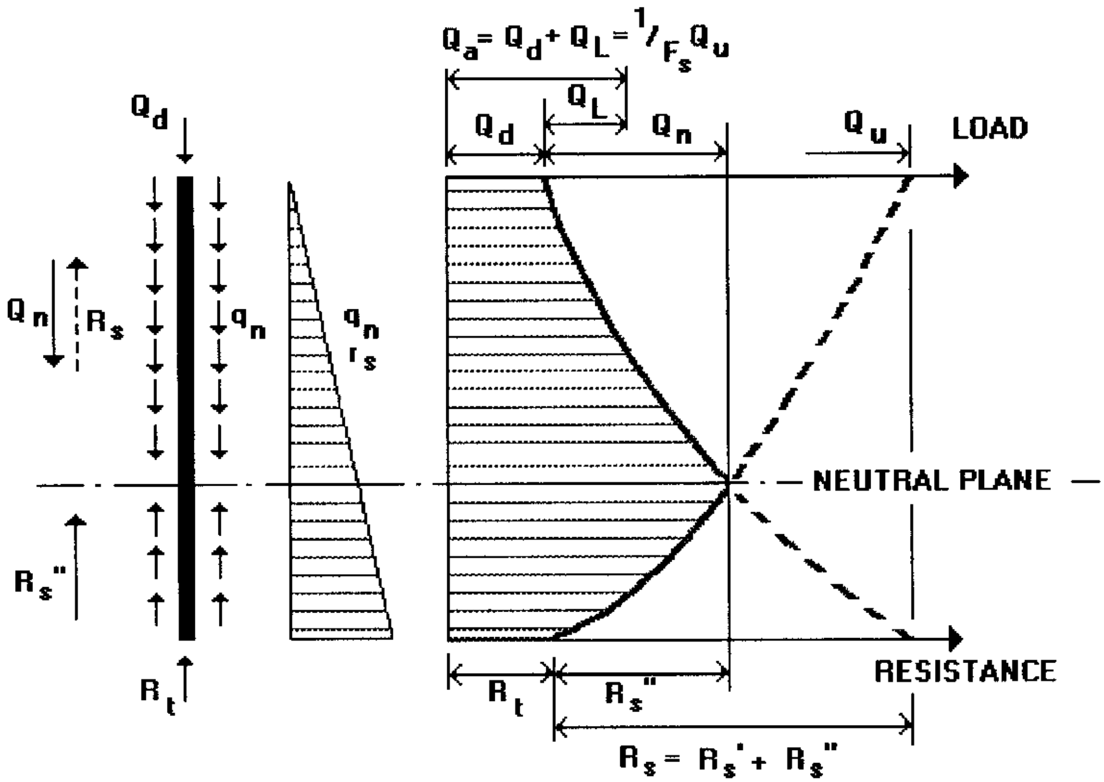

Fig. 7.22 Construing the Neutral Plane and determining allowable load (Guidelines for Static Pile Design, – A Continuing Education Short Course Text, B. H. Fellenius, Deep Foundations Institute, 1991 (www.dfi.org)).

ively. Such forces as Q_{gd} could only be approached for the case of piles driven to rock through heavily surcharged, highly compressible subsoils.

(H. G. Poulos and E. H. Davis, "Elastic Solutions for Soil and Rock Mechanics," and K. Terzaghi and R. B. Peck, "Soil Mechanics and Engineering Practice," John Wiley & Sons, Inc., New York; J. E. Garlanger and W. T. Lambe, *Symposium on Downdrag of Piles*, Research Report 73–56, Soils Publication no. 331, Massachusetts Institute of Technology, Cambridge, 1973; B. H. Fellenius, "Basics of Foundation Design," BiTech Publishers, Richmond, BC, Canada, 1999).

7.19.7 Design of Piles for Lateral Loads

Piles and pile groups are typically designed to sustain lateral loads by the resistance of vertical piles, by inclined, or batter, piles, or by a combination. Tieback systems employing ground anchor or deadmen reactions are used in conjunction with laterally loaded sheetpiles (rarely with foundation piles).

Lateral loads or eccentric loading produce overturning moments and uplift forces on a group of piles. Under these circumstances, a pile may have to be designed for a combination of both lateral and tensile load.

Inclined Piles ▪ Depending on the degree of inclination, piles driven at an angle with the vertical can have a much higher lateral-load capacity than vertical piles since a large part of the lateral load can be carried in axial compression. To minimize construction problems, however, pile batters (rake) should be less than 1 horizontal to 2 vertical.

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Evaluation of the load distribution in a pile group consisting of inclined piles or combined vertical and batter piles is extremely complex because of the three-dimensional nature and indeterminancy of the system. A variety of computer solutions have become available and allow a rational evaluation of the load distribution to inclined group piles. The same methods of axial-capacity evaluation developed for vertical piles are applied to inclined-pile design, although higher driving energy losses during construction suggest that inclined piles would have a somewhat reduced axial-load capacity for the same terminal resistance. (A. Hrennikoff, "Analysis of Pile Foundations with Batter Piles," *ASCE Transactions*, vol. 115, 1950.)

Laterally Loaded Vertical Piles ▪ Vertical-pile resistance to lateral loads is a function of both the flexural stiffness of the pile, the stiffness of the bearing soil in the upper 4D to 6D length of pile, where D = pile diameter, and the degree of pile-head fixity. The lateral-load design capacity is also related to the amount of lateral deflection permitted and, except under very exceptional circumstances, the tolerable-lateral-deflection criteria will control the lateral-load design capacity.

Design loads for laterally loaded piles are usually evaluated by beam theory for both an elastic and nonlinear soil reaction, although elastic and elastoplastic continuum solutions are available. *Nonlinear solutions* require characterization of the soil reaction p versus lateral deflection y along the shaft. In obtaining these solutions, degradation of the soil stiffness by cyclic loading is an important consideration.

The lateral-load vs. pile-head deflection relationship is readily developed from charted nondimensional solutions of Reese and Matlock. The solution assumes the soil modulus K to increase linearly with depth z; that is, $K = n_h z$, where n_h = coefficient of horizontal subgrade reaction. A characteristic pile length T is calculated from

$$T = \sqrt{\frac{EI}{n_h}} \tag{7.47}$$

where EI = pile stiffness. The lateral deflection y of a pile with head free to move and subject to a lateral load P_t and moment M_t applied at the ground line is given by

$$y = A_y P_t \frac{T^3}{EI} + B_y M_t \frac{T^2}{EI} \tag{7.48}$$

where A_y and B_y are nondimensional coefficients. Nondimensional coefficients are also available for evaluation of pile slope, moment, shear, and soil reaction along the shaft.

For positive moment,

$$M = A_m P_t T + B_m M_t \tag{7.49}$$

Positive M_t and P_t values are represented by clockwise moment and loads directed to the right on the pile head at the ground line. The coefficients applicable to evaluation of pile-head deflection and to the maximum positive moment and its approximate position on the shaft z/T, where z = distance below the ground line, are listed in Table 7.9.

The negative moment imposed at the pile head by pile-cap or other structural restraint can be evaluated as a function of the head slope (rotation) from

$$-M_t = \frac{A_\theta P_t T}{B_\theta} - \frac{\theta_s EI}{B_\theta T} \tag{7.50}$$

Table 7.9 Deflection, Moment, and Slope Coefficients

z_{max}	A_y	B_y	A_θ	B_θ	A_m*	B_m*	z/T*
2	4.70	3.39	−3.40	−3.21	0.51	0.84	0.85
3	2.65	1.77	−1.75	−1.85	0.71	0.60	1.49
4	2.44	1.63	−1.65	−1.78	0.78	0.70	1.32
>5	2.43	1.62	−1.62	−1.75	0.77	0.69	1.32

*Coefficients for maximum positive moment are located at about the values given in the table for z/T.

Source: L. C. Reese and H. Matlock, "Non-Dimensional Solutions for Laterally Loaded Piles with Soil Modulus Assumed Proportional to Depth," *8th Texas Conference of Soil Mechanics and Foundation Engineering*, University of Texas, 1956.

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where θ_s, rad, represents the counterclockwise (+) rotation of the pile head and A_θ and B_θ are coefficients (see Table 7.9). The influence of the degrees of fixity of the pile head on y and M can be evaluated by substituting the value of $-M_t$ from Eq. (7.50) in Eqs. (7.48) and (7.49). Note that for the fixed-head case:

$$y_f = \frac{P_t T^3}{EI}\left(A_y - \frac{A_\theta B_y}{B_\theta}\right) \qquad (7.51)$$

Improvement of Lateral Resistance ▪ The lateral-load capacity of a specific pile type can be most effectively increased by increasing the diameter, i.e., the stiffness and lateral-bearing area. Other steps are to improve the quality of the surficial bearing soils by excavation and replacement or in-place densification, to add reinforcement, and to increase the pile-head fixity condition.

Typical lateral-load design criteria for buildings limit lateral pile-head deformations to about ¼ in. Associated design loads for foundation piles driven in medium dense sands or medium clays are typically in the range of 2 to 4 tons, although significantly higher values have been justified by load testing or detailed analyses or a combination.

Resistance of pile groups to lateral loads is not well-documented by field observations. Results of model testing and elastic analysis, however, indicate that pile spacings less than about 8 pile diameters D in the direction of loading reduce the soil modulus K. The reduction factors are assumed to vary linearly from 1.0 at $8D$ to 0.25 at a $3D$ spacing if the number of piles in the group is 5 or more and the passive resistance of the pile cap is ignored. The effect of this reduction is to "soften" the soil reaction and produce smaller lateral resistance for a given group deflection. Elastic analyses also confirm the long-held judgment that batter piles in the center of a pile group are largely ineffective in resisting lateral load.

(B. B. Broms, "Design of Laterally Loaded Piles," *ASCE Journal of Soil Mechanics and Foundation Engineering Division*, vol. 91, no. SM3, 1965. H. Y. Fang, "Foundation Engineering Handbook," Van Nostrand Reinhold, New York. B. H. Fellenius, "Guidelines for Static Pile Design," Deep Foundation Institute, 120 Charolette Place, Englewood Cliffs, NJ 07632 (www.dfi.org). H. G. Poulos and E. H. Davis, "Elastic Solutions for Soil and Rock Mechanics," John Wiley & Sons, Inc., New York. L. C. Reese and R. C. Welch, "Lateral Loading of Deep Foundations in Stiff Clay," *ASCE Journal of Geotechnical Engineering*, vol. 101, no. GT7, 1975.)

7.19.8 Static-Load Pile Testing

Because of the inherent uncertainty in static pile-design methods and the influence of construction procedures on the behavior of piles, static-load tests are desirable or may be required. Static-load tests are almost always conducted on single piles; testing of pile groups is very rare.

Engineers use static-load tests to determine the response of a pile under applied loads. Axial compression testing is the most common, although when other design considerations control, uplift or lateral loading tests are also performed. In some special cases, testing is performed with cyclic loadings or with combined loads; for example, axial and lateral loading. Pile testing may be performed during the design or construction phase of a project so that foundation design data and installation criteria can be developed or verified, or to prove the adequacy of a pile to carry design load.

Use of static-load pile testing is limited by expense and time required for the tests and analyses. For small projects, when testing costs add significantly to the foundation cost, the increased cost often results in elimination of pile testing. For projects involving a large number of piles, static-load pile tests usually are performed, but only a few piles are tested. (A typical recommendation is that of the total number of piles to be installed in normal practice 1% be tested, but the percentage of piles tested in actual practice may be much lower.) The number and location of test piles should be determined by the foundation design engineer after evaluation of the variability of subsurface conditions, pile loadings, type of pile, and installation techniques. Waiting time between pile installation and testing generally ranges from several days to several weeks, depending on pile type and soil conditions.

The foundation contractor generally is responsible for providing the physical setup for conducting a static-load test. The foundation designer should supervise the testing.

Standards detailing procedures on how to arrange and conduct static-load pile tests include "Standard Test Method for Piles under Static Axial Compression Load," ASTM D1143 (www.astm.org); "Standard Method of Testing Individual Piles under Static Axial Tension Load," ASTM D3689;

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and "Standard Method of Testing Piles under Lateral Loads," ASTM D3966. See also "Static Testing of Deep Foundations," U. S. Federal Highway Administration, Report No. FHWA-SA-91-042 (www.fhwa.gov), 1992; "Axial Pile Loading Test—Part 1: Static Loading," International Society for Soil Mechanics and Foundation Engineering, 1985; and "Canadian Foundation Engineering Manual," 2nd ed., Canadian Geotechnical Society, 1985.

Load Application ▪ In a static-load pile test, a hydraulic jack, acting against a reaction, applies load at the pile head. The reaction may be provided by a kentledge, or platform loaded with weights (Fig. 7.23), or by a steel frame supported by reaction piles (Fig. 7.24), or by ground anchors. The distance to be used between the test pile and reaction-system supports depends on the soil conditions and the level of loading but is generally three pile diameters or 8 ft, whichever is greater. It may be necessary to have the test configuration evaluated by a structural engineer.

Hydraulic jacks including their operation should conform to "Safety Code for Jacks," ANSI B30. 1, American National Standards Institute. The jacking system should be calibrated (with load cells, gages, or machines having an accuracy of at least 2%) within a 6-month period prior to pile testing. The available jack extension should be at least 6 in. The jack should apply the load at the center of the pile (Fig. 7.25). When more than one jack is needed for the test, all jacks should be pressurized by a common device.

Loads should be measured by a calibrated pressure gage and also by a load cell placed between the jack and the pile. Internal forces in the pile may be measured with strain gages installed along the pile. Two types of test-loading procedures are used: the *maintained load* (ML) and the *constant rate of penetration* (CRP) methods.

In the ML method, load is applied in increments of 25% of the anticipated pile capacity until failure occurs or the load totals 200% of the design load. Each increment is maintained until pile movement is less than 0.01 in/h or for 2 h, whichever occurs first. The final load is maintained for 24 h. Then, the test load is removed in decrements of 25% of the total test load, with 1 h between decrements. This procedure may require from 1 to 3 days to complete. According to some practices, the ML method is changed to the CRP procedure as soon as the rate exceeds 0.8 in/h.

Tests that consist of numerous load increments (25 to 40 increments) applied at constant time intervals (5 to 15 min) are termed *quick tests*.

In the CRP procedure, the pile is continuously loaded so as to maintain a constant rate of penetration into the ground (typically between 0.01 and 0.10 in/min for granular soils and 0.01 to 0.05 in/min for cohesive soils). Loading is continued until no further increase is necessary for continuous pile penetration at the specified rate. As long as pile penetration continues, the load

Fig. 7.23 Static-load test on a pile with dead weight as the reaction load.

Fig. 7.24 Reaction piles used in static-load test on a pile.

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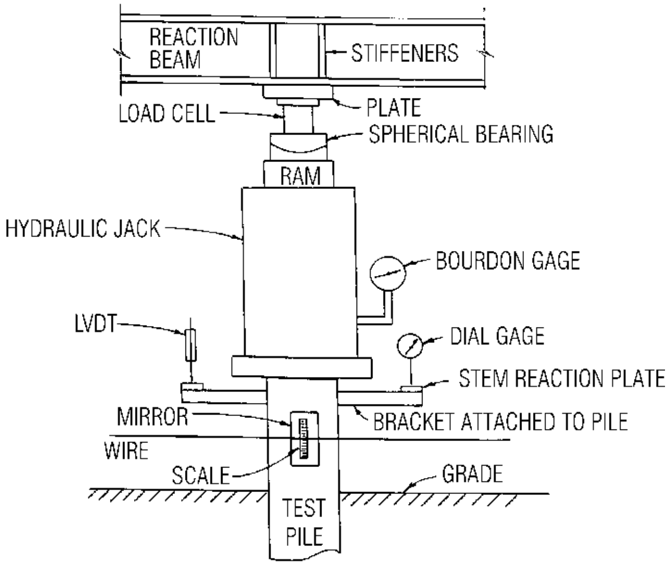

Fig. 7.25 Typical rearrangement of loading equipment and instrumentation at pile head for a compression static-load test. (*From "Static Testing of Deep Foundations," FHWA SA-91-042, Federal Highway Administration*.)

inducing the specified penetration rate is maintained until the total pile penetration is at least 15% of the average pile diameter or diagonal dimension, at which time the load is released. Also, if, under the maximum applied load, penetration ceases, the load is released.

Alternatively, for axial-compression static-load tests, sacrificial jacks or other equipment, such as the Osterberg Cell, may be placed at the bottom of the pile to load it (J. O. Osterberg, "New Load Cell Testing Device," Deep Foundations Institute (www.dfi.org)). One advantage is automatic separation of data on shaft and toe resistance. Another is elimination of the expense and time required for constructing a reaction system, inasmuch as soil resistance serves as a reaction. A disadvantage is that random pile testing is not possible since the loading apparatus and pile installations must be concurrent.

Penetration Measurements ▪ The axial movement of the pile head under applied load may be measured by mechanical dial gages or electromechanical devices mounted on an independently supported (and protected) reference beam. Figure 7.25 shows a typical arrangement of equipment and instruments at the pile head. The gages should have at least 2 in of travel (extendable to 6 in) and typically a precision of at least 0.001 in. For redundancy, measurements may also be taken with a surveyor's rod and precise level and referenced to fixed benchmarks. Another alternative is a tightly stretched piano wire positioned against a mirror and scale that are attached to the side of the pile. Movements at locations along the pile length and at the pile toe may be determined with the use of telltales.

For the ML or quick-test procedures, pile movements are recorded before and after the application of each load increment. For the CRP method, readings of pile movement should be taken at least every 30 s.

Pile-head transverse displacements should be monitored and controlled during the test. For safety and proper evaluation of test results, movements of the reaction supports should also be monitored during the test.

Interpretation of Test Results ▪ A considerable amount of data is generated during a static-load test, particularly with instrumented piles. The most widely used procedure for presenting test results is the plot of pile-head load vs.

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movement. Other results that may be plotted include pile-head time vs. movement and load transfer (from instrumentation along the pile shaft). Shapes of load vs. movement plots vary considerably; so do the procedures for evaluating them for calculation of limit load (often mistakenly referred to as *failure load*).

Problems in data interpretation arise from the lack of a universally recognized definition of *failure*. For a pile that has a load-carrying capacity greater than that of the soil, failure may be considered to occur when pile movement continues under sustained or slightly increasing load (pile plunging). In general, the term *failure load* should be replaced with *interpreted failure load* for evaluations from plots of pile load vs. movement. The definition of interpreted failure load should be based on mathematical rules that produce repeatable results without being influenced by the subjective interpretation of the engineer. In the **offset limit method**, interpreted failure load is defined as the value of the load ordinate of the load vs. movement curve at $\rho + 0.15 + D/120$, where ρ is the movement, in, at the termination of elastic compression and D is the nominal pile diameter, in. One advantage of this technique is the ability to take pile stiffness into consideration. Another advantage is that maximum allowable pile movement for a specific allowable load can be calculated prior to proof testing of a pile. Interpretation methods that rely on extrapolation of the load-movement curve should be avoided. ("Guidelines for the Interpretation and Analysis of the Static Loading Test," 1990, B. H. Fellenius, Deep Foundation Institute, www.dfi.com).

The test report should include the following as well as other relevant data:

1. Information on general site subsurface conditions, emphasizing soil data obtained from exploration near the test pile
2. Descriptions of the pile and pile installation procedure
3. Dates and times of pile installation and static testing
4. Descriptions of testing apparatus and testing procedure
5. Calibration certificates
6. Photographs of test setup
7. Plots of test results
8. Description of interpretation methods
9. Name of testing supervisor

The cost, time, and effort required for a static-load test should be carefully weighed against the many potential benefits. A static-load test on a single pile, however, does not account for the effects of long-term settlement, downdrag loads, time-dependent soil behavior, or pile group action, nor does the test eliminate the need for an adequate foundation design.

7.20 Dynamic Pile Testing and Analysis

Simple observations made during impact pile driving are an important and integral part of the pile installation process. In its most basic form, dynamic-load pile testing encompasses visual observations of hammer operation and pile penetration during pile driving. Some engineers apply equations based on the Newtonian physics of rigid bodies to the pile movements recorded during pile driving to estimate the load-carrying capacity of the pile. The basic premise is that the harder it is to drive the pile into the ground, the more load it will be able to carry. The equations, generally known as energy formulas, typically relate hammer energy and work done on the pile to soil resistance. More than 400 formulas have been proposed, including the widely used and simple *Engineering News formula.*

This method of estimating load capacity, however, has several shortcomings. These include incomplete, crude, and oversimplified representation of pile driving, pile and soil properties, and pile-soil interaction. Often, the method has been found to be grossly inaccurate and unreliable to the extent that many engineers believe that it should be eliminated from contemporary practice.

Modern rational dynamic testing and analysis incorporates pile dynamic measurements analyzed with one-dimensional elastic stress wave propagation principles and theories. Such testing methods have become routine procedures in contemporary foundation engineering practice worldwide. They are covered in many codes and specifications. (ASTM D 4945-96: Standard Test Method for High-Strain Dynamic Testing of Piles; "Application of Stress Wave Theory to Piles: Quality Assurance on Land and Offshore Piling,"

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Proceedings of 6th International Conference, San Paulo, Brazil, 2000, A. A. Balkema Publishers, www.balkema.nl).

7.20.1 Wave Equation

In contrast to the deficiencies of the energy formulas, analysis of pile-driving blow count or penetration per blow yields more accurate estimates of the load-carrying capacity of a pile, if based on accurate modeling and rational principles. One such type of analysis employs the wave equation based on a concept developed by E. A. Smith (*ASCE Journal of Geotechnical Engineering Division*, August 1960). Analysis is facilitated by use of computer programs such as GRLWEAP (Goble Rausche Likins and Associates, Inc., Cleveland, Ohio) that simulate and analyze impact pile driving. Sophisticated numerical modeling, advanced analytical techniques, and one-dimensional elastic-wave-propagation principles are required. Computations can be performed with personal computers. A substantial improvement that the wave equation offers over the energy approach is the ability to model all hammer, cushion, pile-cap, and pile and soil components realistically.

Figure 7.26 illustrates the lumped-mass model used in wave equation analyses. All components that generate, transmit, or dissipate energy are represented by a spring, mass, or dashpot. These permit representation of mass, stiffness, and viscosity.

A series of masses and springs represent the mass and stiffness of the pile. Elastic springs and linear-viscous dashpots model soil-resistance forces along the pile shaft and under the toe. The springs represent the displacement-dependent static-loaded components, and the dashpots the loading-dependent dynamic components. Springs model stiffness and coefficient of restitution (to account for energy dissipation) of hammer and pile cushions. A single mass represents the pile-cap. For external-combustion hammers, the representation is straightforward: a stocky ram, by a single mass; the hammer assembly (cylinder, columns, etc.), by masses and springs. For internal-combustion hammers, the modeling is more involved. The slender ram is divided into several segments. The gas pressure of the diesel combustion cycle is calculated according to the thermodynamic gas law for either liquid or atomized fuel injection.

The parameters needed for execution of a wave equation analysis with the GRLWEAP computer program are:

Hammer: Model and efficiency

Hammer and Pile Cushions: Area, thickness, elastic modulus, and coefficient of restitution

Pile Cap: Weight, including all cushions and any inserts

Pile: Area, elastic modulus, and density, all as a function of length

Soil: Total static capacity, percent shaft resistance and its distribution, quake and damping constants along the shaft and under the toe

In practice, wave equation analysis is employed to deal with the following questions:

1. If the input to the computer program provides a complete description of hammer, cushions, pile cap, pile, and soil, can the pile be driven safely and economically to the required static capacity?
2. If the input provides measurements of pile penetration during pile driving or restriking blow count, what is the static-load capacity of the pile?

For case 1, pile design and proper selection of hammer and driving system can be verified to ensure that expected pile-driving stresses are below allowable limits and reasonable blow count is attainable before actual field work starts. For case 2, given field observations made during pile driving, the analysis is used as a quality-control tool to evaluate pile capacity.

Generally, wave equation analysis is applied to a pile for the cases of several static-load resistances covering a wide range of values (at a constant pile penetration corresponding to the expected final pile-toe depth). Analysis results are then plotted as a *bearing graph* relating static pile capacity and driving stresses to blow counts.

Figure 7.27 presents a bearing graph from an analysis of a single-acting external-combustion hammer (Vulcan 012) and a precast concrete pile (18 in square, 95 ft long).

For a diesel hammer, the stroke or bounce-chamber pressure is also included in the plot. Alternatively, for an open-end diesel hammer (or any hammer with

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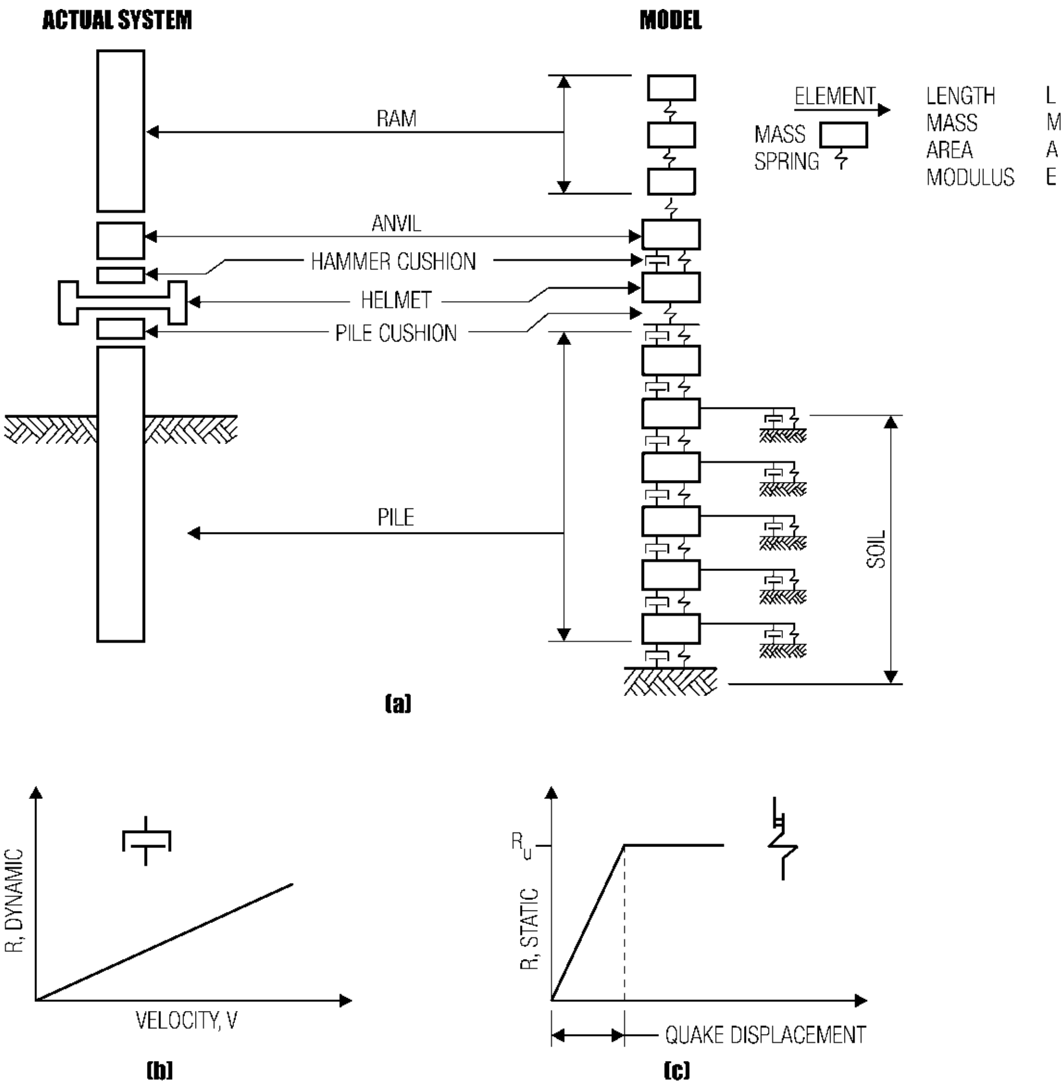

Fig. 7.26 Lumped-mass model of a pile used in wave equation analysis. (*a*) Rectangular block with spring represents mass and stiffness; dashpot, the loading-dependent dynamic components; small square block with spring, the soil resistance forces along the pile shaft. (*b*) Variation of dynamic soil resistance with pile velocity. (*c*) Variation of static soil resistance with pile displacement.

variable stroke), the analysis may be performed with a constant pile static capacity and various strokes. In this way, the required blow count can be obtained as a function of the actual stroke.

Wave equation analysis may also be based on pile penetration (commonly termed pile drivability). In this way, variations of soil resistance with depth can be taken into account. Analysis results are obtained as a function of pile penetration.

Pile specifications prescribe use of wave equation analysis to determine suitability of a pile-driving system. Although it is an excellent tool for analysis of impact pile driving, the wave equation approach has some limitations. These are mainly

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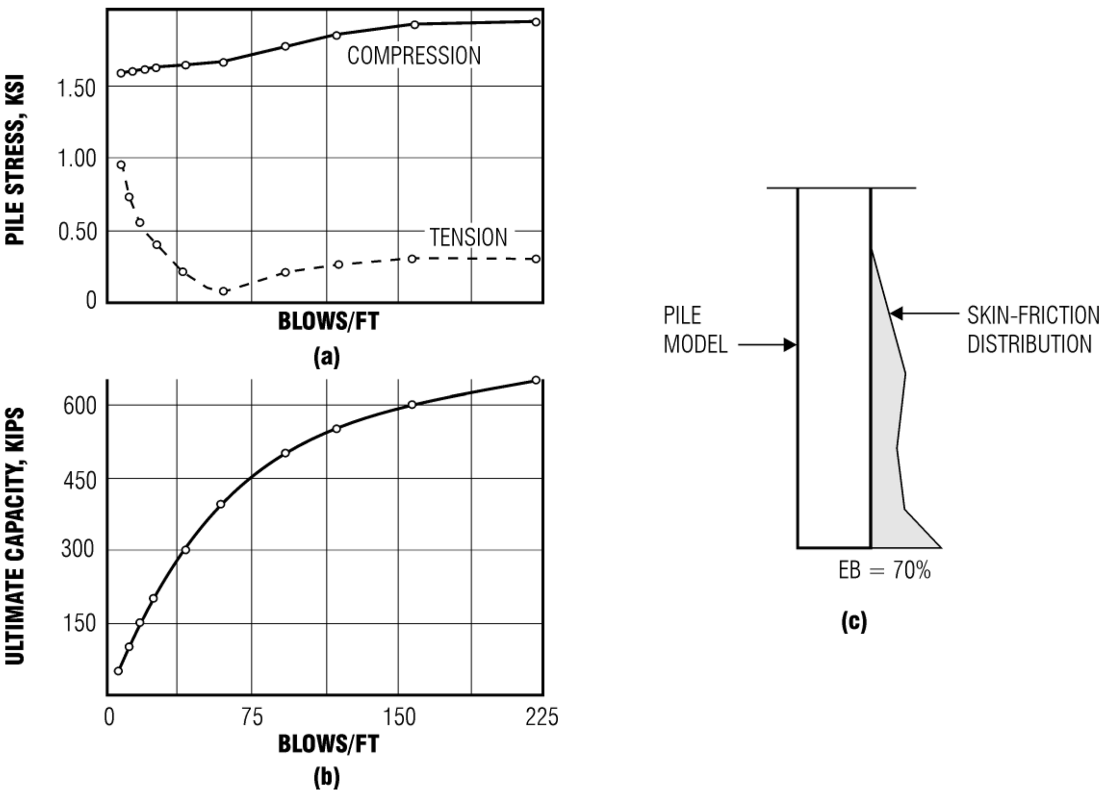

Fig. 7.27 Bearing graph derived from a wave equation analysis. (*a*) Variation of pile tensile and compressive stresses with blows per foot. (*b*) Ultimate capacity of pile indicated by blows per foot. (*c*) Skin friction distribution along the tested pile. Driving was done with a Vulcan hammer, model 012, with 67% efficiency. Helmet weighed 2.22 kips (k). Stiffness of hammer cushion was 5765 k/in and of pile cushion, 1620 k/in. Pile was 95 ft long and had a top area of 324 in^2. Other input parameters were quake (soil maximum elastic deformation), 0.100 in for shaft resistance and 0.150 in for toe resistance; soil damping factor, 0.150 s/ft for shaft and toe resistance.

due to uncertainties in quantifying some of the required inputs, such as hammer performance and soil parameters. The hammer efficiency value needed in the analysis is usually taken as the average value observed in many similar situations. Also, soil damping and quake values (maximum elastic soil deformation) needed in modeling soil behavior cannot be readily obtained from standard field or laboratory soil tests or related to other conventional engineering soil properties.

Dynamic-load pile testing and data analysis yield information regarding the hammer, driving system, and pile and soil behavior that can be used to confirm the assumptions of wave equation analysis. Dynamic-load pile tests are routinely performed on projects around the world for the purposes of monitoring and improving pile installation and as construction control procedures. Many professional organizations have established standards and guidelines for the performance and use of this type of testing; for example, ASTM (D4945), Federal Highway Administration ("Manual on Design and Construction of Driven Pile Foundation"). Dynamic-load testing methods are also effectively employed for evaluating cast-in-place piles ("Dynamic Load Testing of Drilled

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Shaft—Final Report," Department of Civil Engineering, University of Florida, Gainesville 1991).

Main objectives of dynamic-load testing include evaluation of driving resistance and static-load capacity; determination of pile axial stresses during driving; assessment of pile structural integrity; and investigation of hammer and driving system performance.

(ASTM D 4945-96: Standard Test Method for High-Strain Dynamic Testing of Piles; "Dynamic Testing of Pile Foundations During Construction," M. Hussein and G. Likins, Proceedings of ASCE Structures Congress XIII, Boston, MA, 1995).

7.20.2 Case Method

A procedure developed at Case Institute of Technology (now Case Western Reserve University), Cleveland, Ohio, by a research team headed by G. G. Goble, enables calculation of pile static capacity from measurements of pile force and acceleration under hammer impacts during pile driving. The necessary equipment and analytical methods developed have been expanded to evaluate other aspects of the pile-driving process. These procedures are routinely applied in the field using a device called the **Pile Driving Analyzer (PDA)**. As an extension of the original work the researchers developed a computer program known as the CAse Pile Wave Analysis Program (CAPWAP), which is described later.

Measurements of pile force and velocity records under hammer impacts are the basis for modern dynamic pile testing. Data are obtained with the use of reusable strain transducers and accelerometers. Strain gages are bolted on the pile shaft, usually at a distance of about two pile diameters below the pile head. The PDA serves as a data acquisition system and field computer that provides signal conditioning, processing, and calibration of measurement signals. It converts measurements of pile strains and acceleration to pile force and velocity records. Dynamic records and testing results are available in real time following each hammer impact and are permanently stored in digital form. Using wave propagation theory and some assumptions regarding pile and soil, the PDA applies Case method equations and computes in a closed-form solution some 40 variables that fully describe the condition of the hammer-pile-soil system in real time following each hammer impact.

When a hammer or drop weight strikes the pile head, a compressive-stress wave travels down the pile shaft at a speed c, which is a function of the pile elastic modulus and mass density (Art. 6.82.1). The impact induces at the pile head a force F and a particle velocity v. As long as the wave travels in one direction, force and velocity are proportional; that is, $F = Zv$, where Z is the pile impedance, and $Z = EA/c$, where A is the cross-sectional area of the pile and E is its elastic modulus. Changes in impedance in the pile shaft and pile toe, and soil-resistance forces, produce wave reflections. The reflected waves arrive at the pile head after impact at a time proportional to the distance of their location from the toe. Soil-resistance forces or increase in pile impedance cause compressive-wave reflections that increase pile force and decrease velocity. Decrease in pile impedance has the opposite effect.

For a pile of length L, impedance Z, and stress-wave velocity c, the PDA computes total soil resistance from measured force and velocity records during the first stress-wave cycle; that is, when $0 < t \leq 2L/c$, where t is time measured from start of hammer impact. This soil resistance includes both static and viscous components. In the computation of pile bearing capacity under static load RS at the time of testing, effects of soil damping must be considered. Damping is associated with velocity. By definition, the Case method damping force is equal to $ZJ_c v_b$, where J_c is the dimensionless Case damping factor, and v_b the pile toe velocity, which can be computed from measured data at the pile head by applying wave mechanics principles. The static capacity of a pile can be calculated from:

$$RS = \frac{1}{2}[(1 - J_c)(Ft_1 + Zvt_1) + (1 + J_c)(Ft_2 - Zvt_2)] \tag{7.51a}$$

where $t_2 = t_1 + 2L/c$ and t_1 is normally the time of the first relative velocity peak. The damping constant J_c is related to soil grain size and may be taken for clean sands as 0.10 to 0.15, for silty sands as 0.15 to 0.25, for silts as 0.25 to 0.40, for silty clays as 0.4 to 0.7, and for clays as 0.7 to 1.0.

The computed RS value is the pile static capacity at the time of testing. Time-dependent effects can be evaluated by testing during pile restrikes. For this purpose, the pile must have sufficient penetration under the hammer impact to achieve full

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mobilization of soil-resistance forces. (F. Rausche, G. Goble, and G. Likins, "Dynamic Determination of Pile Capacity," *ASCE Journal of Geotechnical Engineering Division*, vol. 111, no. 3, 1985.)

The impact of a hammer subjects piles to a complex combination of compression, tension, torsional, and bending forces. Maximum pile compressive stress at the transducers' location is directly obtained from the measured data as the maximum recorded force divided by the pile area. For piles with mainly toe soil resistance, the compressive force at the pile toe is calculated from pile-head measurements and one-dimensional wave propagation considerations. Maximum tension force in the pile shaft can be computed from measurements near the pile head by considering the magnitude of both upward- and downward-traveling force components. Pile damage occurs if driving stresses exceed the strength of the pile material.

For a pile with a uniform cross-sectional area initially, damage after driving can be indicated by a change in area. Since pile impedance is proportional to pile area, a change in impedance would indicate pile damage. Hence, a driven pile can be tested for underground damage by measuring changes in pile impedance. Changes in pile impedance cause wave reflections and changes in the upward-traveling wave measured at the pile head. From the magnitude and time after impact of the relative wave changes, the extent and location of impedance change and hence of pile damage can be determined. Determination of pile damage can be assisted with the use of the PDA, which computes a relative integrity factor (unity for uniform piles and zero for a pile end) based on measured data near the pile head. (F. Rausche and G. G. Goble, "Determination of Pile Damage by Top Measurements," ASTM STP-670; "Structured Failure of Pile Foundations During Installation," M. J. Hussein and G. G. Goble, ASCE, Proceedings of the Construction Congress VI, Orlando, FL, 2000.)

The PDA also is helpful in determining the energy actually received by a pile from a hammer blow. Whereas hammers are assigned an energy rating by manufacturers, only the energy reaching the pile is of significance in effecting pile penetration. Due to many factors related to the hammer mechanical condition, driving-system behavior, and general dynamic hammer-cushions-pile-soil incompatibility, the percentage of potential hammer energy that actually reaches the pile is quite variable and often less than 50%. ("The Performance of Pile Driving Systems—Main Report," vol. 1–4, FHWADTFH 61-82-1-00059, Federal Highway Administration.) Figure 7.28 presents a summary of data obtained on hundreds of sites to indicate the percentage of all hammers of a specific type with an energy-transfer efficiency less than a specific percentage. Given records of pile force and velocity, the PDA calculates the transferred energy as the time integral of the product of force and velocity. The maximum transferred energy value for each blow represents the single most important parameter for an overall evaluation of driving-system performance.

7.20.3 CAPWAP Method

The CAse Pile Wave Analysis Program (CAPWAP) combines field-measured dynamic-load data and wave-equation-type analytical procedures to predict pile static-load capacity, soil-resistance distribution, soil-damping and quake values, pile load vs. movement plots, and pile-soil load-transfer characteristics. CAPWAP is a signal-matching or system identification method; that is, its results are based on a best possible match between a computed variable and its measured equivalent.

The pile is modeled with segments about 3 ft long with linearly elastic properties. Piles with nonuniform cross sections or composite construction can be accurately modeled. Static and dynamic forces along the pile shaft and under its toe represent soil resistance. Generally, the soil model follows that of the Smith approach (Art. 7.20.1) with modifications to account for full pile-penetration and rebound effects, including radiation damping. At the start of the analysis, an accurate pile model (incorporating splices, if present) is established and a complete set of soil constants is assumed. The hammer model used for the wave equation method is replaced by the measured velocity imposed as a boundary condition. The program calculates the force necessary to induce the imposed velocity. Measured and calculated forces are compared. If they do not agree, the soil model is adjusted and the analysis repeated. This iterative process is continued until no further improvement in the match can be obtained. The total number of unknowns to be evaluated during the analysis is $N_s + 18$, where N_s is the number of soil elements. Typically, one soil

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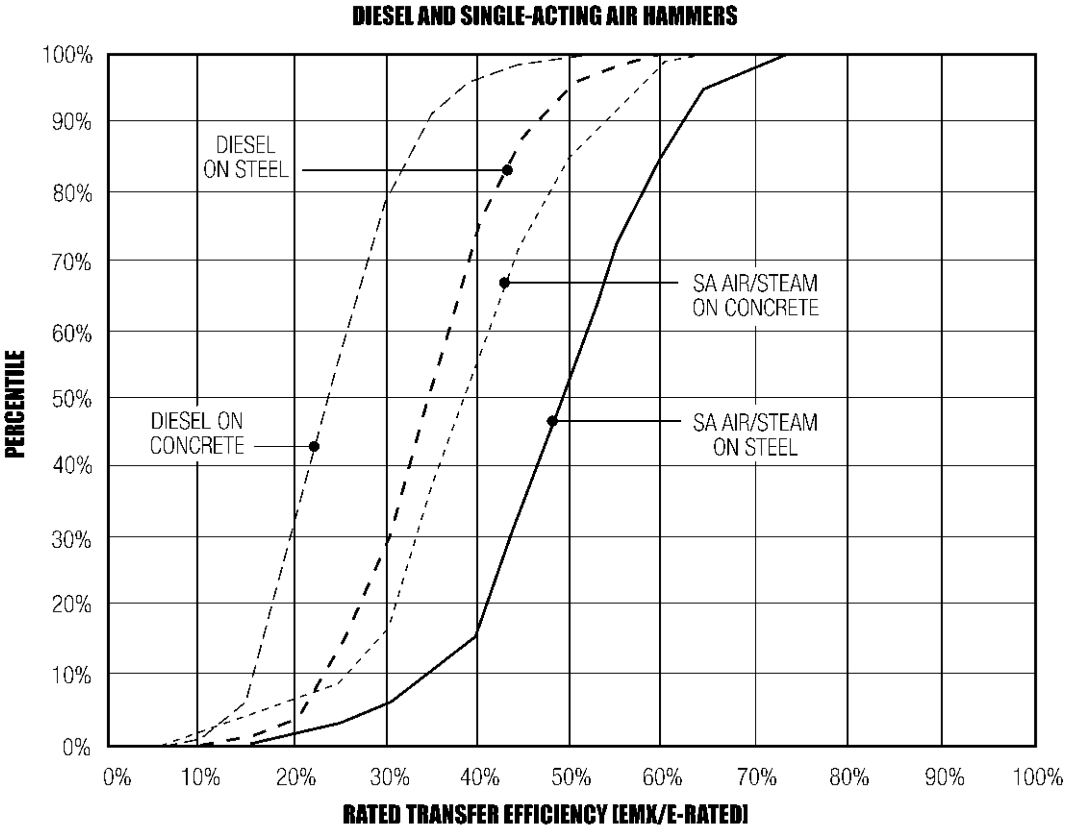

Fig. 7.28 Comparison of performance of two types of hammers when driving steel or concrete piles. The percentile indicates the percentage of all hammers in each case with a rated transfer efficiency less than a specific percentage.

element is placed at every 6 ft of pile penetration plus an additional one under the toe.

Results that can be obtained from a CAPWAP analysis include the following:

Comparisons of measured values with corresponding computed values

Soil-resistance forces and their distribution for static loads

Soil-stiffness and soil-damping parameters along the pile shaft and under its toe

Forces, velocities, displacements, and energies as a function of time for all pile segments

Simulation of the relationship between static loads and movements of pile head and pile toe

Pile forces at ultimate soil resistance

Correlations between CAPWAP predicted values and results from static-load tests indicate very good agreement. (ASCE Geotechnical Special Publication No. 40, 1994.)

7.20.4 Low-Strain Dynamic Integrity Testing

The structural integrity of driven or cast-in-place concrete piles may be compromised during installation. Piles may also be damaged after installation by large lateral movements from impacts of heavy equipment or from slope or retaining-wall failures. Procedures such as excavation around a suspect pile or drilling and coring through its shaft are crude methods for investigating possible pile damage. Several testing techniques are available, however, for evaluation

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of the structural integrity of deep foundation elements in a more sophisticated manner (W. G. Fleming, A. J. Weltmen, M. F. Randolph, and W. K. Elson, "Piling Engineering," Surrey University Press, London). Some of these tests, though, require that the pile be prepared or instrumented before or during installation. These requirements make random application prohibitively expensive, if not impossible. A convenient and economical method is the low-strain pulse-echo technique, which requires relatively little instrumentation and testing effort and which is employed in low-strain, dynamic-load integrity testing. This method is based on one-dimensional wave mechanics principles and the measurement of dynamic-loading effects at the pile head under impacts of a small hand-held hammer. (ASTM D 5882-00: Standard Test Method for Low-Strain Integrity Testing of Piles.)

The following principle is utilized: When impacted at the top, a compressive-stress wave travels down the pile shaft at a constant speed c and is reflected back to the pile head from the toe. Changes in pile impedance Z change wave characteristics and indicate changes in pile cross-sectional size and quality and thus possible pile damage (Art. 7.20.2). Low-strain integrity testing is based on the premise that changes in pile impedance and soil-resistance forces produce predictable wave reflections at the pile head. The time after impact that the reflected wave is recorded at the pile head can be used to calculate the location on the pile of changes in area or soil resistance.

Field equipment consists of an accelerometer, a hand-held hammer (instrumented or without instrumentation), dedicated software, and a Pile Integrity Tester (Fig. 7.29), a data acquisition system capable of converting analog signals to digital form, data processing, and data storage. Pile preparation involves smoothing and leveling of a small area of the pile top. The accelerometer is affixed to the pile top with a jell-type material, and hammer blows are applied to the pile head. Typically, pile-head data resulting from several hammer blows are averaged and analyzed.

Data interpretation may be based on records of pile-top velocity (integral of measured acceleration), data in time or frequency domains, or more rigorous dynamic analysis. For a specific stress-wave speed (typically 13,000 ft/s), records of velocity at the pile head can be interpreted for pile

Fig. 7.29 Pile Integrity Tester. (*Courtesy of Pile Dynamics, Inc., Cleveland, Ohio.*)

nonuniformities and length. As an example, Fig. 7.30 shows a plot in which the abscissa is time, measured starting from impact, and the ordinate is depth below the pile top. The times that

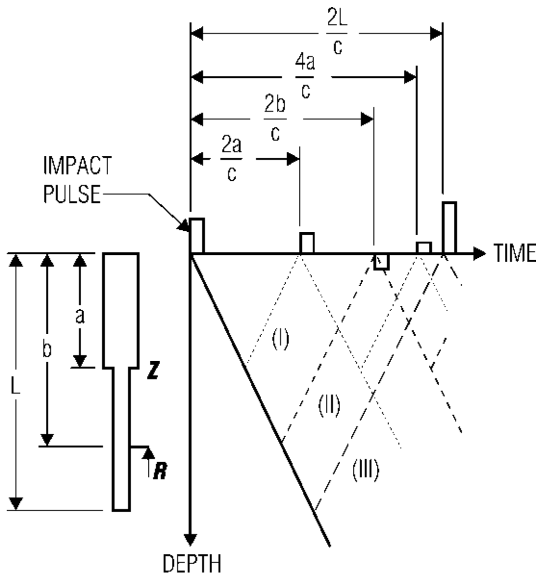

Fig. 7.30 Graph relates distance from a pile head to a depth where a change in the pile cross section or soil resistance occurs to the time it takes an impact pulse applied at the pile head and traveling at a velocity c to reach and then be reflected from the change back to the pile head. Line I indicates the reflection due to impedance, II the reflection due to passive resistance R (modeled velocity proportional), and III the reflection from the pile toe.

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changes in wave characteristics due to pile impedance or soil resistance are recorded at the pile head are represented along the time axis by small rectangles. The line from the origin extending downward to the right presents the position of the wave traveling with velocity c after impact. Where a change in pile impedance Z occurs, at depth a and time a/c, a line (I) extends diagonally upward to the right and indicates that the wave reaches the pile top at time $2a/c$. Hence, with the time and wave velocity known, the distance a can be calculated. Similarly, from time $2b/c$, as indicated by line II, the distance b from the pile top of the change in soil resistance R can be computed. Line III indicates that the wave from the toe at distance L from the pile head reaches the head at time $2L/c$.

Dynamic analysis may be done in a signal-matching process or by a method that generates a pile impedance profile from the measured pile-top data. (F. Rausche et al., "A Formalized Procedure for Quality Assessment of Cast-in-Place Shafts Using Sonic Pulse Echo Methods," Transportation Research Board, Washington, D.C. 1994.)

The low-strain integrity method is applicable to concrete (cast-in-place and driven) and wood piles. Usually, piles are tested shortly after installation so that deficiencies may be detected early and corrective measures taken during foundation construction and before erection of the superstructure. As for other nondestructive testing methods, the results of measurements recorded may be divided into four main categories: (1) clear indication of a sound pile, (2) clear indication of a serious defect, (3) indication of a somewhat defective pile, and (4) records do not support any conclusions. The foundation engineer, taking into consideration structural, geotechnical, and other relevant factors, should decide between pile acceptability or rejection.

The low-strain integrity method may be used to determine the length and condition of piles under existing structures. (M. Hussein, G. Likins, and G. Goble, "Determination of Pile Lengths under Existing Structures," Deep Foundations Institute, 1992, www.dfi.org.)

The method has some limitations. For example, wave reflections coming from locations greater than about 35 pile diameters may be too weak to be detected at the pile head with instruments currently available. Also, gradual changes in pile impedance may escape detection. Furthermore, the method may not yield reliable results for steel piles.

Concrete-filled steel pipe piles may be evaluated with this method.

7.21 Specification Notes

Specifications for pile installation should provide realistic criteria for pile location, alignment, and minimum penetration or termination driving resistance. Particular attention should be given to provisions for identification of pile heave and relaxation and for associated remedial measures. Corrective actions for damaged or out-of-position piles should also be identified. Material quality and quality control should be addressed, especially for cast-in-place concrete piles. Tip protection of piles is an important consideration for some types of high-capacity, end-bearing piles or piles driven through obstructions. Other items that may be important are criteria for driving sequence in pile groups, preexcavation procedures, protection against corrosive subsoils, and control of pile driving in proximity to open or recently concreted pile shells. Guidelines for selected specification items are in Table 7.10.

The following is a list of documents containing sample guidelines, standards, and specifications related to deep foundations design and construction. Clear, comprehensive, reasonable, and fair project specifications greatly reduce the potential for disputes and costly delays.

"Guidelines for writing Construction Specifications for Piling" — Deep Foundations Institute, www.dfi.org.

"Recommended Design Specifications for Driven Bearing Piles" — Pile Driving Contractors Association, www.piledrivers.org.

"Standard Guidelines for the Design and Installation of Pile Foundations" — American Society of Civil Engineers, www.asce.org.

"Standards and Specifications for the Foundation Drilling Insustry" — International Association of Foundation Drilling, www.adsc-iafd.com.

"Standard Specifications for Road and Bridge Construction, Section 455: Structures Foundations" — Florida Department of Transportation, www.dot.state.fl.us.

"Standard Specification for the Construction of Drilled Piers" — Americal Concrete Institute, www.aci-int.org.

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Table 7.10 Guide to Selected Specification Provisions

Position	... within 6 in of plan location (3 in for pile groups with less than 5 piles)
Plumbness	... deviation from vertical shall not exceed 2% in any interval (4% from axis for batter piles)
Pile-hammer assembly	Verification of the suitability of the proposed hammer assembly to drive the designated piles shall be provided by wave equation or equivalent analysis subject to the engineer's approval.
Pile-driving leaders	All piles shall be driven with fixed leaders sufficiently rigid to maintain pile position and axial alignment during driving.
Driving criteria	... to an elevation of at least ___ and/or to a terminal driving resistance of __ blows/__in
Indicator piles	Before start of production driving, indicator piles should be driven at locations determined by the engineer. Continuous driving resistance records shall be maintained for each indicator pile.
Preboring	Preboring immediately preceding pile installation shall extend to elevation ___. The bore diameter for friction piles shall not be less than 1 or more than 2 in smaller than the pile diameter.
Heave*	The elevation of pile butts or tips of CIPC pile shells shall be established immediately after driving and shall be resurveyed on completion of the pile group. Should heave in excess of ¼ in be detected, the piles shall be redriven to their initial elevation or as directed by the engineer.
Relaxation or setup†	Terminal driving resistance of piles in-place for at least 24 h shall be redriven as directed by the engineer.

* Can be initially conducted on a limited number of pile groups and subsequently extended to all piles if required.

† May be specified as part of initial driving operations.

"Recommended Practice for Design, Manufacturing, and Installation of Prestressed Concrete Piling" — Precast/Prestressed Concrete Institute, www.pci.org.

"International Building Code — Chapter 18: Soils and Foundations" — International Code Council, 5203 Leesburg Pike, Falls Church, Virginia 22041.

7.22 Drilled Shafts

Drilled shafts are commonly used to transfer large axial and lateral loads to competent bearing materials by shaft or base resistance or both. Also known as drilled piers, drilled-in caissons, or large-diameter bored piles, drilled shafts are cylindrical, cast-in-place concrete shafts installed by large-diameter, auger drilling equipment. Shaft diameters commonly range from 2.5 to 10 ft and lengths from 10 to 150 ft, although shafts with dimensions well outside these ranges can be installed. Shafts may be of constant diameter (straight shafts, Fig. 7.31*a*) or may be underreamed (belled Fig. 7.31*b*) or socketed into rock (Fig. 7.31*c*). Depending on load requirements, shafts may be concreted with or without steel reinforcement.

Under appropriate foundation conditions, a single drilled shaft is well-suited for support of very heavy concentrated loads; 2000 tons with rock bearing is not unusual.

Subsurface conditions favoring drilled shafts are characterized by materials and groundwater

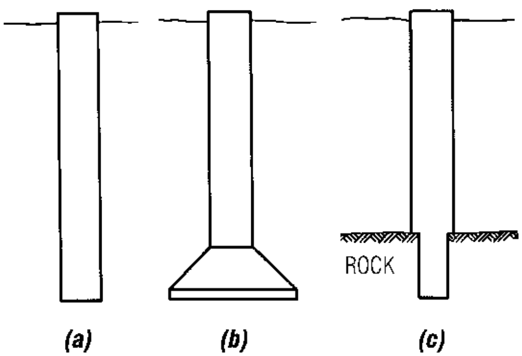

Fig. 7.31 Types of drilled shafts.

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conditions that do not induce caving or squeezing of subsoils during drilling and concrete placement. High-capacity bearing levels at moderate depths and the absence of drilling obstructions, such as boulders or rubble, are also favorable conditions. Current construction techniques allow drilled shafts to be installed in almost any subsurface condition, although the cost-effectiveness or reliability of the system will vary significantly.

7.22.1 Construction Methods for Drilled Shafts

In stable soil deposits, such as stiff clays, concrete with or without reinforcement may be placed in uncased shafts. Temporary casing, however, may be employed during inspection of bearing conditions. Temporary casing may also be installed in the shaft during or immediately after drilling to prevent soil intrusion into the concrete during placement. During this process, the height of concrete in the casing should at all times be sufficient so that the weight more than counterbalances hydrostatic heads imposed by groundwater or by fluid trapped in the annular space between the soil and the casing. The lack of attention to this requirement is perhaps the greatest contributor to drilled-shaft failures.

Unstable soil conditions encountered within a limited interval of shaft penetration can be handled by advancing a casing into stable subsoils below the caving zone via vibratory driving or by screwing down the casing with a torque-bar attachment. The hole is continued by augering through the casing, which may subsequently be retracted during concrete placement or left in place. A shaft through unstable soils may also be advanced without a casing if a weighted drilling fluid (slurry) is used to prevent caving.

For a limited unstable zone, underlain by relatively impervious soils, a casing can be screwed into these soils so as to form a water seal. This allows the drilling slurry to be removed and the shaft continued through the casing and completed by normal concreting techniques.

The shaft may also be advanced entirely by slurry drilling techniques. With this method, concrete is tremied in place so as to completely displace the slurry.

Reinforcing steel must be carefully designed to be stable under the downward force exerted by the concrete during placement. Utilization of reinforcement that is not full length is not generally recommended where temporary casing is used to facilitate concrete placement or slurry methods of construction.

Concrete can be placed in shafts containing not more than about 4 in of water (less for belled shafts). Free-fall placement can be used if an unobstructed flow is achieved. Bottom-discharge hoppers centered on the shaft facilitate unrestricted flow, whereas flexible conduits (elephant trunks) attached to the hopper can be used to guide concrete fall in heavily reinforced shafts. Rigid tremie pipes are employed to place concrete in water or in slurry-filled shafts.

Equipment and Tools ▪ Large-diameter drills are crane- and truck-mounted, depending on their size and weight. The capacity of the drill is rated by its maximum continuous torque, ft-lb, and the force exerted on the drilling tool. This force is the weight of the **Kelly bar** (drill stem) plus the force applied with some drills by their Kelly-bar down-crowd mechanism.

Downward force on the auger is a function of the Kelly-bar length and cross section. Telescoping Kelly bars with cross sections up to 12 in square have been used to drill 10-ft-diameter shafts in earth to depths over 220 ft. Solid pin-connected Kelly sections up to 8 in square have also been effectively used for drilling deep holes. Additional down-crowd forces exerted by some drills are on the order of 20 to 30 kips (crane-mounts) and 15 to 50 kips (truck-mounts).

Drilling tools consisting of open helix (single-flight) and bucket augers are typically used for earth drilling and may be interchanged during construction operations. To drill hard soil and soft and weathered rock more efficiently, flight augers are fitted with hard-surfaced teeth. This type of auger can significantly increase the rate of advance in some materials and provides a more equitable definition of "rock excavation" when compared to the refusal of conventional earth augers. Flight augers allow a somewhat faster operation and in some circumstances have a superior penetration capability. Bucket augers are usually more efficient for excavating soft soils or running sands and provide a superior bottom cleanout.

Belled shafts in soils and soft rocks are constructed with special underreaming tools. These are usually limited in size to a diameter three times

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the diameter of the shaft. Hand mining techniques may be required where hard seams or other obstructions limit machine belling.

Cutting tools consisting of roller bits or core barrels are typically used to extend shafts into harder rock and to form rock sockets. Multiroller bits are often used with a *reverse circulation* type of rotary drilling rig. This technique, together with percussion bits activated by pneumatically powered drills, usually provide the most rapid advance in rock but have the disadvantage of requiring special drill types that may not be efficient in earth drilling.

7.22.2 Construction Quality Control and Assurance for Drilled Shafts

During the preparation of drilled-shaft design and construction specifications, special attention should be given to construction-related design features, including shaft types, shaft-diameter variations, site trafficability, ground-loss potential, and protection of adjacent facilities. Proper technical specifications and contract provisions for such items as payment for rock excavation and drilling obstructions are instrumental in preventing significant cost overruns and associated claims.

Some cost-reduction or quality-related precautions in preparation of drilled-shaft designs and specifications are:

1. Minimize the number of different shaft sizes; extra concrete quantities for diameters larger than actually needed are usually far less costly than use of a multiplicity of drilling tools and casings.
2. Delete a requirement for concrete vibration and use concrete slumps not less than 6 ± 1 in.
3. Do not leave a casing in place in oversized holes unless pressure grouting is used to prevent ground loss.
4. Shaft diameters should be at least 2.5 ft, preferably 3 ft.

Tolerances for location of drilled shafts should not exceed 3 in or $\frac{1}{24}$ of the shaft diameter, whichever is less. Vertical deviation should not be more than 2% of the shaft length or 12.5% of the diameter, whichever controls, except for special conditions.

Provisions for proof testing are extremely important for shafts designed for high-capacity end bearing. This is particularly true for bearing materials that may contain discontinuities or have random variations in quality. Small percussion drills (jackhammers) are often used for proof testing and may be supplemented by diamond coring, if appropriate.

Because many drilled-shaft projects involve variations in bearing levels that cannot be quantified during the design stage, the limitations of the bearing level and shaft quantities estimated for bidding purposes must be clearly identified. Variations in bearing level and quality are best accommodated by specifications and contract provisions that facilitate field changes. The continuous presence of a qualified engineer inspector, experienced in drilled-shaft construction, is required to ensure the quality and cost-effectiveness of the construction.

7.22.3 Drilled-Shaft Design

Much of the design methodology for drilled shafts is similar to that applied to pile foundations and usually differs only in the manner in which the design parameters are characterized. Consequently, drilled-shaft design may be based on precedent (experience), load testing, or static analyses. The ultimate-load design approach is currently the most common form of static analysis applied, although load-deformation compatibility methods are being increasingly used (see Art. 7.18).

7.22.4 Skin Friction in Cohesive Soils

Ultimate skin friction for axially loaded shafts drilled into cohesive soils is usually evaluated by application of an empirically derived reduction (adhesion) factor to the undrained shear strength of the soil in contact with the shaft [see Eq. (7.37)]. For conventionally drilled shafts in stiff clays ($c_u \geq 0.50$ tons/ft^2), the adhesion factor α has been observed to range usually between 0.3 and 0.6.

Based on analysis of high-quality load-test results, primarily in stiff, fissured Beaumont and London clays, α factors of 0.5 and 0.45 have been recommended. Unlike the criteria applied to pile design, these factors are independent of c_u, but are largely dependent on construction methods and practices. Reese has recommended that the shaft length assumed to be effective in transferring load

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should be reduced by 5 ft to account for base interaction effects and that a similar reduction be applied to account for surface effects such as soil shrinkage.

Table 7.11 lists recommended α factors for straight shafts as a function of the normalized shear strength c_u/σ'_{vo} (Art. 7.5.1) and plasticity index I_p (Art. 7.4). These factors reflect conventional dry-hole construction methods and the influence of the stress history and plasticity of the soil in contact with the shaft. The α factors in Table 7.11 may be linearly interpreted for specific values of c_u/σ'_{vo} and I_p. To account for tip effects, the part of the shaft located 1 diameter above the base should be ignored in evaluation of Q_{su} (see Art. 7.17).

Where shafts are drilled to bearing on relatively incompressible materials, the amount of relative movement between the soil and the shaft may be insufficient to develop a substantial portion of the ultimate skin friction, particularly for short, very stiff shafts. Under these circumstances, Q_s should be ignored in design or analyzed with load-displacement compatibility procedures.

Because belled shafts usually require larger deformations than straight shafts to develop design loads, Q_{su} may be reduced for such shafts as a result of a progressive degradation at relative deformations greater than that required to develop peak values. Limited data on belled vs. straight shafts suggest a reduction in the α factor on the order of 15% to account for the reduced shaft friction of the belled shafts. It is also conservative to assume that there is no significant load transfer by friction in that portion of the shaft located about 1 diameter above the top of the bell.

There is some evidence that when shaft drilling is facilitated by the use of a weighted drilling fluid (mud slurry), there may be a substantial reduction in Q_{su}, presumably as a result of entrapment of the slurry between the soil and shaft concrete. Where this potential exists, it has been suggested that the α factor be reduced about 40%.

(A. W. Skempton, "Summation, Symposium on Large Bored Piles," Institute of Civil Engineers, London; L. C. Reese, F. T. Toma, and M. W. O'Neill, "Behavior of Drilled Piers under Axial Loading," *ASCE Journal of Geotechnical Engineering Division*, vol. 102, no. GT5, 1976; W. S. Gardner, "Investigation of the Effects of Skin Friction on the Performance of Drilled Shafts in Cohesive Soils," Report to U.S. Army Engineers Waterways Experiment Station (Contract no. DACA 39-80-C-0001), vol. 3, Vicksburg, Miss., 1981.)

Table 7.11 Adhesion Factors α for Drilled Shafts

Plasticity Index	Normalized Shear Strength c_u/σ'_{vo}*		
	0.3 or Less	1.0	2.5 or More
20	0.33	0.44	0.55
30	0.36	0.48	0.60
60	0.40	0.52	0.65

* Based on *UU* tests on good-quality samples selected so that c_u is not significantly influenced by the presence of fissures.

7.22.5 Skin Friction in Cohesionless Soils

Ultimate skin friction in cohesionless soils can be evaluated approximately with Eq. (7.40). In the absence of more definitive data, K in Eq. (7.40) may be taken as 0.6 for loose sands and 0.7 for medium dense to dense sands, on the assumption that the soil-shaft interface friction angle is taken as $\phi' - 5°$. As for piles, limited test data indicate that the average friction stress f_{su} is independent of overburden pressure for shafts drilled below a critical depth z_c of from 10 (loose sand) to 20 (dense sand) shaft diameters. The limiting skin friction f_i for shafts with ratio of length to diameter $L/D \leq 25$ should appreciably not exceed 1.0 ton/ft^2. Average f_{su} may be less than 1.0 ton/ft^2 for shafts longer than about 80 ft.

Equation (7.52) approximately represents a correlation between f_{su} and the average standard penetration test blow count $\bar{N}$ within the embedded pile length recommended for shafts in sand with effective $L/D \leq 10$. The stress f_{su} so computed, however, is less conservative than the foregoing design approach, particularly for $\bar{N} \geq 30$ blows per foot.

$$f_{su} = 0.03\bar{N} \leq 1.6\ \text{tons/ft}^2 \qquad (7.52)$$

7.22.6 End Bearing on Soils

End bearing of drilled shafts in cohesive soils is typically evaluated as described for driven piles [Eq. (7.42)]. The shear-strength term in this equation represents the average c_u within a zone of 2

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diameters below the shaft space. For smaller shafts, the suggested reduction factor is 0.8.

End bearing in cohesionless soils can be estimated in accordance with Eq. (7.43) with the same critical-depth limitations described for pile foundations. N_q for drilled shafts, however, has been observed to be significantly smaller than that applied to piles (see Fig. 7.19). Meyerhof has suggested that N_q should be reduced by 50%. Alternatively, q_u can be expressed in terms of the average SPT blow count $\bar{N}$ as:

$$q_u = 0.67\bar{N} \le 40 \text{ tons/ft}^2 \tag{7.53}$$

where q_u = ultimate base resistance at a settlement equivalent to 5% of the base diameter. (G. G. Meyerhof, "Bearing Capacity and Settlement of Pile Foundations," *ASCE Journal of Geotechnical Engineering Division*, vol. 102, no. GT3, 1976.)

7.22.7 Shaft Settlement

Drilled-shaft settlements can be estimated by load-deformation compatibility analyses (see Art. 7.18). Other methods used to estimate settlement of drilled shafts, singly or in groups, are identical to those used for piles (Art. 7.19.5). These include elastic, semiempirical elastic, and load-transfer solutions for single shafts drilled in cohesive or cohesionless soils. (H. G. Poulos and E. H. Davis, "Elastic Solutions for Soil and Rock Mechanics," John Wiley & Sons, Inc., New York; A. S. Vesic, "Principles of Pile Foundation Design," Soil Mechanics Series no. 38, Duke University, Durham, N.C., 1975; H. Y. Fang, "Foundation Engineering Handbook," 2nd ed., Van Nostrand Reinhold, New York.)

Resistance to tensile and lateral loads by straight-shaft drilled shafts should be evaluated as described for pile foundations (see Art. 7.19).

7.22.8 Rock-Supported Shafts

Drilled shafts may be designed to be supported on rock or to be socketed into rock. Except for long, relatively small-diameter (comparatively compressible) shafts, conventional design ignores the skin friction of belled or straight shafts founded on relatively incompressible materials. Where shafts are socketed in rock, the design capacity is considered a combination of the sidewall shearing resistance (bond) and the end bearing of the socket. In practice, both end-bearing and rock-socket designs are based on local experience, presumptive values in codes or semi empirical design methods. Latter methods based on field load test results are preferred for design efficiency.

Bearing values on rock given in design codes typically range from 50 to 100 tons/ft^2 for massive crystalline rock, 20 to 50 tons/ft^2 for sound foliated rock, 15 to 25 tons/ft^2 for sound sedimentary rock, 8 to 10 tons/ft^2 for soft and fractured rock, and 4 to 8 tons/ft^2 for soft shales.

The supporting ability of a specific rock type is primarily dependent on the frequency, orientation, and size of the discontinuities within the rock mass and the degree of weathering of the rock minerals. Consequently, application of presumptive bearing values is not recommended without specific local performance correlations. (R. W. Woodward, W. S. Gardner, and D. M. Greer, "Drilled Pier Foundations," McGraw-Hill Book Company, New York (books.mcgraw-hill.com).)

Some analyses relate the bearing values q_u in jointed rock to the uniaxial compressive (UC) strength of representative rock cores. These analyses indicate that q_u should not be significantly less than UC, possibly excluding weak sedimentary rocks such as compacted shales and siltstones. With a safety factor of 3, the maximum allowable bearing value q_a can be taken as: $q_a \le 0.3UC$. In most instances, however, the compressibility of the rock mass rather than rock strength governs. Elastic solutions can be used to evaluate the settlement of shafts bearing on rock if appropriate deformation moduli of the rock mass E_r can be determined. (H. G. Poulos and E. H. Davis, "Elastic Solutions for Soil and Rock Mechanics," John Wiley & Sons, Inc., New York (www.wiley.com); D. U. Deere, A. J. Hendron, F. D. Patton, and E. J. Cording, "Breakage of Rock," *Eighth Symposium on Rock Mechanics*, American Institute of Mining and Metallurgical Engineers, Minneapolis, Minn., 1967; F. H. Kulhawy, "Geotechnical Model for Rock Foundation Settlement," *ASCE Journal of Geotechnical Engineering Division*, vol. 104, no. GT2, 1978.)

Concrete-rock bond stresses f_R used for the design of rock sockets have been empirically established from a limited number of load tests. Typical values range from 70 to 200 psi, increasing with rock quality. For good-quality rock, f_R may be related to the 28-day concrete strength f'_c and to the uniaxial compressive (UC) strength of rock cores. For rock with $RQD \ge 50\%$ (Table 7.3), f_R can be

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estimated as $0.05f_c'$ or $0.05UC$ strength, whichever is smaller, except that f_R should not exceed 250 psi. As shown by Fig. 7.31, the ultimate rock-concrete bond f_{Ru} is significantly higher than f_R except for very high UC values. (P. Rosenberg and N. L. Journeaux, "Friction and End-Bearing Tests on Bedrock for High-Capacity Socket Design," *Canadian Geotechnical Journal*, vol. 13, no. 3, 1976.)

Design of rock sockets is conventionally based on

$$Q_d = \pi d_s L_s f_R + \frac{\pi}{4} d_s^2 q_a \qquad (7.54)$$

where Q_d = allowable design load on rock socket

d_s = socket diameter

L_s = socket length

f_R = allowable concrete-rock bond stress

q_a = allowable bearing pressure on rock

Load-distribution measurements show, however, that much less of the load goes to the base than is indicated by Eq. (7.54). This behavior is demonstrated by the data in Table 7.12, where L_s/d_s is the ratio of the shaft length to shaft diameter and E_r/E_p is the ratio of rock modulus to shaft modulus. The finite-element solution summarized in Table 7.12 probably reflects a realistic trend if the average socket-wall shearing resistance does not exceed the ultimate f_R value; that is, slip along the socket sidewall does not occur.

A simplified design approach, taking into account approximately the compatibility of the socket and base resistance, is applied as follows:

1. Proportion the rock socket for design load Q_d with Eq. (7.54) on the assumption that the end-bearing stress is less than q_a [say $q_a/4$, which is equivalent to assuming that the base load $Q_b = (\pi/4)d_s^2 q_a/4$].
2. Calculate $Q_b = RQ_d$, where R is the base-load ratio interpreted from Table 7.12.
3. If RQ_d does not equal the assumed Q_b, repeat the procedure with a new q_a value until an approximate convergence is achieved and $q \leq q_a$.

Table 7.12 Percent of Base Load Transmitted to Rock Socket

	E_r/E_p		
L_s/d_s	0.25	1.0	4.0
0.5	54*	48	44
1.0	31	23	18
1.5	17*	12	8*
2.0	13*	8	4

* Estimated by interpretation of finite-element solution; for Poisson's ratio = 0.26.

The final design should be checked against the established settlement tolerance of the drilled shaft. (B. Ladanyi, discussion of "Friction and End-Bearing Tests on Bedrock," *Canadian Geotechnical Journal*, vol. 14, no. 1, 1977; H. G. Poulos and E. H. Davis, "Elastic Solutions for Rock and Soil Mechanics," John Wiley & Sons, Inc., New York (www.wiley.com).)

Following the recommendations of Rosenberg and Journeaux, a more realistic solution by the above method is obtained if f_{Ru} is substituted for f_R. Ideally, f_{Ru} should be determined from load tests. If this parameter is selected from Fig. 7.32 or from other data that are not site-specific, a safety factor of at least 1.5 should be applied to f_{Ru} in recognition of the uncertainties associated with the UC strength correlations. (P. Rosenberg and N. L. Journeaux, "Friction and End-Bearing Tests on Bedrock for High-Capacity Socket Design," *Canadian Geotechnical Journal*, vol. 13, no. 3, 1976.)

7.22.9 Testing of Drilled Shafts

Static-load capacity of drilled shafts may be verified by either static-load or dynamic-load testing (Arts. 7.19 and 7.20). Testing by applying static loads on the shaft head (conventional static-load test) or against the toe (Osterberg cell) provides information on shaft capacity and general behavior. Dynamic-load testing in which pile-head force and velocity under the impact of a falling weight are measured with a Pile Driving Analyzer and subsequent analysis with the CAPWAP method (Art. 7.20.3) provide information on the static-load capacity and shaft-movement and shaft-soil load-transfer relationships of the shaft.

Structural integrity of a drilled shaft may be assessed after excavation or coring through the shaft. Low-strain dynamic-load testing with a Pile Integrity Tester (Art. 7.20.4), offers many advantages, however. Alternative integrity evaluation methods are parallel seismic or cross-hole sonic logging.

For parallel seismic testing, a small casing is inserted into the ground near the tested shaft and

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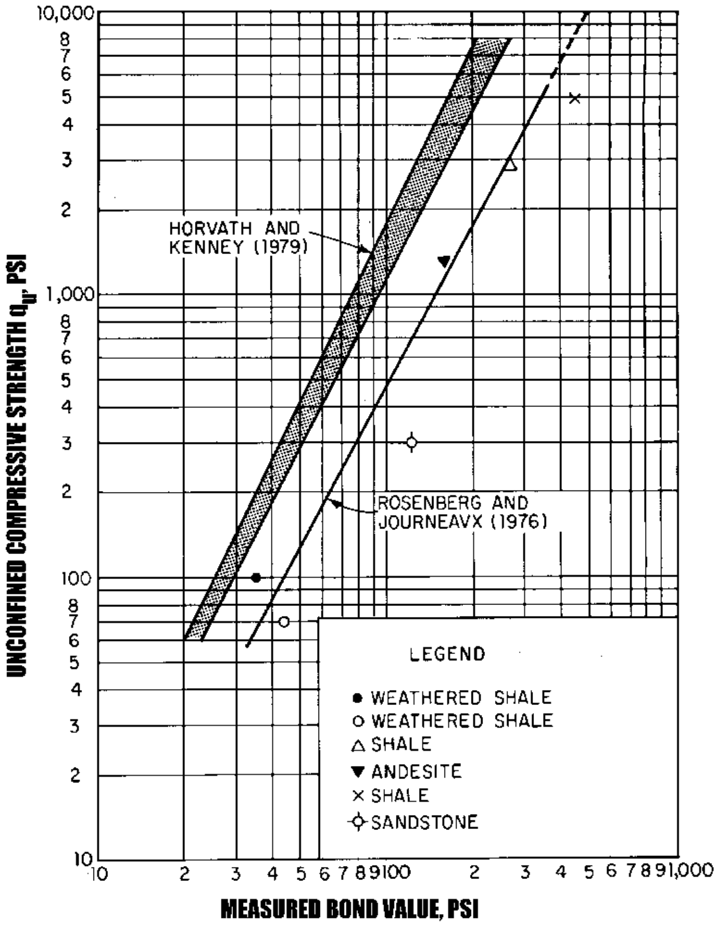

Fig. 7.32 Chart relates the bond of a rock socket to the unconfined compressive strength of cores.

to a greater depth than the shaft length. A hydrophone is lowered into the casing to pick up the signals resulting from blows on the shaft head from a small hand-held hammer. Inasmuch as wave velocity in the soil and shaft are different, the unknown length of the pile can be discerned from a series of measurements. One limitation of this method is the need to bore a hole adjacent to the shaft to be tested.

Cross-hole testing requires two full-length longitudinal access tubes in the shaft. A transmitter is lowered through one of the tubes to send a signal to a receiver lowered into the other tube. The arrival time and magnitude of the received signal are interpreted to assess the integrity of the shaft between the two tubes. For large-diameter shafts, more than two tubes may be needed for thorough shaft evaluation. A disadvantage of this method is the need to form two or more access tubes in the shaft during construction. Furthermore, random testing or evaluations of existing shafts may not be possible with this method.

(C. L. Crowther, "Load Testing of Deep Foundations," John Wiley & Sons, Inc., New York (www.wiley.com); "New Failure Load Criterion for Large Diameter Bored Piles in Weathered Geomaterials," by Charles W. W. Ng, et al., *ASCE Journal of Geotechnical and Geoenvironmental Engineering,* vol. 127, no. 6, June 2001.)

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Retaining Methods For Excavation

The simplest method of retaining the sides of an excavation in soil is to permit the soil to form a natural slope that will be stable even in the presence of water. When there is insufficient space for this inside the excavation or when the excavation sides must be vertical, construction such as that described in the following must be used.

7.23 Caissons

Load-bearing enclosures known as caissons are formed in the ground, usually to protect excavation for a foundation, aid construction of the substructure, and serve as part of the permanent structure. Sometimes, a caisson is used to enclose a subsurface space to be used for such purposes as a pump well, machinery pit, or access to a deeper shaft or tunnel. Several caissons may be aligned to form a bridge pier, bulkhead, seawall, foundation wall for a building, or impervious core wall for an earth dam.

For foundations, caissons are used to facilitate construction of shafts or piers extending from near the surface of land or water to a bearing stratum. This type of construction can carry heavy loads to great depths. Built of common structural materials, they may have any shape in cross section. They range in size from about that of a pile to over 100 ft in length and width. Some small ones are considered drilled shafts. Previously described construction methods for drilled shafts are employed based on subsurface conditions and drilled shaft depth and diameter.

Caissons often are installed by sinking them under their own weight or with a surcharge. The operation is assisted by jacking, jetting, excavating, and undercutting. Care must be taken during this operation to maintain alignment. The caissons may be built up as they sink, to permit construction to be carried out at the surface, or they may be completely prefabricated. Types of caissons used for foundation work are as follows:

Chicago caissons are used for constructing foundation shafts through a thick layer of clay to hardpan or rock. The method is useful where the soil is sufficiently stiff to permit excavation for short distances without caving. A circular pit about 5 ft deep is dug and lined with wood staves. This vertical lagging is braced with two rings made with steel channels. Then, 5 ft of soil is removed, and the operation is repeated. If the ground is poor, shorter lengths are dug until the bearing stratum is reached. If necessary, the caissons can be belled at the bottom to carry large loads. Finally, the hole is filled with concrete. Minimum economical diameter for hand digging is 4 ft.

Sheeted piers or caissons are similarly constructed, but the vertical lagging of wood or steel is driven down during or before excavation. This system usually is used for shallow depths in wet ground.

In dry ground, horizontal wood sheeting may be used. This is economical and necessary where there is inadequate vertical clearance. Louvered construction should be used to provide drainage and to permit packing behind the wood sheeting where soil will not maintain a vertical face long enough to permit insertion of the next sheet. This type of construction requires over-excavating so that the wood sheets can be placed. Openings must be wide enough between sheets to allow backfilling and tamping, to correct the excavation irregularities and equalize pressure on all sides. Small blocks may be inserted between successive sheets to leave packing gaps. If the excavation is large, **soldier beams**, vertical cantilevers, can be driven to break up the long sheeting spans.

Benoto caissons up to 39 in in diameter may be sunk through water-bearing sands, hardpan, and boulders to depths of 150 ft. Excavation is done with a hammer grab, a single-line orange-peel bucket, inside a temporary, cylindrical steel casing. The hammer grab is dropped to cut into or break up the soil. After impact, the blades close around the soil. Then, the bucket is lifted out and discharged. Boulders are broken up with heavy, percussion-type drills. Rock is drilled out by churn drills. To line the excavation, a casing is bolted together in 20-ft-deep sections, starting with a cutting edge. A hydraulic attachment oscillates the casing continuously to ease sinking and withdrawal, while jacks force the casing into the ground. As concrete is placed, the jacks withdraw the casing in a way that allows concreting of the caisson. Benoto caissons are slower to place and more expensive than drilled shafts, except in wet granular material and where soil conditions are too tough for augers or rotating-bucket diggers.

Open caissons (Fig. 7.33) are enclosures without top and bottom during the lowering process. When

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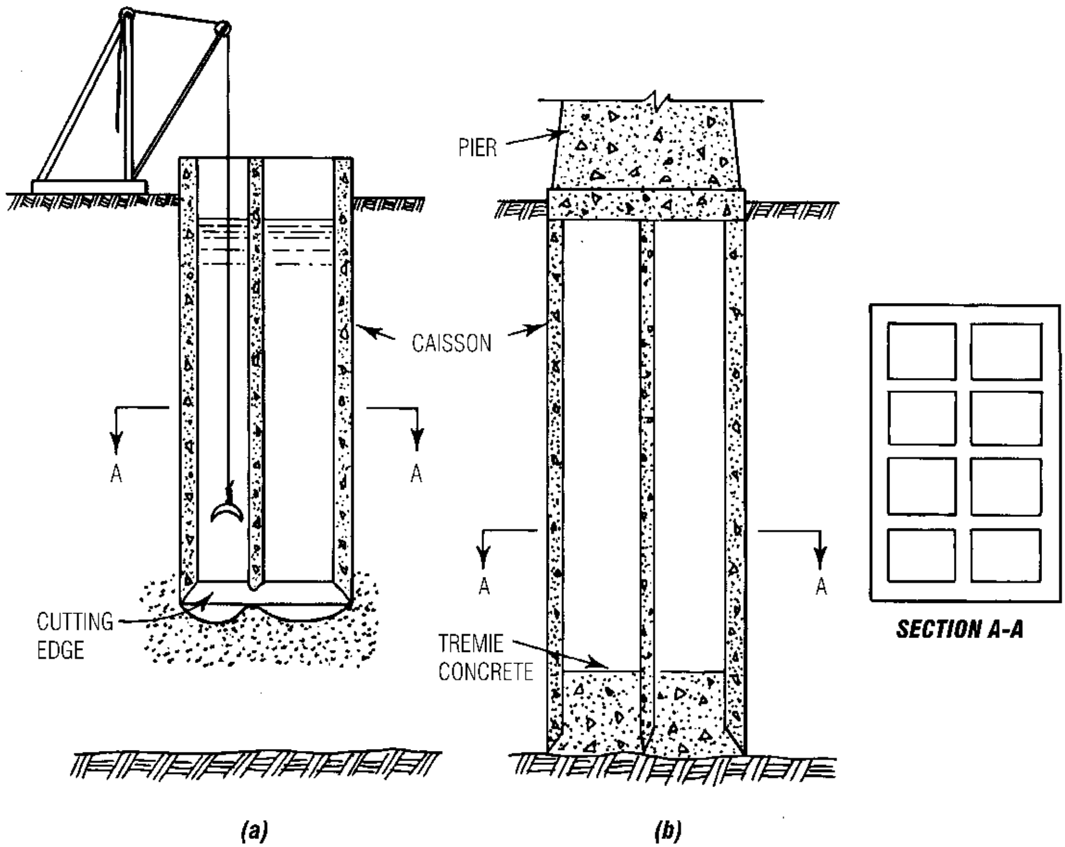

Fig. 7.33 Construction with an open concrete caisson.

used for pump wells and shafts, they often are cylindrical. For bridge piers, these caissons usually are rectangular and compartmented. The compartments serve as dredging wells, pipe passages, and access shafts. Dredging wells usually have 12- to 16-ft clear openings to facilitate excavation with clamshell or orange-peel buckets.

An open caisson may be a braced steel shell that is filled with concrete, except for the wells, as it is sunk into place. Or a caisson may be constructed entirely of concrete.

Friction along the caisson sides may range from 300 to over 1000 lb/ft^2. So despite steel cutting edges at the wall bottoms, the caisson may not sink. Water and compressed-air jets may be used to lubricate the soil to decrease the friction. For that purpose, vertical jetting pipes should be embedded in the outer walls.

If the caisson does not sink under its own weight with the aid of jets when soil within has been removed down to the cutting edge, the caisson must be weighted. One way is to build it higher, to its final height, if necessary. Otherwise, a platform may have to be built on top and weights piled on it, a measure that can be expensive.

Care must be taken to undercut the edges evenly, or the caisson will tip. Obstructions and variations in the soil also can cause uneven sinking.

When the caisson reaches the bearing strata, the bottom is plugged with concrete (Fig. 7.41*b*). The plug may be placed by tremie or made by injecting grout into the voids of coarse aggregate.

When a caisson must be placed through water, marine work sometimes may be converted to a land job by construction of a sand island. Fill is placed until it projects above the water surface.

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Then, the caisson is constructed and sunk as usual on land.

Pneumatic caissons contain at the base a working chamber with compressed air at a pressure equal to the hydrostatic pressure of the water in the soil. Without the balancing pressure, the water would force soil from below up into a caisson. A working chamber clear of water also permits hand work to remove obstructions that buckets, air lifts, jets, and divers cannot. Thus, the downward course of the caisson can be better controlled. But sinking may be slower and more expensive, and compressed-air work requires precautions against safety and health hazards.

Access to the working chamber for workers, materials, and equipment is through air locks, usually placed at the top of the caisson (Fig. 7.34). Steel access cylinders 3 ft in diameter connect the air locks with the working chamber in large caissons.

Entrance to the working chamber requires only a short stay for a worker in an air lock. But the return stop may be lengthy, depending on the pressure in the chamber, to avoid the bends, or caisson disease, which is caused by air bubbles in muscles, joints, and the blood. Slow decompression gives the body time to eliminate the excess air. In addition to slow decompression, it is necessary to restrict the hours worked at various pressures and limit the maximum pressure to 50 psi above atmospheric or less. The restriction on pressure limits the maximum depth at which compressed-air work can be done to about 115 ft. A medical, or recompression, lock is also required on the site for treatment of workers attacked by the bends.

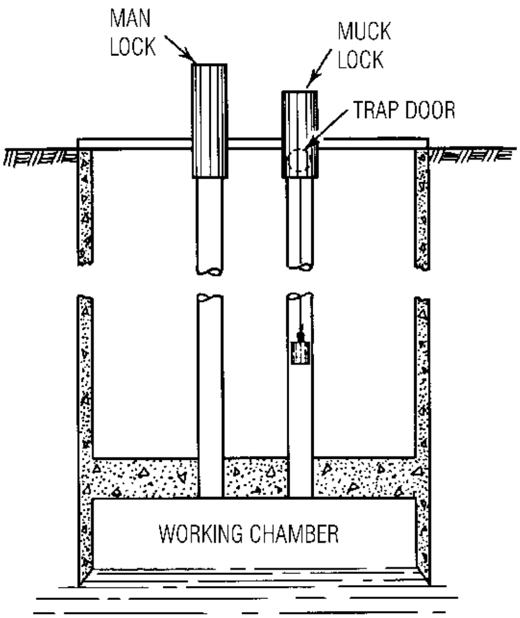

Fig. 7.34 Pneumatic caisson. Pressure in working chamber is above atmospheric.

Floating caissons are used when it is desirable to fabricate caissons on land, tow them into position, and sink them through water. They are constructed much like open or pneumatic caissons but with a "false" bottom, "false" top, or buoyant cells. When floated into position, a caisson must be kept in alignment as it is lowered. A number of means may be used for the purpose, including anchors, templates supported on temporary piles, anchored barges, and cofferdams. Sinking generally is accomplished by adding concrete to the walls. When the cutting edges reach the bottom, the temporary bulkheads at the base, or false bottoms, are removed since buoyancy no longer is necessary. With false tops, buoyancy is controlled with compressed air, which can be released when the caisson sits on the bottom. With buoyant cells, buoyancy is gradually lost as the cells are filled with concrete.

Closed-box caissons are similar to floating caissons, except the top and bottom are permanent. Constructed on land, of steel or reinforced concrete, they are towed into position. Sometimes, the site can be dredged in advance to expose soil that can safely support the caisson and loads that will be imposed on it. Where loads are heavy, however, this may not be practicable; then, the box caisson may have to be supported on piles, but allowance can be made for its buoyancy. This type of caisson has been used for breakwaters, seawalls, and bridge-pier foundations.

Potomac caissons have been used in wide tidal rivers with deep water underlain by deep, soft deposits of sand and silt. Large timber mats are placed on the river bottom, to serve as a template for piles and to retain tremie concrete. Long, steel pipe or H piles are driven in clusters, vertical and battered, as required. Prefabricated steel or concrete caissons are set on the mat over the pile clusters, to serve as permanent forms for concrete shafts to be supported on the piles. Then, concrete is tremied into the caissons. Since the caissons are

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used only as forms, construction need not be so heavy as for conventional construction, where they must withstand launching and sinking stresses, and cutting edges are not required.

(H. Y. Fang, "Foundation Engineering Handbook," 2nd ed., Van Nostrand Reinhold Company, New York.)

7.24 Dikes and Cribs

Earth dikes, when fill is available, are likely to be the least expensive for keeping water out of an excavation. If impervious material is not easily obtained, however, a steel sheetpile cutoff wall may have to be driven along the dike, to permit pumps to handle the leakage. With an impervious core in the dike, wellpoints, deep-well pumps, or sumps and ditches may be able to keep the excavation unwatered.

Timber cribs are relatively inexpensive excavation enclosures. Built on shore, they can be floated to the site and sunk by filling with rock. The water side may be faced with wood boards for watertightness (Fig. 7.35). For greater watertightness, two lines of cribs may be used to support two lines of wood sheeting between which clay is tamped to form a "puddle" wall. Design of timber cribs should provide ample safety against overturning and sliding.

7.25 Cofferdams

Temporary walls or enclosures for protecting an excavation are called cofferdams. Generally, one of the most important functions is to permit work to be carried out on a nearly dry site.

Cofferdams should be planned so that they can be easily dismantled for reuse. Since they are temporary, safety factors can be small, 1.25 to 1.5, when all probable loads are accounted for in the design. But design stresses should be kept low when stresses, unit pressure, and bracing reactions are uncertain. Design should allow for construction loads and the possibility of damage from construction equipment. For cofferdams in water, the design should provide for dynamic effect of flowing water and impact of waves. The height of the cofferdam should be adequate to keep out floods that occur frequently.

7.25.1 Double-Wall Cofferdams

These may be erected in water to enclose large areas. Double-wall cofferdams consist of two lines of sheetpiles tied to each other; the space between

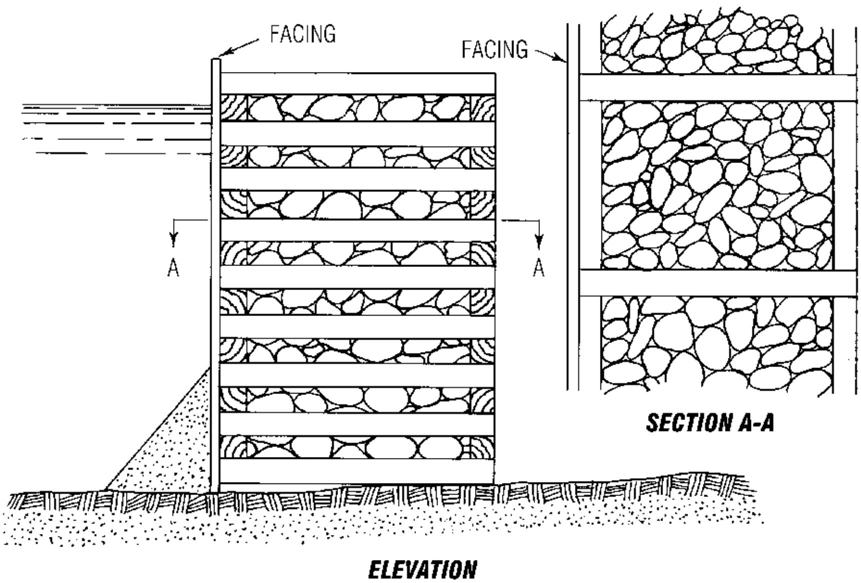

Fig. 7.35 Timber crib with stone filling.

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is filled with sand (Fig. 7.36). For sheetpiles driven to irregular rock, or gravel, or onto boulders, the bottom of the space between walls may be plugged with a thick layer of tremie concrete to seal gaps below the tips of the sheeting. Double-wall cofferdams are likely to be more watertight than single-wall ones and can be used to greater depths.

A berm may be placed against the outside face of a cofferdam for stability. If so, it should be protected against erosion. For this purpose, riprap, woven mattresses, streamline fins or jetties, or groins may be used. If the cofferdam rests on rock, a berm needs to be placed on the inside only if required to resist sliding, overturning, or shearing. On sand, an ample berm must be provided so that water has a long path to travel to enter the cofferdam (Fig. 7.36). (The amount of percolation is proportional to the length of path and the head.) Otherwise, the inside face of the cofferdam may settle, and the cofferdam may overturn as water percolates under the cofferdam and causes a quick, or boiling, excavation bottom. An alternative to a wide berm is wider spacing of the cofferdam walls. This is more expensive but has the added advantage that the top of the fill can be used by construction equipment and for construction plant.

7.25.2 Cellular Cofferdams

Used in construction of dams, locks, wharves, and bridge piers, cellular cofferdams are suitable for enclosing large areas in deep water. These enclosures are composed of relatively wide units. Average width of a cellular cofferdam on rock should be 0.70 to 0.85 times the head of water against the outside. When constructed on sand, a cellular cofferdam should have an ample berm on the inside to prevent the excavation bottom from becoming quick (Fig. 7.37*d*).

Steel sheetpiles interlocked form the cells. One type of cell consists of circular arcs connected by straight diaphragms (Fig. 7.37*a*). Another type comprises circular cells connected by circular arcs (Fig. 7.37*b*). Still another type is the cloverleaf, composed of large circular cells subdivided by straight diaphragms (Fig. 7.37*c*). The cells are filled with sand. The internal shearing resistance of the sand contributes substantially to the strength of the cofferdam. For this reason, it is unwise to fill a cofferdam with clay or silt. Weepholes on the inside sheetpiles drain the fill, thus relieving the hydrostatic pressure on those sheets and increasing the shear strength of the fill.

In circular cells, lateral pressure of the fill causes only ring tension in the sheetpiles. Maximum stress in the pile interlocks usually is limited to 8000 lb/lin in. This in turn limits the maximum diameter of the circular cells. Because of numerous uncertainties, this maximum generally is set at 60 ft. When larger-size cells are needed, the cloverleaf type may be used.

Circular cells are preferred to the diaphragm type because each circular cell is a self-supporting unit. It may be filled completely to the top before construction of the next cell starts. (Unbalanced fills in a cell may distort straight diaphragms.)

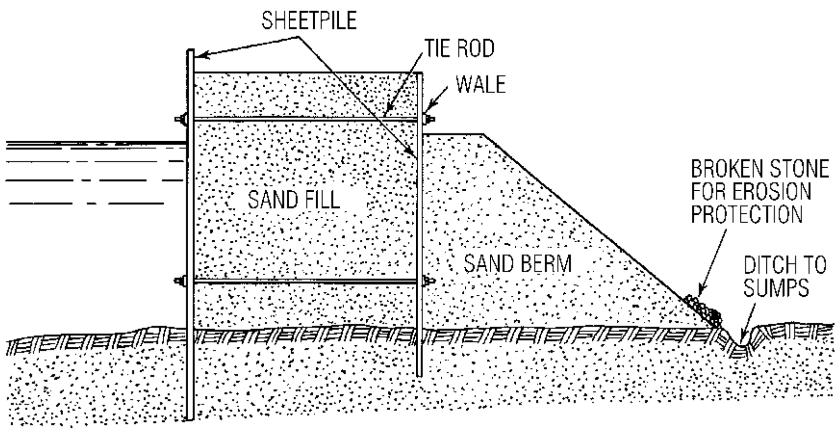

Fig. 7.36 Double-wall cofferdam.

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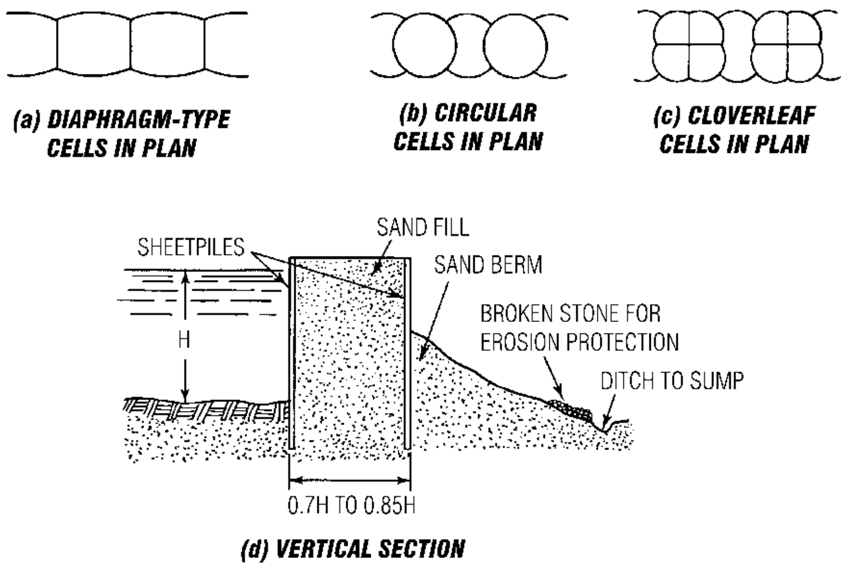

Fig. 7.37 Cellular sheetpile cofferdam.

When a circular cell has been filled, the top may be used as a platform for construction of the next cell. Also, circular cells require less steel per linear foot of cofferdam. The diaphragm type, however, may be made as wide as desired.

When the sheetpiles are being driven, care must be taken to avoid breaking the interlocks. The sheetpiles should be accurately set and plumbed against a structurally sound template. They should be driven in short increments, so that when uneven bedrock or boulders are encountered, driving can be stopped before the cells or interlocks are damaged. Also, all the piles in a cell should be started until the cell is ringed. This can reduce jamming troubles with the last piles to be installed for the cell.

7.25.3 Single-Wall Cofferdams

These form an enclosure with only one line of sheeting. If there will be no water pressure on the sheeting, they may be built with **soldier beams** (piles extended to the top of the enclosure) and horizontal wood lagging (Fig. 7.38). If there will be water pressure, the cofferdam may be constructed of sheetpiles. Although they require less wall material than double-wall or cellular cofferdams, single-wall cofferdams generally require bracing on the inside. Also, unless the bottom is driven into a thick, impervious layer, they may leak excessively at the bottom. There may also be leakage at interlocks. Furthermore, there is danger of flooding and collapse due to hydrostatic forces when these cofferdams are unwatered.

For marine applications, therefore, it is advantageous to excavate, drive piles, and place a seal of tremie concrete without unwatering single-wall sheetpile cofferdams. Often, it is advisable to predredge the area before the cofferdam is constructed, to facilitate placing of bracing and to remove obstructions to pile driving. Also, if blasting is necessary, it would severely stress the sheeting and bracing if done after they were installed.

For buildings, single-wall cofferdams must be carefully installed. Small movements and consequent loss of ground usually must be prevented to avoid damaging neighboring structures, streets, and utilities. Therefore, the cofferdams must be amply braced. Sheeting close to an existing structure should not be a substitute for underpinning.

Bracing ▪ Cantilevered sheetpiles may be used for shallow single-wall cofferdams in water or on land where small lateral movement will not be troublesome. Embedment of the piles in the bottom must be deep enough to insure stability. Design usually is based on the assumptions that lateral passive resistance varies linearly with depth and the point of inflection is about two-thirds the

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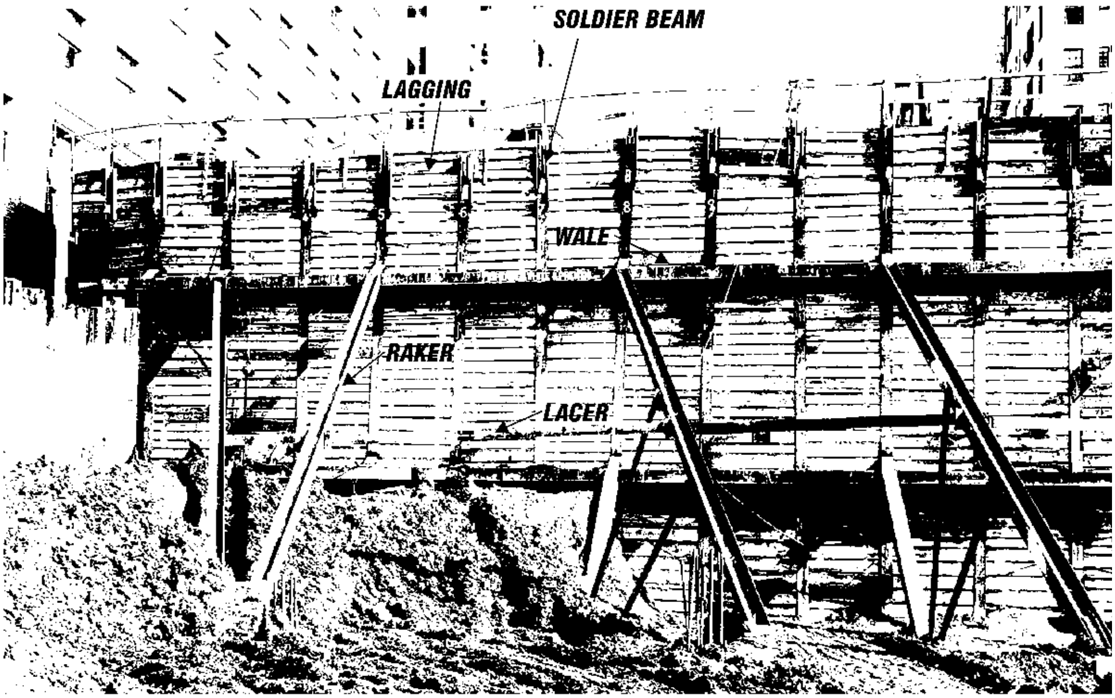

Fig. 7.38 Soldier beams and wood lagging retain the sides of an excavation.

embedded length below the surface. In general, however, cofferdams require bracing.

Cofferdams may be braced in many ways. Figure 7.39 shows some commonly used methods. Circular cofferdams may be braced with horizontal rings (Fig. 7.39*a*). For small rectangular cofferdams, horizontal braces, or wales, along sidewalls and end walls may be connected to serve only as struts. For larger cofferdams, diagonal bracing (Fig. 7.39*b*) or cross-lot bracing (Fig. 7.39*d* and *e*) is necessary. When space is available at the top of an excavation, pile tops can be anchored with concrete dead men (Fig. 7.39*c*). Where rock is close, the wall can be tied back with tensioned wires or bars that are anchored in grouted sockets in the rock (Fig. 7.40). See also Art. 7.40.4.

Horizontal cross braces should be spaced to minimize interference with excavation, form construction, concreting, and pile driving. Spacing of 12 and 18 ft is common. Piles and wales selected should be strong enough as beams to permit such spacing. In marine applications, divers often have to install the wales and braces underwater. To reduce the amount of such work, tiers of bracing may be prefabricated and lowered into the cofferdam from falsework or from the top set of wales and braces, which is installed above the water surface. In some cases, it may be advantageous to prefabricate and erect the whole cage of bracing before the sheetpiles are driven. Then, the cage, supported on piles, can serve also as a template for driving the sheetpiles.

All wales and braces should be forced into bearing with the sheeting by wedges and jacks.

When pumping cannot control leakage into a cofferdam, excavation may have to be carried out in compressed air. This requires a sealed working chamber, access shafts, and air locks, as for pneumatic caissons (Art. 7.23). Other techniques, such as use of a tremie concrete seal or chemical solidification or freezing of the soil, if practicable, however, will be more economical.

Braced sheetpiles may be designed as continuous beams subjected to uniform loading for earth and to loading varying linearly with depth for water (Art. 7.27). (Actually, earth pressure depends on the flexibility of the sheeting and relative stiffness of supports.) Wales may be designed for

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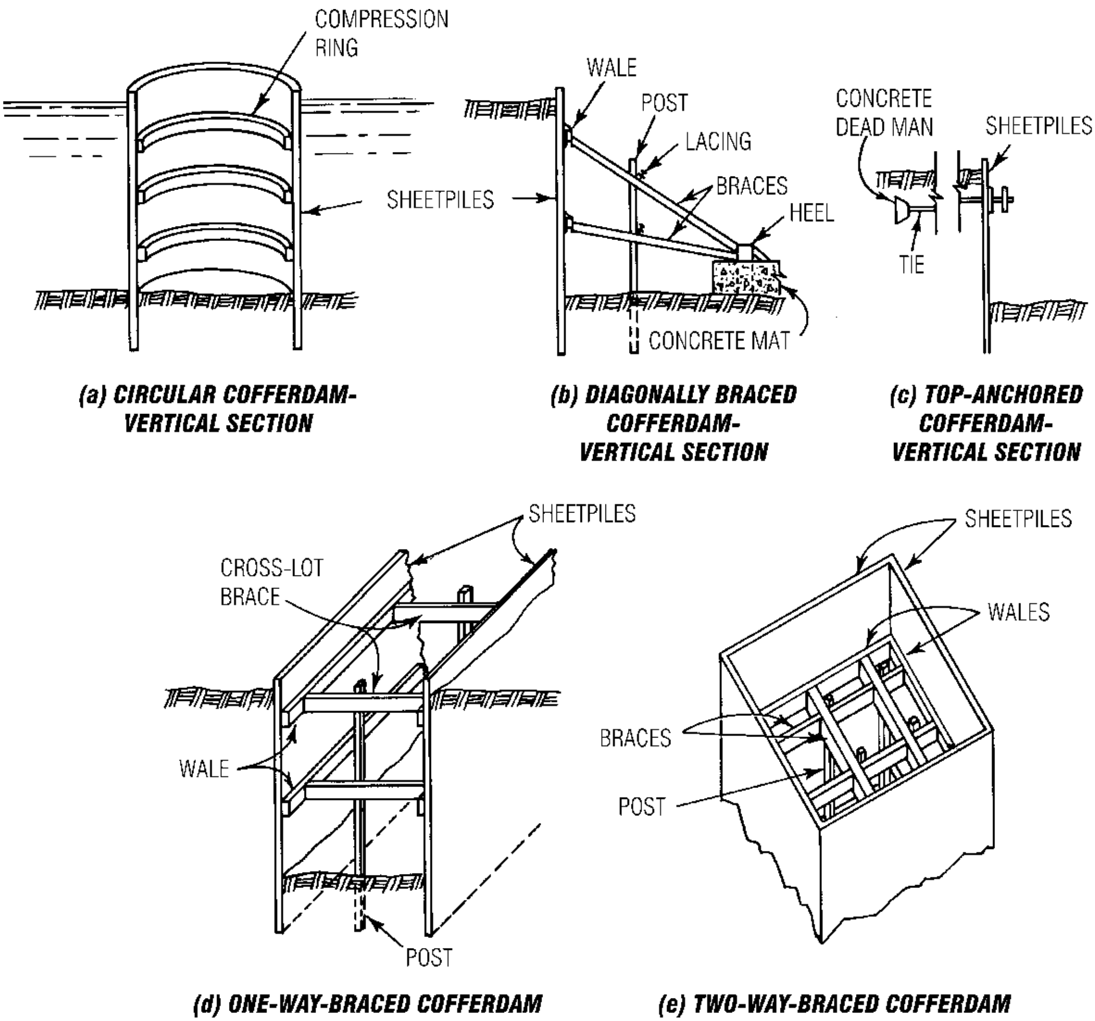

Fig. 7.39 Types of cofferdam bracing include compression rings; bracing, diagonal (rakers) or cross lot; wales, and tiebacks.

uniform loading. Allowable unit stresses in the wales, struts, and ties may be taken at half the elastic limit for the materials because the construction is temporary and the members are exposed to view. Distress in a member can easily be detected and remedial steps taken quickly.

Soldier beams and horizontal wood sheeting are a variation of single-wall cofferdams often used where impermeability is not required. The soldier beams, or piles, are driven vertically into the ground to below the bottom of the proposed excavation. Spacing usually ranges from 5 to 10 ft (Table 7.13). (The wood lagging can be used in the thicknesses shown in Table 7.13 because of arching of the earth between successive soldier beams.)

As excavation proceeds, the wood boards are placed horizontally between the soldiers (Fig. 7.38). Louvers or packing spaces, 1 to 2 in high, are left between the boards so that earth can be tamped behind them to hold them in place. Hay may also be stuffed behind the boards to keep the ground from running through the gaps. The louvers permit

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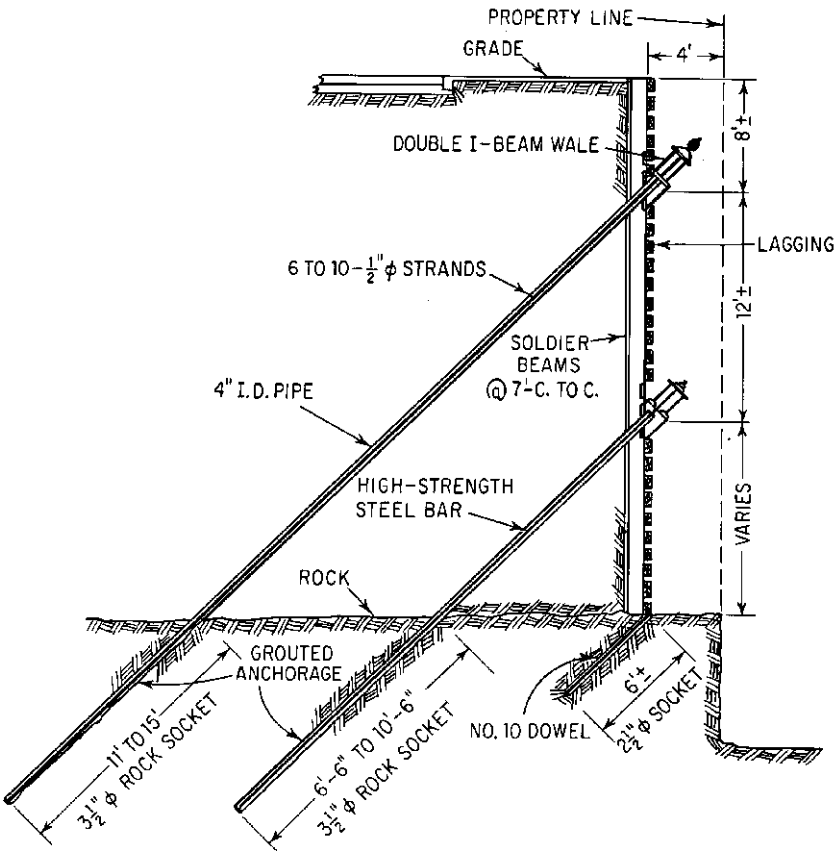

Fig. 7.40 Vertical section shows prestressed tiebacks for soldier beams.

drainage of water, to relieve hydrostatic pressure on the sheeting and thus allow use of a lighter bracing system. The soldiers may be braced directly with horizontal or inclined struts; or wales and braces may be used.

Advantages of soldier-beam construction include fewer piles; the sheeting does not have to extend below the excavation bottom, as do sheetpiles; and the soldiers can be driven more easily in hard ground than can sheetpiles. Varying the spacing of the soldiers permits avoidance of underground utilities. Use of heavy sections for the piles allows wide spacing of wales and braces. But the soldiers and lagging, as well as sheetpiles, are no substitute for underpinning; it is necessary to support and underpin even light adjoining structures.

Table 7.13 Usual Maximum Spans of Horizontal Sheeting with Soldier Piles, ft

Nominal Thickness of Sheeting, in	In Well-Drained Soils	In Cohesive Soils with Low Shear Resistance
2	5	4.5
3	8.5	6
4	10	8

Liner-plate cofferdams may be used for excavating circular shafts. The plates are placed in horizontal rings as excavation proceeds. Stamped from steel plate, usually about 16 in high and 3 ft long, light enough to be carried by one person, liner plates have inward-turned flanges along all edges. Top and bottom flanges provide a seat for successive rings. End flanges permit easy bolting

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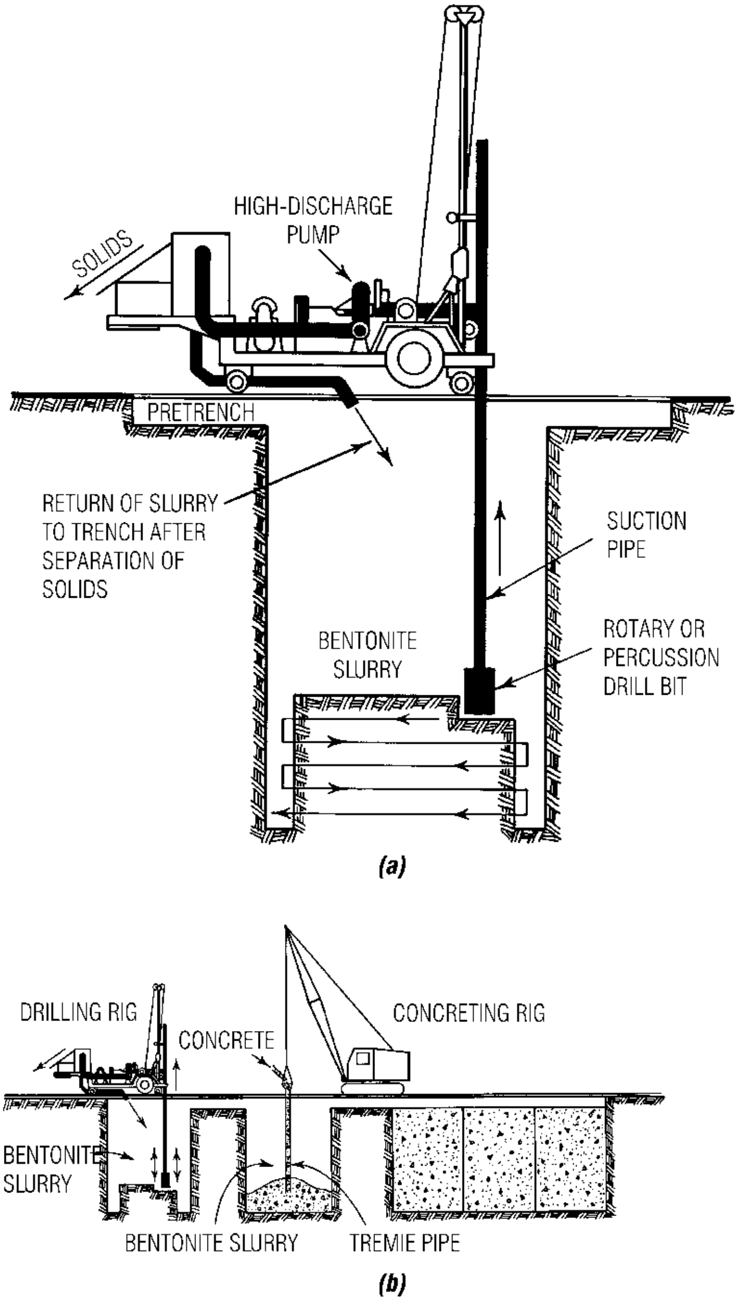

Fig. 7.41 Slurry-trench method for constructing a continuous concrete wall: (*a*) Excavating one section; (*b*) concreting one section while another is being excavated.

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of adjoining plates in a ring. The plates also are corrugated for added stiffness. Large-diameter cofferdams may be constructed by bracing the liner plates with steel beam rings.

Vertical-lagging cofferdams, with horizontal-ring bracing, also may be used for excavating circular shafts. The method is similar to that used for Chicago caissons (Art. 7.23). It is similarly restricted to soils that can stand without support in depths of 3 to 5 ft for a short time.

Slurry trenches may be used for constructing concrete walls. The method permits building a wall in a trench without the earth sides collapsing. While excavation proceeds for a 24- to 36-in-wide trench, the hole is filled with a bentonite slurry with a specific gravity of 1.05 to 1.10 (Fig. 7.41*a*). The fluid pressure against the sides and caking of bentonite on the sides prevent the earth walls of the trench from collapsing. Excavation is carried out a section at a time. A section may be 20 ft long and as much as 100 ft deep. When the bottom of the wall is reached in a section, reinforcing is placed in that section. (Tests have shown that the bond of the reinforcing to concrete is not materially reduced by the bentonite.) Then, concrete is tremied into the trench, replacing the slurry, which may flow into the next section to be excavated or be pumped into tanks for reuse in the next section (Fig. 7.41*b*). The method has been used to construct cutoffs for dams, cofferdams, foundations, walls of buildings, and shafts.

7.26 Soil Solidification

Grouting is the injection of cement or chemicals into soil or rock to enhance engineering properties. During the past 20 years significant developments in materials and equipment have transformed grouting from art to science. See Federal Highway Administration publication "Ground Improvement Technical Summaries" Publication No. FHWA-SA-98-086 December 1999 for an extensive primer on grouting techniques, applications and procedures.

Freezing is another means of solidifying water-bearing soils where obstructions or depth preclude pile driving. It can be used for deep shaft excavations and requires little material for temporary construction; the refrigeration plant has high salvage value. But freezing the soil may take a very long time. Also, holes have to be drilled below the bottom of the proposed excavation for insertion of refrigeration pipes.

(L. White and E. A. Prentis, "Cofferdams," Columbia University Press, New York; H. Y. Fang, "Foundation Engineering Handbook," 2nd ed., Van Nostrand Reinhold Company, New York.)

7.27 Lateral Active Pressures on Retaining Walls

Water exerts against a vertical surface a horizontal pressure equal to the vertical pressure. At any level, the vertical pressure equals the weight of a 1-ft^2 column of water above that level. Hence, the horizontal pressure p, lb/ft^3 at any level is

$$p = wh \qquad (7.55)$$

where w = unit weight of water, lb/ft^3

h = depth of water, ft

The pressure diagram is triangular (Fig. 7.42). Equation (7.55) also can be written

$$p = Kwh \qquad (7.56)$$

where K = pressure coefficient = 1.00.

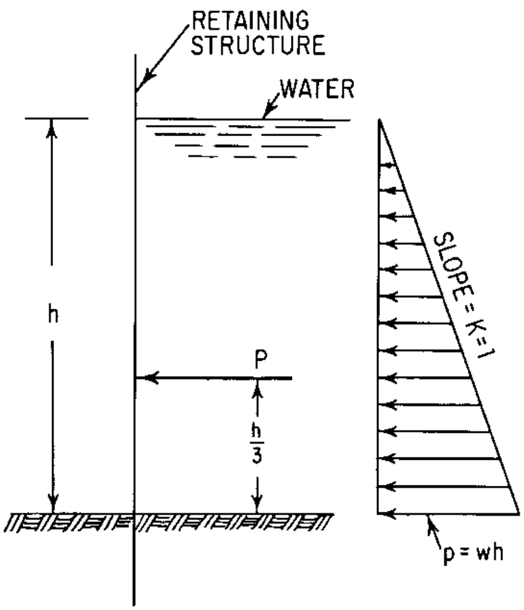

Fig. 7.42 Pressure diagram for water.

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The resultant, or total, pressure, lb/lin ft, represented by the area of the hydrostatic-pressure diagram, is

$$P = K\frac{wh^2}{2} \tag{7.57}$$

It acts at a distance $h/3$ above the base of the triangle.

Soil also exerts lateral pressure. But the amount of this pressure depends on the type of soil, its compaction or consistency, and its degree of saturation, and on the resistance of the structure to the pressure. Also, the magnitude of passive pressure differs from that of active pressure.

Active pressure tends to move a structure in the direction in which the pressure acts. **Passive pressure** opposes motion of a structure.

Retaining walls backfilled with cohesionless soils (sands and gravel) tend to rotate slightly around the base. Behind such a wall, a wedge of sand *ABC* (Fig. 7.43*a*) tends to shear along plane *AC*. C. A. Coulomb determined that the ratio of sliding resistance to sliding force is a minimum when AC makes an angle of $45° + \phi/2$ with the horizontal, where ϕ is the angle of internal friction of the soil, deg.

For triangular pressure distribution (Fig. 7.43*b*), the active lateral pressure of a cohesionless soil at a depth *h*, ft, is

$$p = K_a wh \tag{7.58}$$

where K_a = coefficient of active earth pressure
w = unit weight of soil, lb/ft^3

The total active pressure, lb/lin ft, is

$$E_a = K_a\frac{wh^2}{2} \tag{7.59}$$

Because of frictional resistance to sliding at the face of the wall, E_a is inclined at an angle δ with the normal to the wall, where δ is the angle of wall friction, deg (Fig. 7.43*a*). If the face of the wall is vertical, the horizontal active pressure equals $E_a \cos \delta$. If the face makes an angle β with the vertical (Fig. 7.43*a*), the pressure equals $E_a \cos(\delta + \beta)$. The resultant acts at a distance of $h/3$ above the base of the wall.

If the ground slopes upward from the top of the wall at an angle α, deg, with the horizontal, then for cohesionless soils

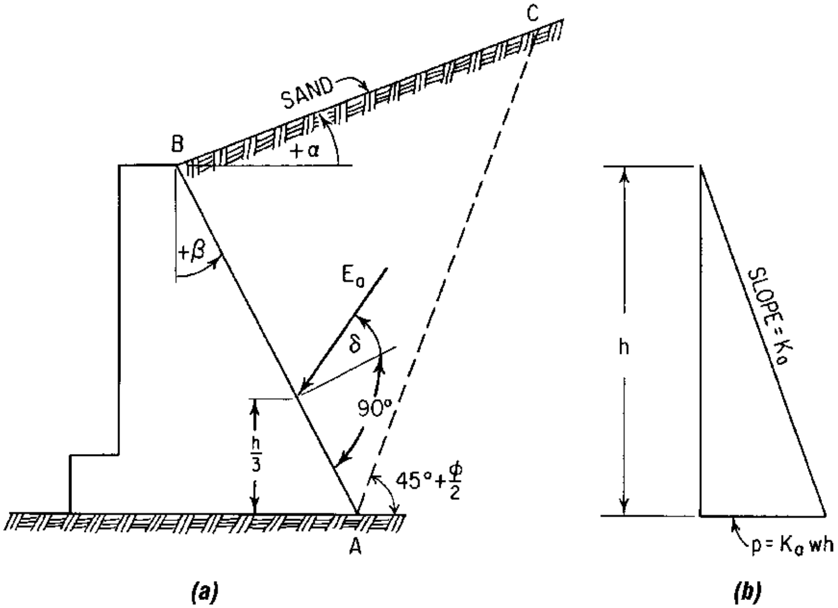

Fig. 7.43 Free-standing wall with sand backfill (*a*) is subjected to (*b*) triangular pressure distribution.

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Table 7.14 Active-Lateral-Pressure Coefficients K_a

		ϕ						
		10°	15°	20°	25°	30°	35°	40°
$\beta = 0$	$\alpha = 0$	0.70	0.59	0.49	0.41	0.33	0.27	0.22
	$\alpha = 10°$	0.97	0.70	0.57	0.47	0.37	0.30	0.24
	$\alpha = 20°$	—	—	0.88	0.57	0.44	0.34	0.27
	$\alpha = 30°$	—	—	—	—	0.75	0.43	0.32
	$\alpha = \phi$	0.97	0.93	0.88	0.82	0.75	0.67	0.59
$\beta = 10°$	$\alpha = 0$	0.76	0.65	0.55	0.48	0.41	0.43	0.29
	$\alpha = 10°$	1.05	0.78	0.64	0.55	0.47	0.38	0.32
	$\alpha = 20°$	—	—	1.02	0.69	0.55	0.45	0.36
	$\alpha = 30°$	—	—	—	—	0.92	0.56	0.43
	$\alpha = \phi$	1.05	1.04	1.02	0.98	0.92	0.86	0.79
$\beta = 20°$	$\alpha = 0$	0.83	0.74	0.65	0.57	0.50	0.43	0.38
	$\alpha = 10°$	1.17	0.90	0.77	0.66	0.57	0.49	0.43
	$\alpha = 20°$	—	—	1.21	0.83	0.69	0.57	0.49
	$\alpha = 30°$	—	—	—	—	1.17	0.73	0.59
	$\alpha = \phi$	1.17	1.20	1.21	1.20	1.17	1.12	1.06
$\beta = 30°$	$\alpha = 0$	0.94	0.86	0.78	0.70	0.62	0.56	0.49
	$\alpha = 10°$	1.37	1.06	0.94	0.83	0.74	0.65	0.56
	$\alpha = 20°$	—	—	1.51	1.06	0.89	0.77	0.66
	$\alpha = 30°$	—	—	—	—	1.55	0.99	0.79
	$\alpha = \phi$	1.37	1.45	1.51	1.54	1.55	1.54	1.51

Table 7.15 Angles of Internal Friction and Unit Weights of Soils

Types of Soil	Density or Consistency	Angle of Internal Friction ϕ, deg	Unit Weight w, lb/ft^3
Coarse sand or sand and gravel	Compact	40	140
	Loose	35	90
Medium sand	Compact	40	130
	Loose	30	90
Fine silty sand or sandy silt	Compact	30	130
	Loose	25	85
Uniform silt	Compact	30	135
	Loose	25	85
Clay-silt	Soft to medium	20	90–120
Silty clay	Soft to medium	15	90–120
Clay	Soft to medium	0–10	90–120

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$$K_a = \frac{\cos^2(\phi - \beta)}{\cos^2\beta\cos(\delta + \beta)\left[1 + \sqrt{\dfrac{\sin(\phi + \delta)\sin(\phi - \alpha)}{\cos(\delta + \beta)\cos(\alpha - \beta)}}\right]^{-2}} \tag{7.60}$$

The effect of wall friction on K_a is small and usually is neglected. For $\delta = 0$,

$$K_a = \frac{\cos^2(\phi - \beta)}{\cos^3\beta\left[1 + \sqrt{\dfrac{\sin\phi\sin(\phi - \alpha)}{\cos\beta\cos(\alpha - \beta)}}\right]^{2}} \tag{7.61}$$

Table 7.14 lists values of K_a determined from Eq. (7.61). Approximate values of ϕ and unit weights for various soils are given in Table 7.15.

For level ground at the top of the wall ($\alpha = 0$):

$$K_a = \frac{\cos^2(\phi - \beta)}{\cos^3\beta\left(1 + \dfrac{\sin\phi}{\cos\beta}\right)^{2}} \tag{7.62}$$

If in addition, the back face of the wall is vertical ($\beta = 0$), Rankine's equation is obtained:

$$K_a = \frac{1 - \sin\phi}{1 + \sin\phi} \tag{7.63}$$

Coulomb derived the trigonometric equivalent:

$$K_a = \tan^2\left(45^\circ - \frac{\phi}{2}\right) \tag{7.64}$$

The selection of the wall friction angle should be carefully applied as it has a significant effect on the resulting earth pressure force.

Unyielding walls retaining walls backfilled with sand and gravel, such as the abutment walls of a rigid-frame concrete bridge or foundation walls braced by floors, do not allow shearing resistance to develop in the sand along planes that can be determined analytically. For such walls, triangular pressure diagrams may be assumed, and K_a may be taken equal to 0.5.

Braced walls retaining cuts in sand (Fig. 7.44*a*) are subjected to earth pressure gradually and

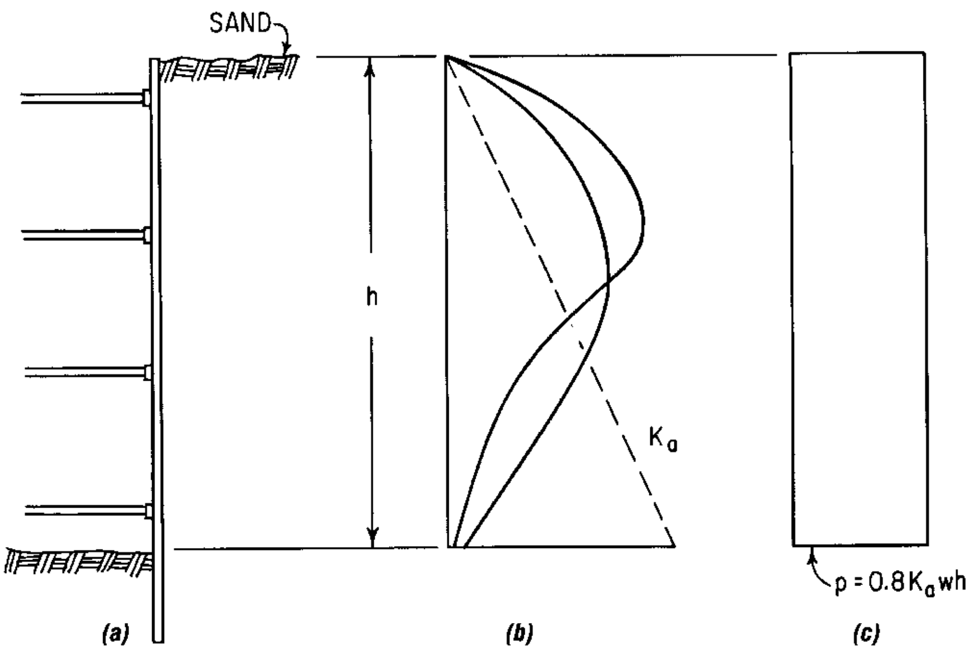

Fig. 7.44 Braced wall retaining sand (*a*) may have to resist pressure distributions of the type shown in (*b*). (*c*) Uniform pressure distribution may be assumed for design.

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develop resistance in increments as excavation proceeds and braces are installed. Such walls tend to rotate about a point in the upper portion. Hence, the active pressures do not vary linearly with depth. Field measurements have yielded a variety of curves for the pressure diagram, of which two types are shown in Fig. 7.44*b*. Consequently, some authorities have recommended a trapezoidal pressure diagram, with a maximum ordinate

$$p = 0.8K_a wh \tag{7.65}$$

K_a may be obtained from Table 7.14. The total pressure exceeds that for a triangular distribution.

Figure 7.45 shows earth-pressure diagrams developed for a sandy soil and a clayey soil. In both cases, the braced wall is subjected to a 3-ft-deep surcharge, and height of wall is 34 ft. For the sandy soil (Fig. 7.45*a*), Fig. 7.45*b* shows the pressure diagram assumed. The maximum pressure can be obtained from Eq. (7.65), with $h = 34 + 3 = 37$ ft and K_a assumed as 0.30 and w as 110 lb/ft^3.

$$p_1 = 0.8 \times 0.3 \times 110 \times 37 = 975\,\text{lb/ft}^2$$

The total pressure is estimated as

$$P = 0.8 \times 975 \times 37 = 28{,}900\,\text{lb/lin ft}$$

The equivalent maximum pressure for a trapezoidal diagram for the 34-ft height of the wall then is

$$p = \frac{28{,}900}{0.8 \times 34} = 1060\,\text{lb/ft}^2$$

Assumption of a uniform distribution (Fig. 7.44*c*), however, simplifies the calculations and has little or no effect on the design of the sheeting and braces, which should be substantial to withstand construction abuses. Furthermore, trapezoidal loading terminating at the level of the excavation may not apply if piles are driven inside the completed

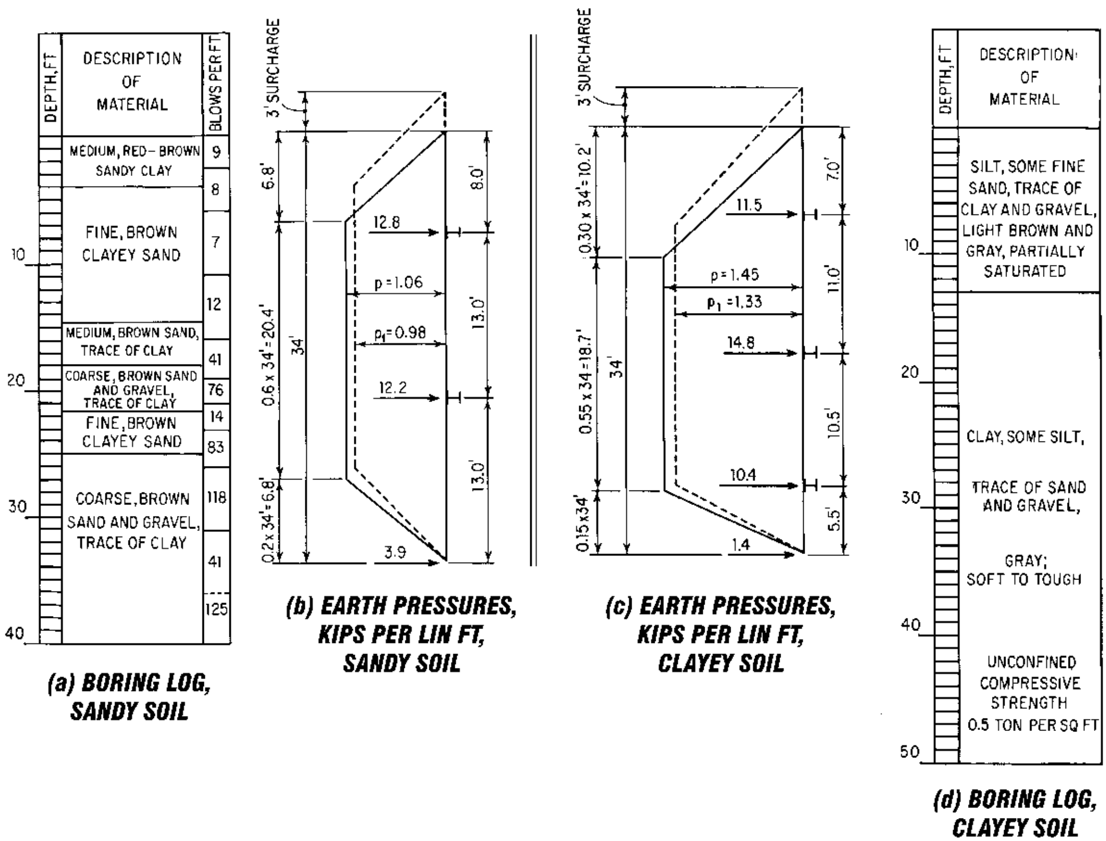

Fig. 7.45 Assumed trapezoidal diagrams for a braced wall in soils described by boring logs in (*a*) and (*d*).

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excavation. The shocks may temporarily decrease the passive resistance of the sand in which the wall is embedded and lower the inflection point. This would increase the span between the inflection point and the lowest brace and increase the pressure on that brace. Hence, uniform pressure distribution may be more applicable than trapezoidal for such conditions.

See Note for free-standing walls.

Flexible retaining walls in sand cuts are subjected to pressures that depend on the fixity of the anchorage. If the anchor moves sufficiently or the tie from the anchor to the upper portion of the bulkhead stretches enough, the bulkhead may rotate slightly about a point near the bottom. In that case, the sliding-wedge theory may apply. The pressure distribution may be taken as triangular, and Eqs. (7.58) to (7.64) may be used. But if the anchor does not yield, then pressure distributions much like those in Fig. 7.44*b* for a braced cut may occur. Either a trapezoidal or uniform pressure distribution may be assumed, with maximum pressure given by Eq. (7.65). Stresses in the tie should be kept low because it may have to resist unanticipated pressures, especially those resulting from a redistribution of forces from soil arching. (Federal Highway Administration Geotechnical Engineering Circular No.4 "Ground Anchors and Anchored Systems" FHWA-IF-99015, June 1999).

Free-standing walls retaining plastic-clay cuts (Fig. 7.46*a*) may have to resist two types of active lateral pressure, both with triangular distribution. In the short term the shearing resistance is due to cohesion only, a clay bank may be expected to stand with a vertical face without support for a height, ft, of

$$h' = \frac{2c}{w} \tag{7.66}$$

where $2c$ = unconfined compressive strength of clay, lb/ft^2
w = unit weight of clay, lb/ft^3

So if there is a slight rotation of the wall about its base, the upper portion of the clay cut will stand vertically without support for a depth h'. Below that, the pressure will increase linearly with depth as if the clay were a heavy liquid (Fig. 7.46*b*):

$$p = wh - 2c$$

The total pressure, lb/lin ft, then is

$$E_a = \frac{w}{2}\left(h - \frac{2c}{w}\right)^2 \tag{7.67}$$

It acts at a distance $(h - 2c/w)/3$ above the base of the wall. These equations assume wall friction is zero, the back face of the wall is vertical, and the ground is level.

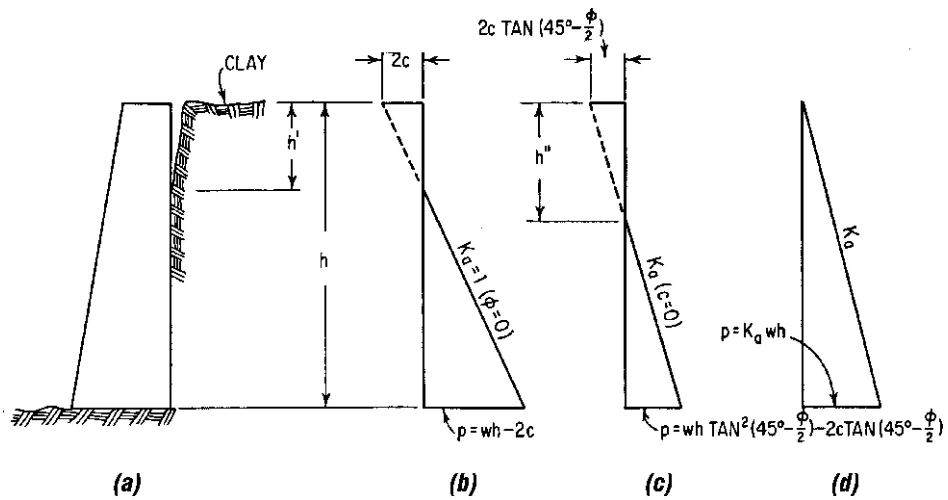

Fig. 7.46 Free-standing wall retaining clay (*a*) may have to resist the pressure distribution shown in (*b*) or (*d*). For mixed soils, the distribution may approximate that shown in (*c*).

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In time the clay will reach its long term strength and the pressure distribution may become approximately triangular (Fig. 7.46*d*) from the top of the wall to the base. The pressures then may be calculated from Eqs. (7.58) to (7.64) with an apparent angle of internal friction for the soil (for example, see the values of ϕ in Table 7.15). The wall should be designed for the pressures producing the highest stresses and overturning moments.

Note: The finer the backfill material, the more likely it is that pressures greater than active will develop, because of plastic deformations, water-level fluctuation, temperature changes, and other effects. As a result, it would be advisable to use in design at least the **coefficient for earth pressure at rest:**

$$K_o = 1 - \sin\phi \tag{7.68}$$

The safety factor should be at least 2.5.

Clay should not be used behind retaining walls, where other economical alternatives are available. The swelling type especially should be avoided because it can cause high pressures and progressive shifting or rotation of the wall.

For a mixture of cohesive and cohesionless soils, the pressure distribution may temporarily be as shown in Fig. 7.46*c*. The height, ft, of the unsupported vertical face of the clay is

$$h'' = \frac{2c}{w\tan(45^\circ - \phi/2)} \tag{7.69}$$

The pressure at the base is

$$p = wh\tan^2\left(45^\circ - \frac{\phi}{2}\right) - 2c\tan\left(45^\circ - \frac{\phi}{2}\right) \tag{7.70}$$

The total pressure, lb/lin ft, is

$$E_a = \frac{w}{2}\left[h\tan\left(45^\circ - \frac{\phi}{2}\right) - \frac{2c}{w}\right]^2 \tag{7.71}$$

It acts at a distance $(h - h'')/3$ above the base of the wall.

Braced walls retaining clay cuts (Fig. 7.47*a*) also may have to resist two types of active lateral pressure. As for sand, the pressure distribution may temporarily be approximated by a trapezoidal diagram (Fig. 7.47*b*). On the basis of field observations, R. B. Peck has recommended a maximum pressure of

$$p = wh - 4c \tag{7.72}$$

and a total pressure, lb/lin ft, of

$$E_a = \frac{1.55h}{2}(wh - 4c) \tag{7.73}$$

[R. B. Peck, "Earth Pressure Measurements in Open Cuts, Chicago (Ill.) Subway," *Transactions, American Society of Civil Engineers*, 1943, pp. 1008–1036.]

Figure 7.47*c* shows a trapezoidal earth-pressure diagram determined for the clayey-soil condition of Fig. 7.47*d*. The weight of the soil is taken as 120 lb/ft^3; c is assumed as zero and the active-

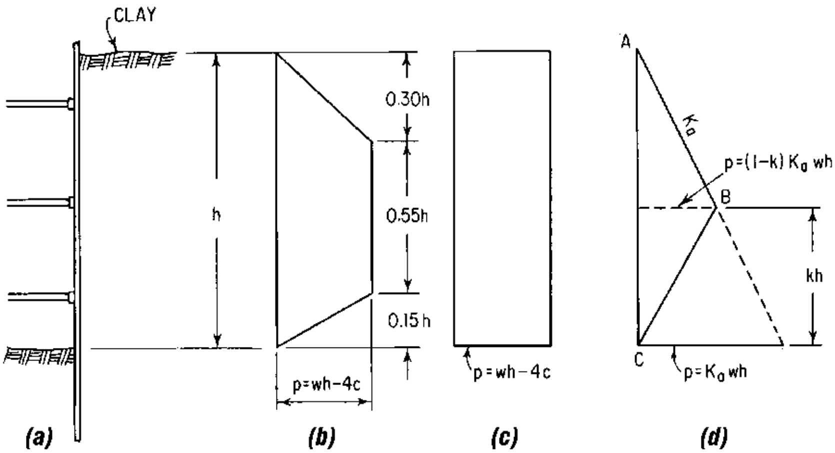

Fig. 7.47 Braced wall retaining clay (*a*) may have to resist pressures approximated by the pressure distribution in (*b*) or (*d*). Uniform distribution (*c*) may be assumed in design.

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lateral-pressure coefficient as 0.3. Height of the wall is 34 ft, surcharge 3 ft. Then, the maximum pressure, obtained from Eq. (7.58) since the soil is clayey, not pure clay, is

$$p_1 = 0.3 \times 120 \times 37 = 1330\ \text{lb/ft}^2$$

From Eq. (7.73) with the above assumptions, the total pressure is

$$P = \frac{1.55}{2} \times 37 \times 1330 = 38{,}100\ \text{lb/lin ft}$$

The equivalent maximum pressure for a trapezoidal diagram for the 34-ft height of wall is

$$p = \frac{38{,}100}{34} \times \frac{2}{1.55} = 1450\ \text{lb/ft}^2$$

To simplify calculations, a uniform pressure distribution may be used instead (Fig. 7.47*c*).

If after a time the clay should attain a consolidated equilibrium state, the pressure distribution may be better represented by a triangular diagram *ABC* (Fig. 7.47*d*), as suggested by G. P. Tschebotarioff. The peak pressure may be assumed at a distance of $kh = 0.4h$ above the excavation level for a stiff clay; that is, $k = 0.4$. For a medium clay, k may be taken as 0.25, and for a soft clay, as 0. For computing the pressures, K_a may be estimated from Table 7.14 with an apparent angle of friction obtained from laboratory tests or approximated from Table 7.15. The wall should be designed for the pressures producing the highest stresses and overturning moments.

See also Note for free-standing walls.

Flexible retaining walls in clay cuts and anchored near the top similarly should be checked for two types of pressures. When the anchor is likely to yield slightly or the tie to stretch, the pressure distribution in Fig. 7.47*d* with $k = 0$ may be applicable. For an unyielding anchor, any of the pressure distributions in Fig. 7.47 may be assumed, as for a braced wall. The safety factor for design of ties and anchorages should be at least twice that used in conventional design. See also Note for free-standing walls.

Backfill placed against a retaining wall should preferably be sand or gravel (free draining) to facilitate drainage. Also, weepholes should be provided through the wall near the bottom and a drain installed along the footing, to conduct water from the back of the wall and prevent buildup of hydrostatic pressures.

Saturated or submerged soil imposes substantially greater pressure on a retaining wall than dry or moist soil. The active lateral pressure for a soil-fluid backfill is the sum of the hydrostatic pressure and the lateral soil pressure based on the buoyed unit weight of the soil. This weight roughly may be 60% of the dry weight.

Surcharge, or loading imposed on a backfill, increases the active lateral pressure on a wall and raises the line of action of the total, or resultant, pressure. A surcharge w_s, lb/ft^2, uniformly distributed over the entire ground surface may be taken as equivalent to a layer of soil of the same unit weight w as the backfill and with a thickness of w_s/w. The active lateral pressure, lb/ft^2, due to the surcharge, from the backfill surface down, then will be $K_a w_s$. This should be added to the lateral pressures that would exist without the surcharge. K_a may be obtained from Table 7.14.

(A. Caquot and J. Kérisel, "Tables for Calculation of Passive Pressure, Active Pressure, and Bearing Capacity of Foundations," Gauthier-Villars, Paris.)

7.28 Passive Lateral Pressure on Retaining Walls and Anchors

As defined in Art. 7.27, active pressure tends to move a structure in the direction in which pressure acts, whereas passive pressure opposes motion of a structure.

Passive pressures of cohesionless soils, resisting movement of a wall or anchor, develop because of internal friction in the soils. Because of friction between soil and wall, the failure surface is curved, not plane as assumed in the Coulomb sliding-wedge theory (Art. 7.27). Use of the Coulomb theory yields unsafe values of passive pressure when the effects of wall friction are included.

Total passive pressure, lb/lin ft, on a wall or anchor extending to the ground surface (Fig. 7.48*a*) may be expressed for sand in the form

$$P = K_p \frac{wh^2}{2} \tag{7.74}$$

where K_p = coefficient of passive lateral pressure

w = unit weight of soil, lb/ft^3

h = height of wall or anchor to ground surface, ft

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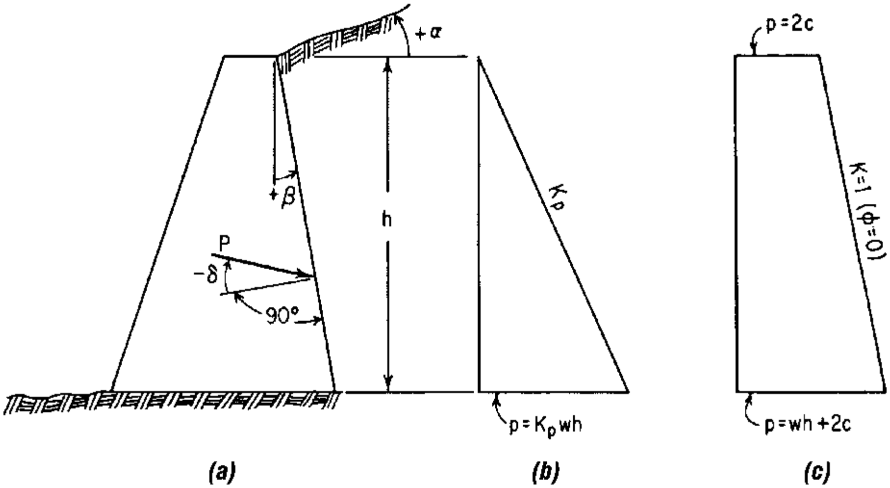

Fig. 7.48 Passive pressure on a wall (*a*) may vary as shown in (*b*) for sand or as shown in (*c*) for clay.

The pressure distribution usually assumed for sand is shown in Fig. 7.48*b*. Table 7.16 lists values of K_p for a vertical wall face ($\beta = 0$) and horizontal ground surface ($\alpha = 0$), for curved surfaces of failure. (Many tables and diagrams for determining passive pressures are given in A. Caquot and J. Kérisel, "Tables for Calculation of Passive Pressure, Active Pressure, and Bearing Capacity of Foundations," Gauthier-Villars, Paris.)

Since a wall usually transmits a downward shearing force to the soil, the angle of wall friction δ correspondingly is negative (Fig. 7.48*a*). For embedded portions of structures, such as anchored sheetpile bulkheads, δ and the angle of internal friction ϕ of the soil reach their peak values simultaneously in dense sand. For those conditions, if specific information is not available, δ may be assumed as $-\frac{2}{3}\phi$ (for $\phi > 30°$). For such structures as a heavy anchor block subjected to a horizontal pull or thrust, δ may be taken as $-\phi/2$ for dense sand. For those cases, the wall friction develops as the sand is pushed upward by the anchor and is unlikely to reach its maximum value before the internal resistance of the sand is exceeded.

When wall friction is zero ($\delta = 0$), the failure surface is a plane inclined at an angle of $45° - \phi/2$ with the horizontal. The sliding-wedge theory then yields

$$K_p = \frac{\cos^2(\phi + \beta)}{\cos^3\beta\left[1 - \sqrt{\dfrac{\sin\phi\sin(\phi+\alpha)}{\cos\beta\cos(\alpha-\beta)}}\right]^2} \tag{7.75}$$

When the ground is horizontal ($\alpha = 0$):

$$K_p = \frac{\cos^2(\phi+\beta)}{\cos^3\beta(1 - \sin\phi/\cos\beta)^2} \tag{7.76}$$

If, in addition, the back face of the wall is vertical ($\beta = 0$):

$$K_p = \frac{1+\sin\phi}{1-\sin\phi} = \tan^2\left(45° + \frac{\phi}{2}\right) = \frac{1}{K_a} \tag{7.77}$$

Table 7.16 Passive Lateral-Pressure Coefficients K_p*

	$\phi = 10°$	$\phi = 15°$	$\phi = 20°$	$\phi = 25°$	$\phi = 30°$	$\phi = 35°$	$\phi = 40°$
$\delta = 0$	1.42	1.70	2.04	2.56	3.00	3.70	4.60
$\delta = -\phi/2$	1.56	1.98	2.59	3.46	4.78	6.88	10.38
$\delta = -\phi$	1.65	2.19	3.01	4.29	6.42	10.20	17.50

* For vertical wall face ($\beta = 0$) and horizontal ground surface ($\alpha = 0$).

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The first line of Table 7.16 lists values obtained from Eq. (7.77).

Continuous anchors in sand ($\phi = 33°$), when subjected to horizontal pull or thrust, develop passive pressures, lb/lin ft, of about

$$P = 1.5wh^2 \tag{7.78}$$

where h = distance from bottom of anchor to the surface, ft.

This relationship holds for ratios of h to height d, ft, of anchor of 1.5 to 5.5, and assumes a horizontal ground surface and vertical anchor face.

Square anchors within the same range of h/d develop about

$$P = \left(2.50 + \frac{h}{8d}\right)^2 d\frac{wh^2}{2} \tag{7.79}$$

where P = passive lateral pressure, lb

d = length and height of anchor, ft

Passive pressures of cohesive soils, resisting movement of a wall or anchor extending to the ground surface, depend on the unit weight of the soil w and its unconfined compressive strength $2c$, psf. At a distance h, ft, below the surface, the passive lateral pressure, psf, is

$$p = wh + 2c \tag{7.80}$$

The total pressure, lb/lin ft, is

$$P = \frac{wh^2}{2} + 2ch \tag{7.81}$$

and acts at a distance, ft, above the bottom of the wall or anchor of

$$\bar{x} = \frac{h(wh + 6c)}{3(wh + 4c)}$$

The pressure distribution for plastic clay is shown in Fig. 7.48*c*.

Continuous anchors in plastic clay, when subjected to horizontal pull or thrust, develop passive pressures, lb/lin ft, of about

$$P = cd\left[8.7 - \frac{11{,}600}{(h/d + 11)^3}\right] \tag{7.82}$$

where h = distance from bottom of anchor to surface, ft

d = height of anchor, ft

Equation (7.82) is based on tests made with horizontal ground surface and vertical anchor face.

Safety factors should be applied to the passive pressures computed from Eqs. (7.74) to (7.82) for design use. Experience indicates that a safety factor of 2 is satisfactory for clean sands and gravels. For clay, a safety factor of 3 may be desirable because of uncertainties as to effective shearing strength.

(G. P. Tschebotarioff, "Soil Mechanics, Foundations, and Earth Structures," McGraw-Hill Book Company, New York (books.mcgraw-hill.com); K. Terzaghi and R. B. Peck, "Soil Mechanics in Engineering Practice," John Wiley & Sons, Inc., New York (www.wiley.com); Leo Casagrande, "Comments on Conventional Design of Retaining Structures," *ASCE Journal of Soil Mechanics and Foundations Engineering Division*, 1973, pp. 181–198; H. Y. Fang, "Foundation Engineering Handbook," 2nd ed., Van Nostrand Reinhold Company, New York.)

7.29 Vertical Earth Pressure on Conduit

The vertical load on an underground conduit depends principally on the weight of the prism of soil directly above it. But the load also is affected by vertical shearing forces along the sides of this prism. Caused by differential settlement of the prism and adjoining soil, the shearing forces may be directed up or down. Hence, the load on the conduit may be less or greater than the weight of the soil prism directly above it.

Conduits are classified as ditch or projecting, depending on installation conditions that affect the shears. A ditch conduit is a pipe set in a relatively narrow trench dug in undisturbed soil (Fig. 7.49). Backfill then is placed in the trench up to the original ground surface. A projecting conduit is a pipe over which an embankment is placed.

A projecting conduit may be positive or negative, depending on the extent of the embankment vertically. A positive projecting conduit is installed in a shallow bed with the pipe top above the surface of the ground. Then, the embankment is placed over the pipe (Fig. 7.50*a*). A negative projecting conduit is set in a narrow, shallow trench with the pipe top below the original ground surface (Fig. 7.50*b*). Then, the ditch is backfilled, after which the embankment is placed. The load on the conduit is less when the backfill is not compacted.

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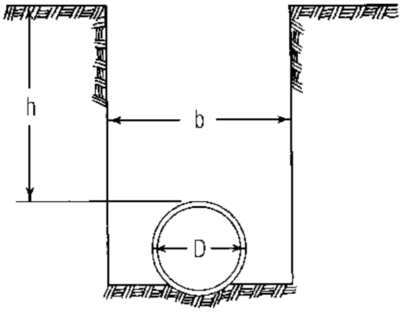

Fig. 7.49 Ditch conduit.

Load on underground pipe also may be reduced by the imperfect-ditch method of construction. This starts out as for a positive projecting conduit, with the pipe at the original ground surface. The embankment is placed and compacted for a few feet above the pipe. But then, a trench as wide as the conduit is dug down to it through the compacted soil. The trench is backfilled with a loose, compressible soil (Fig. 7.50*c*). After that, the embankment is completed.

The load, lb/lin ft, on a rigid ditch conduit may be computed from

$$W = C_D whb \tag{7.83}$$

and on a flexible ditch conduit from

$$W = C_D whD \tag{7.84}$$

where C_D = load coefficient for ditch conduit
w = unit weight of fill, lb/ft^3
h = height of fill above top of conduit, ft
b = width of ditch at top of conduit, ft
D = outside diameter of conduit, ft

From the equilibrium of vertical forces, including shears, acting on the backfill above the conduit, C_D may be determined:

$$C_D = \frac{1 - e^{-kh/b}}{k}\,\frac{b}{h} \tag{7.85}$$

where $e = 2.718$
$k = 2K_a \tan\theta$
K_a = coefficient of active earth pressure [Eq. (7.64) and Table 7.14]
θ = angle of friction between fill and adjacent soil ($\theta \le \phi$, angle of internal friction of fill)

Table 7.17 gives values of C_D for $k = 0.33$ for cohesionless soils, $k = 0.30$ for saturated topsoil, and $k = 0.26$ and 0.22 for clay (usual maximum and saturated).

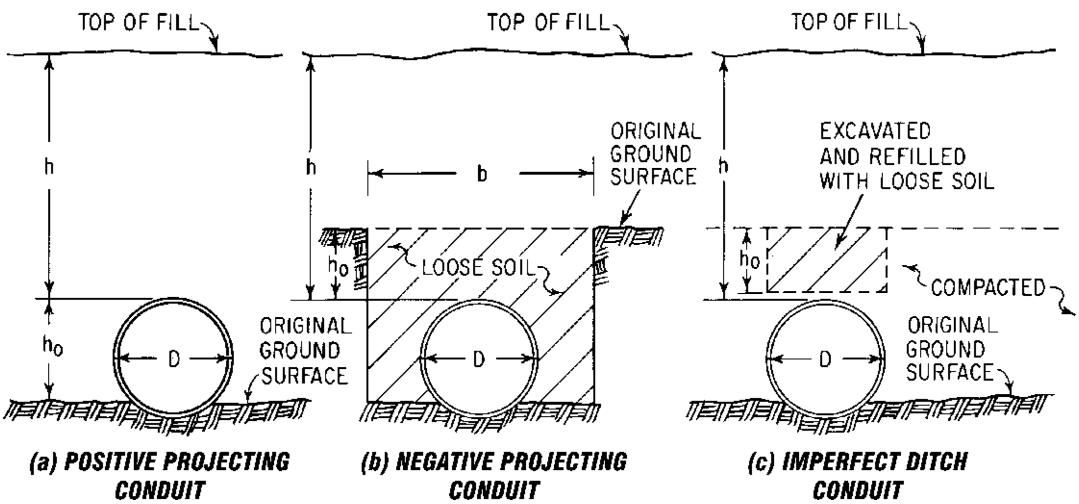

Fig. 7.50 Type of projecting conduit depends on method of backfilling.

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Table 7.17 Load Coefficients C_D for Ditch Conduit

h/b	Cohesionless Soils	Saturated Topsoil	Clay $k = 0.26$	Clay $k = 0.22$
1	0.85	0.86	0.88	0.89
2	0.75	0.75	0.78	0.80
3	0.63	0.67	0.69	0.73
4	0.55	0.58	0.62	0.67
5	0.50	0.52	0.56	0.60
6	0.44	0.47	0.51	0.55
7	0.39	0.42	0.46	0.51
8	0.35	0.38	0.42	0.47
9	0.32	0.34	0.39	0.43
10	0.30	0.32	0.36	0.40
11	0.27	0.29	0.33	0.37
12	0.25	0.27	0.31	0.35
Over 12	$3.0b/h$	$3.3b/h$	$3.9b/h$	$4.5b/h$

Vertical load, lb/lin ft, on conduit installed by tunneling may be estimated from

$$W = C_D b(wh - 2c) \tag{7.86}$$

where c = cohesion of the soil, or half the unconfined compressive strength of the soil, psf. The load coefficient C_D may be computed from Eq. (7.85) or obtained from Table 7.17 with b = maximum width of tunnel excavation, ft, and h = distance from tunnel top to ground surface, ft.

For a ditch conduit, shearing forces extend from the pipe top to the ground surface. For a projecting conduit, however, if the embankment is sufficiently high, the shear may become zero at a horizontal plane below grade, the plane of equal settlement. Load on a projecting conduit is affected by the location of this plane.

Vertical load, lb/lin ft, on a positive projecting conduit may be computed from

$$W = C_P whD \tag{7.87}$$

where C_P = load coefficient for positive projecting conduit. Formulas have been derived for C_P and the depth of the plane of equal settlement. These formulas, however, are too lengthy for practical application, and the computation does not appear to be justified by the uncertainties in actual relative settlement of the soil above the conduit. Tests may be made in the field to determine C_P. If so, the possibility of an increase in earth pressure with time should be considered. For a rough estimate, C_P may be assumed as 1 for flexible conduit and 1.5 for rigid conduit.

The vertical load, lb/lin ft, on negative projecting conduit may be computed from

$$W = C_N whb \tag{7.88}$$

where C_N = load coefficient for negative projecting conduit

h = height of fill above top of conduit, ft

b = horizontal width of trench at top of conduit, ft

The load on an imperfect ditch conduit may be obtained from

$$W = C_N whD \tag{7.89}$$

where D = outside diameter of conduit, ft.

Formulas have been derived for C_N, but they are complex, and insufficient values are available for the parameters involved. As a rough guide, C_N may be taken as 0.9 when depth of cover exceeds conduit diameter. (See also Art. 10.31.)

Superimposed surface loads increase the load on an underground conduit. The magnitude of the increase depends on the depth of the pipe below grade and the type of soil. For moving loads, an impact factor of about 2 should be applied. A superimposed uniform load w', lb/ft^2, of large extent may be treated for projecting conduit as an equivalent layer of embankment with a thickness, ft, of w'/w. For ditch conduit, the load due to the

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soil should be increased by $bw'e^{-kh/b}$, where $k = 2K_a \tan\theta$, as in Eq. (7.85). The increase due to concentrated loads can be estimated by assuming the loads to spread out linearly with depth, at an angle of about 30° with the vertical. (See also Art. 7.11).

(M. G. Spangler, "Soil Engineering," International Textbook Company, Scranton, Pa.; "Handbook of Steel Drainage and Highway Construction Products," American Iron and Steel Institute, Washington, D. C. (www.steel.org).)

7.30 Dewatering Methods for Excavations

The main purpose of dewatering is to enable construction to be carried out under relatively dry conditions. Good drainage stabilizes excavated slopes, reduces lateral loads on sheeting and bracing, and reduces required air pressure in tunneling. Dewatering makes excavated material lighter and easier to handle. It also prevents loss of soil below slopes or from the bottom of the excavation, and prevents a "quick" or "boiling" bottom. In addition, permanent lowering of the groundwater table or relief of artesian pressure may allow a less expensive design for the structure, especially when the soil consolidates or becomes compact. If lowering of the water level or pressure relief is temporary, however, the improvement of the soil should not be considered in foundation design. Increases in strength and bearing capacity may be lost when the soil again becomes saturated.

To keep an excavation reasonably dry, the groundwater table should be kept at least 2 ft, and preferably 5 ft, below the bottom in most soils.

Site investigations should yield information useful for deciding on the most suitable and economical dewatering method. Important is a knowledge of the types of soil in and below the site, probable groundwater levels during construction, permeability of the soils, and quantities of water to be handled. A pumping test may be desirable for estimating capacity of pumps needed and drainage characteristics of the ground.

Many methods have been used for dewatering excavations. Those used most often are listed in Table 7.18 with conditions for which they generally are most suitable. (See also Art. 7.37.)

In many small excavations, or where there are dense or cemented soils, water may be collected in ditches or sumps at the bottom and pumped out. This is the most economical method of dewatering, and the sumps do not interfere with future construction as does a comprehensive wellpoint system. But the seepage may slough the slopes, unless they are stabilized with gravel, and may hold up excavation while the soil drains. Also, springs may develop in fine sand or silt and cause underground erosion and subsidence of the ground surface.

For sheetpile-enclosed excavations in pervious soils, it is advisable to intercept water before it enters the enclosure. Otherwise, the water will put high pressures on the sheeting. Seepage also can cause the excavation bottom to become quick, overloading the bottom bracing, or create piping, undermining the sheeting. Furthermore, pumping from the inside of the cofferdam is likely to leave the soil to be excavated wet and tough to handle.

Wellpoints often are used for lowering the water table in pervious soils. They are not suitable, however, in soils that are so fine that they will flow with the water or in soils with low permeability. Also, other methods may be more economical for deep excavations, very heavy flows, or considerable lowering of the water table (Table 7.18).

Wellpoints are metal well screens about 2 to 3 in in diameter and up to about 4 ft long. A pipe connects each wellpoint to a header, from which water is pumped to discharge (Fig. 7.51). Each pump usually is a combined vacuum and centrifugal pump. Spacing of wellpoints generally ranges from 3 to 12 ft c to c.

A wellpoint may be jetted into position or set in a hole made with a hole puncher or heavy steel casing. Accordingly, wellpoints may be self-jetting or plain-tip. To insure good drainage in fine and dirty sands or in layers of silt or clay, the wellpoint and riser should be surrounded by sand to just below the water table. The space above the filter should be sealed with silt or clay to keep air from getting into the wellpoint through the filter.

Wellpoints generally are relied on to lower the water table 15 to 20 ft. Deep excavations may be dewatered with multistage wellpoints, with one row of wellpoints for every 15 ft of depth. Or when the flow is less than about 15 gal/min per wellpoint, a single-stage system of wellpoints may be installed above the water table and operated with

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jet-eductor pumps atop the wellpoints. These pumps can lower the water table up to about 100 ft, but they have an efficiency of only about 30%.

Deep wells may be used in pervious soils for deep excavations, large lowering of the water table, and heavy water flows. They may be placed along the top of an excavation to drain it, to intercept seepage before pressure makes slopes unstable, and to relieve artesian pressure before it heaves the excavation bottom.

Usual spacing of wells ranges from 20 to 250 ft. Diameter generally ranges from 6 to 20 in. Well screens may be 20 to 75 ft long, and they are surrounded with a sand-gravel filter. Generally, pumping is done with a submersible or vertical turbine pump installed near the bottom of each well.

Figure 7.52 shows a deep-well installation used for a 300-ft-wide by 600-ft-long excavation for a building of the Smithsonian Institution, Washington, D. C. Two deep-well pumps lowered the general water level in the excavation 20 ft. The well installation proceeded as follows: (1) Excavation to water level (elevation 0.0). (2) Driving of sheetpiles around the well area (Fig. 7.52*a*). (3) Excavation underwater inside the sheetpile enclosure to elevation −37.0 ft (Fig. 7.52*b*). Bracing installed as digging progressed. (4) Installation of a wire-mesh-wrapped timber frame, extending from elevation 0.0 to −37.0 (Fig. 7.52*c*). Weights added to sink the frame. (5) Backfilling of space between sheetpiles and mesh with $\frac{3}{16}$- to $\frac{3}{8}$-in gravel. (6) Removal of sheetpiles. (7) Installation of pump and start of pumping.

Vacuum well or wellpoint systems may be used to drain silts with low permeability (coefficient between 0.01 and 0.0001 mm/s). In these systems,

Table 7.18 Methods for Dewatering Excavations

Saturated-Soil Conditions	Dewatering Method Probably Suitable
Surface water	Ditches; dikes; sheetpiles and pumps or underwater excavation and concrete tremie seal
Gravel	Underwater excavation, grout curtain; gravity drainage with large sumps with gravel filters
Sand (except very fine sand)	Gravity drainage
Waterbearing strata near surface; water table does not have to be lowered more than 15 ft	Wellpoints with vacuum and centrifugal pumps
Waterbearing strata near surface; water table to be lowered more than 15 ft, low pumping rate	Wellpoints with jet-eductor pumps
Excavations 30 ft or more below water table; artesian pressure; high pumping rate; large lowering of water table—all where adequate depth of pervious soil is available for submergence of well screen and pump	Deep wells, plus, if necessary, wellpoints
Sand underlain by rock near excavation bottom	Wellpoints to rock, plus ditches, drains, automatic “mops”
Sand underlain by clay	Wellpoints in holes 3 or 4 into the clay, backfilled with sand
Silt; very find sand (permeability coefficient between 0.01 and 0.0001 mm/s)	For lifts up to 15 ft, wellpoints with vacuum; for greater lifts, wells with vacuum; sumps
Silt or silty sand underlain by pervious soil	At top of excavation, and extending to the pervious soil, vertical sand drains plus well points or wells
Clay-silts, silts	Electro-osmosis
Clay underlain by pervious soil	At top of excavation, wellpoints or deep wells extending to pervious soil
Dense or cemented soils: small excavations	Ditches and sumps

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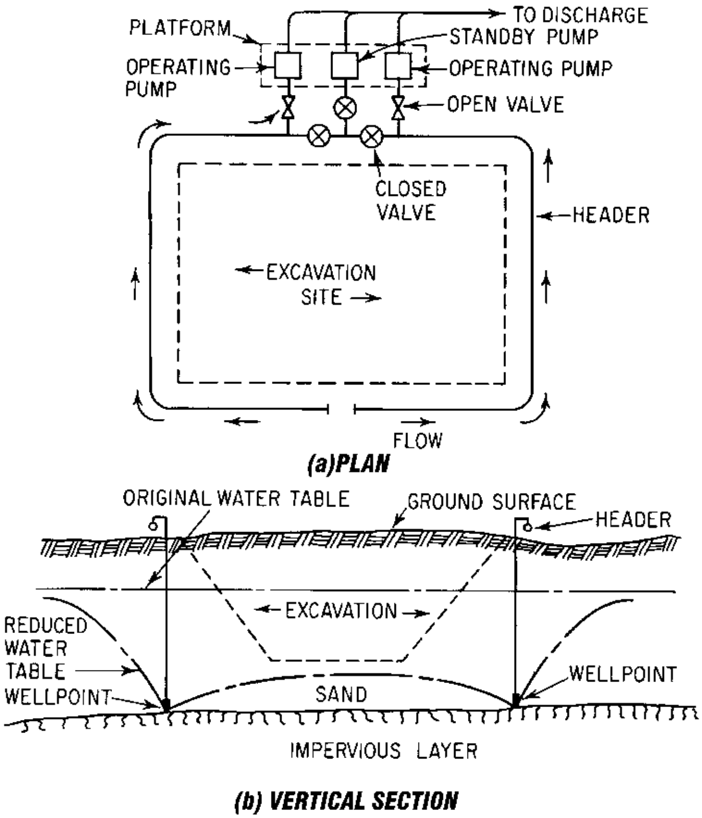

Fig. 7.51 Wellpoint system for dewatering an excavation.

wells or wellpoints are closely spaced, and a vacuum is held with vacuum pumps in the well screens and sand filters. At the top, the filter, well, and risers should be sealed to a depth of 5 ft with bentonite or an impervious soil to prevent loss of the vacuum. Water drawn to the well screens is pumped out with submersible or centrifugal pumps.

Where a pervious soil underlies silts or silty sands, vertical sand drains and deep wells can team up to dewater an excavation. Installed at the top, and extending to the pervious soil, the sand piles intercept seepage and allow it to drain down to the pervious soil. Pumping from the deep wells relieves the pressure in that deep soil layer.

For some silts and clay-silts, electrical drainage with wells or wellpoints may work, whereas gravity methods may not (Art. 7.37). In saturated clays, thermal or chemical stabilization may be necessary (Arts. 7.38 and 7.39).

Small amounts of surface water may be removed from excavations with "mops." Surrounded with gravel to prevent clogging, these drains are connected to a header with suction hose or pipe. For automatic operation, each mop should be opened and closed by a float and float valve.

When structures on silt or soft material are located near an excavation to be dewatered, care should be taken that lowering of the water table does not cause them to settle. It may be necessary to underpin the structures or to pump discharge water into recharge wells near the structures to maintain the water table around them.

(L. Zeevaert, "Foundation Engineering for Difficult Subsoil Conditions," H. Y. Fang, "Foundation Engineering Handbook," 2nd ed., Van Nostrand Reinhold Company, New York.)

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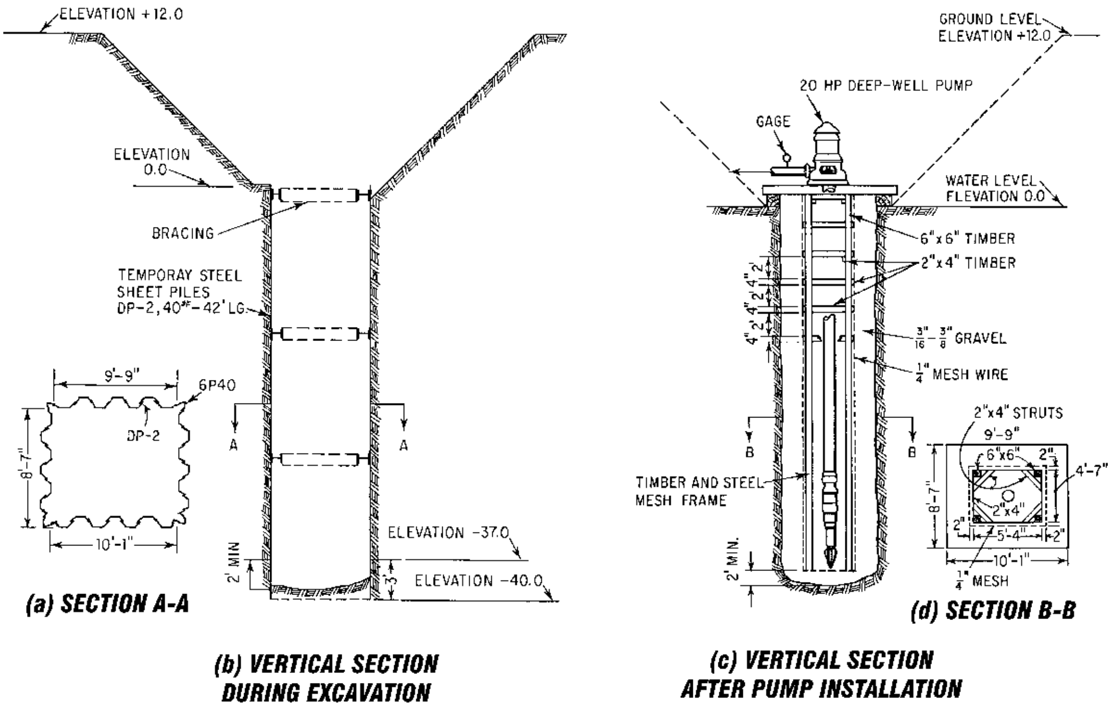

Fig. 7.52 Deep-well installation used at Smithsonian Institution, Washington, D.C. (*Spencer, White & Prentis, Inc.*)

Underpinning

The general methods and main materials used to give additional support at or below grade to structures are called underpinning. Usually, the added support is applied at or near the footings.

7.31 Underpinning Procedures

Underpinning may be remedial or precautionary. Remedial underpinning adds foundation capacity to an inadequately supported structure. Precautionary underpinning is provided to obtain adequate foundation capacity to sustain higher loads, as a safeguard against possible settlement during adjacent excavating, or to accommodate changes in ground conditions. Usually, this type of underpinning is required for the foundations of a structure when deeper foundations are to be constructed nearby for an addition or another structure. Loss of ground, even though small, into an adjoining excavation may cause excessive settlement of existing foundations.

Presumably, an excavation influences an existing substructure when a plane through the outermost foundations, on a 1-on-1 slope for sand or a 1-on-2 slope for unconsolidated silt or soft clay, penetrates the excavation. For a cohesionless soil, underpinning exterior walls within a 1-on-1 slope usually suffices; interior columns are not likely to be affected if farther from the edge of the excavation than half the depth of the cut.

The commonly accepted procedures of structural and foundation design should be used for underpinning. Data for computing dead loads may be obtained from plans of the structure or a field survey. Since underpinning is applied to existing structures, some of which may be old, engineers in charge of underpinning design and construction should be familiar with older types of construction as well as the most modern.

Before underpinning starts, the engineers should investigate and record existing defects in the structure. Preferably, the engineers should be

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accompanied in this investigation by a representative of the owner. The structure should be thoroughly inspected, from top to bottom, inside (if possible) and out. The report should include names of inspectors, dates of inspection, and description and location of defects. Photographs are useful in verifying written descriptions of damaged areas. The engineers should mark existing cracks in such a way that future observations would indicate whether they are continuing to open or spread.

Underpinning generally is accompanied by some settlement. If design and field work are good, the settlement may be limited to about $\frac{1}{4}$ to $\frac{3}{8}$ in. But as long as settlement is uniform in a structure, damage is unlikely. Differential settlement should be avoided. To check on settlement, elevations of critical points, especially columns and walls, should be measured frequently during underpinning. Since movement may also occur laterally, the plumbness of walls and columns also should be checked.

One of the first steps in underpinning usually is digging under a foundation, which decreases its load-carrying capacity temporarily. Hence, preliminary support may be necessary until underpinning is installed. This support may be provided by shores, needles, grillages, and piles. Sometimes, it is desirable to leave them in place as permanent supports.

Generally, it is advisable to keep preliminary supports at a minimum, for economy and to avoid interference with other operations. For the purpose, advantage may be taken of arching action and of the ability of a structure to withstand moderate overloads. Also, columns centrally supported on large spread footings need not be shored when digging is along an edge and involves only a small percent of the total footing area. A large part of the column load is supported by the soil directly under the column.

When necessary, weak portions of a structure, especially masonry, should be repaired or strengthened before underpinning starts.

7.32 Shores

Installed vertically or on a slight incline, shores are used to support walls or piers while underpinning pits are dug (Fig. 7.53*a*). Good bearing should be provided at top and bottom of the shores. One way of providing bearing at the top is to cut a niche and mortar a steel bearing plate against the upper face. An alternate to the plate is a Z shape, made by removing diagonally opposite half flanges from an H beam. When the top of the shore is cut to fit between flange and web of the Z, movement of the shore is restrained. For a weak masonry wall, the load may have to be distributed over a larger area. One way of doing this is to insert a few lintel angles about 12 in apart vertically and bolt them to a vertical, heavy timber or steel distributing beam. The horizontal leg of an angle on the beam then transmits the load to a shore.

Inclined shores on only one side of a wall require support at the base for horizontal as well as vertical forces. One way is to brace the shores against an opposite wall at the floor. Preferably, the base of each shore should sit on a footing perpendicular to the axis of the shore. Sized to provide sufficient bearing on the soil, the footing may be made of heavy timbers, steel beams, or reinforced concrete, depending on the load on the shore.

Loads may be transferred to a shore by wedges or jacks. Oak wedges are suitable for light loads; forged steel wedges and bearing plates are desirable for heavy loads. Jacks, however, offer greater flexibility in length adjustments and allow corrections during underpinning for settlement of shore footings.

(H. A. Prentis and L. White, "Underpinning," Columbia University Press, New York; M. J. Tomlinson, "Foundation Design and Construction," Halsted Press, New York.)

7.33 Needles and Grillages

Needles are beams installed horizontally to transfer the load of a wall or column to either or both sides of its foundation, to permit digging of underpinning pits (Fig. 7.53*b*). These beams are more expensive than shores, which transmit the load directly into the ground. Needles usually are steel wide-flange beams, sometimes plate girders, used in pairs, with bolts and pipe spreaders between the beams. This arrangement provides resistance to lateral buckling and torsion. The needles may be prestressed with jacks to eliminate settlement when the load is applied.

The load from steel columns may be transmitted through brackets to the needles. For masonry walls, the needles may be inserted through niches. The

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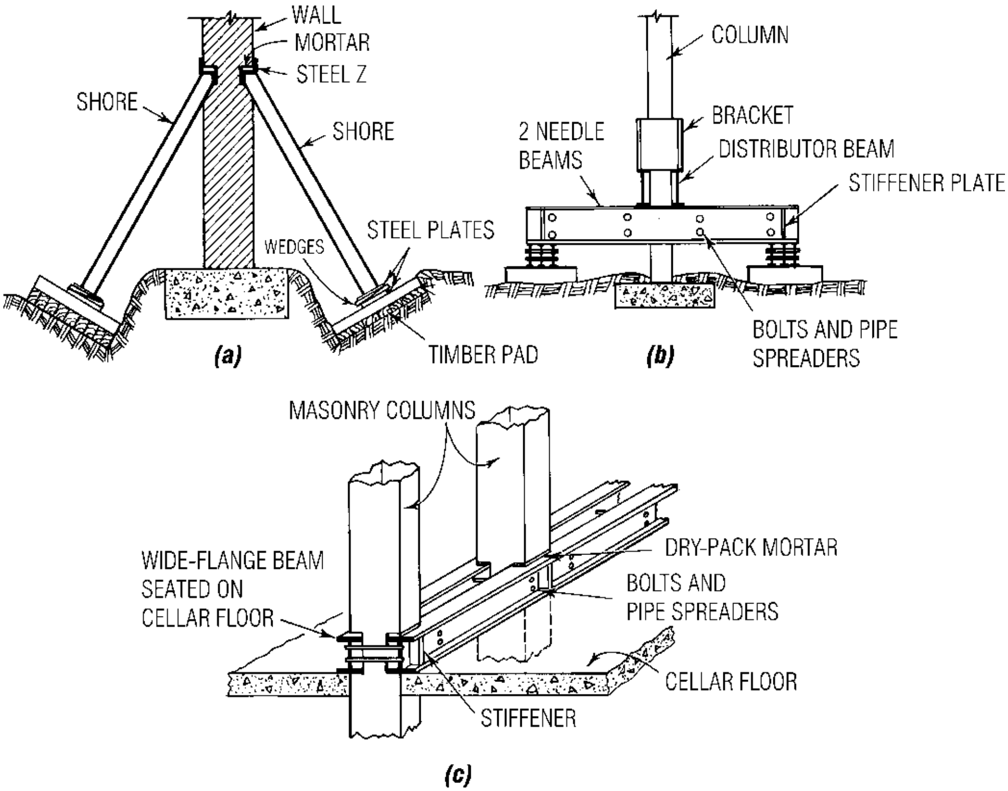

Fig. 7.53 Temporary supports used in underpinning: (*a*) shores; (*b*) needle beams; (*c*) grillage.

load should be transferred from the masonry to the needles through thin wood fillers that crush when the needles deflect and maintain nearly uniform bearing.

Wedges may be placed under the ends of the needles to shift the load from the member to be supported to those beams. The beam ends may be carried on timber pads, which distribute the load over the soil.

Grillages, which have considerably more bearing on the ground than needles, often are used as an alternative to needles and shores for closely spaced columns. A grillage may be installed horizontally on soil at foundation level to support and tie together two or more column footings, or it may rest on a cellar floor (Fig. 7.53*c*). These preliminary supports may consist of two or more steel beams, tied together with bolts and pipe spreaders, or of a steel-concrete composite. Also, grillages sometimes are used to strengthen or repair existing footings by reinforcing them and increasing their bearing area. The grillages may take the form of dowels or of encircling concrete or steel-concrete beams. They should be adequately cross-braced against buckling and torsion. Holes should be made in steel beams to be embedded in concrete, to improve bond.

7.34 Pit Underpinning

After preliminary supports have been installed and weak construction strengthened or repaired, underpinning may start. The most common method of underpinning a foundation is to construct concrete piers down to deeper levels with adequate bearing capacity and to transfer the load to the piers by wedging up with dry packing. To build the piers, pits must be dug under the foundation. Because of the danger of loss of ground and consequent settle-

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ment where soils are saturated, the method usually is restricted to dry subsoil.

When piers have to be placed close together, a continuous wall may be constructed instead. But the underpinning wall should be built in short sections, usually about 5 ft long, to avoid undermining the existing foundation. Alternate sections are built first, and then the gaps are filled in.

Underpinning pits rarely are larger than about 5 ft square in cross section. Minimum size for adequate working room is 3 × 4 ft. Access to the pit is provided by an approach pit started alongside the foundation and extending down about 6 ft. The pits must be carefully sheeted and adequately braced to prevent loss of ground, which can cause settlement of the structure.

In soils other than soft clay, 2-in-thick wood planks installed horizontally may be used to sheet pits up to 5 ft square, regardless of depth. Sides of the pits should be trimmed back no more than absolutely necessary. The boards, usually 2 × 8′s, are installed one at a time with 2-in spaces between

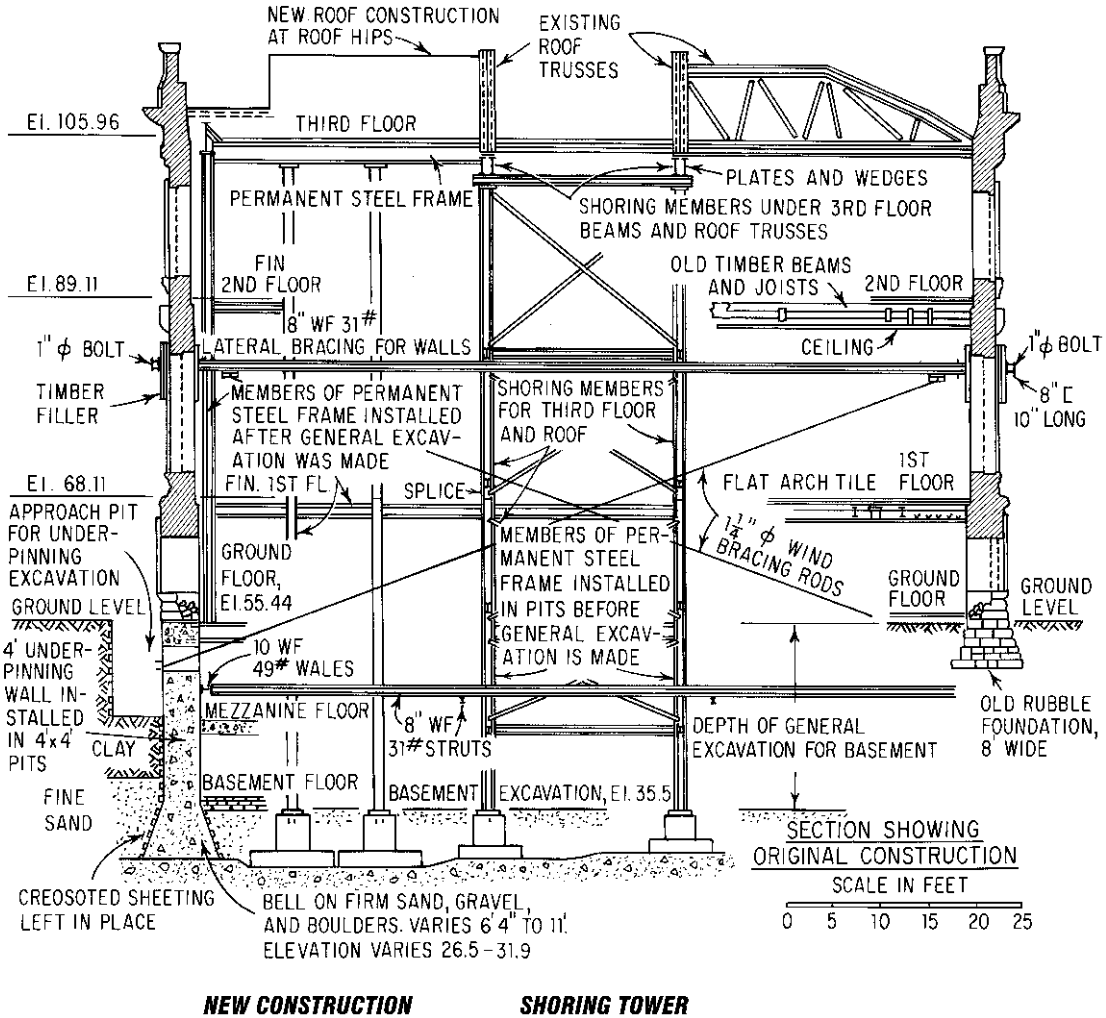

Fig. 7.54 Vertical section through White House, Washington, D.C., during restoration. Pit underpinning was used for the walls. (*Spencer, White & Prentis, Inc.*)

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them vertically. Soil is repacked through these louvers to fill the voids behind the boards. In running sands, hay may be stuffed behind the boards to block the flow. Corners of the sheeting often are nailed with vertical 2 × 4 in wood cleats.

In soft clay, sheeting must be tight and braced against earth pressure. Chicago caissons, or a similar type, may be used (Art. 7.23).

In water-bearing soil with a depth not exceeding about 5 ft, vertical sheeting can sometimes be driven to cut off the water. For the purpose, light steel or tongue-and-groove wood sheeting may be used. The sheeting should be driven below the bottom of the pit a sufficient distance to prevent boiling of the bottom due to hydrostatic pressure. With water cut off, the pit can be pumped dry and excavation continued.

After a pit has been dug to the desired level, it is filled with concrete to within 3 in of the foundation to be supported. The gap is dry-packed, usually by ramming stiff mortar in with a 2 × 4 pounded by an 8-lb hammer. The completed piers should be laterally braced if soil is excavated on one side to a depth of more than about 6 ft. An example of pit underpinning is the work done in the restoration of the White House, in which a cellar and subcellar were created (Fig. 7.54).

7.35 Pile Underpinning

If water-bearing soil more than about 5 ft deep underlies a foundation, the structure may have to be underpinned with piles. Driven piles generally are preferred to jacked piles because of lower cost. The feasibility of driven piles, however, depends on availability of at least 12 ft of headroom and space alongside the foundations. Thus, driven piles often can be used to underpin interior building columns when headroom is available. But they are hard to install for exterior walls unless there is ample space alongside the walls. For very lightly loaded structures, brackets may be attached to underpinning piles to support the structure. But such construction puts bending into the piles, reducing their load-carrying capacity.

Driven piles usually are 12- to 14-in diameter steel pipe, ⅜ in thick. They are driven open-ended, to reduce vibration, and in lengths determined by available headroom. Joints may be made with cast-steel sleeves. After soil has been removed from the pipe interior, it is filled with concrete.

Jacked piles require less headroom and may be placed under a footing. Also made of steel pipe installed open-ended, these piles are forced down by hydraulic jacks reacting against the footing. The operation requires an approach pit under the footing to obtain about 6 ft of headroom.

Pretest piles, originally patented by Spencer, White & Prentis, New York City, are used to prevent the rebound of piles when jacking stops and subsequent settlement when the load of the structure is transferred to the piles. A pipe pile is jacked down, in 4-ft lengths, to the desired depth. The hydraulic jack reacts against a steel plate mortared to the underside of the footing to be supported. After the pile has been driven to the required depth and cleaned out, it is filled with concrete and capped with a steel bearing plate. Then two hydraulic jacks atop the pile overload it 50%. As the load is applied, a bulb of pressure builds up in the soil at the pile bottom. This pressure stops downward movement of the pile. While the jacks maintain the load, a short length of beam is wedged between the pile top and the steel plate under the footing. Then, the jacks are unloaded and removed. The load, thus, is transferred without further settlement. Later the space under the footing is concreted. Figure 7.55 shows how pretest piles were used for underpinning existing structures during construction of a subway in New York City.

7.36 Miscellaneous Underpinning Methods

Spread footings may be pretested in much the same way as piles. The weight of the structure is used to jack down the footings, which then are wedged in place, and the gap is concreted. The method may be resorted to for unconsolidated soils where a high water table makes digging under a footing unsafe or where a firm stratum is deep down.

A form of underpinning may be used for slabs on ground. When concrete slabs settle, they may be restored to the proper elevation by mud jacking. In this method, which will not prevent future settlement, a fluid grout is pumped under the slab through holes in it, raising it. Pressure is maintained until the grout sets. The method also may be used to fill voids under a slab.

Chemical or thermal stabilization (Arts. 7.38 and 7.39) sometimes may be used as underpinning.

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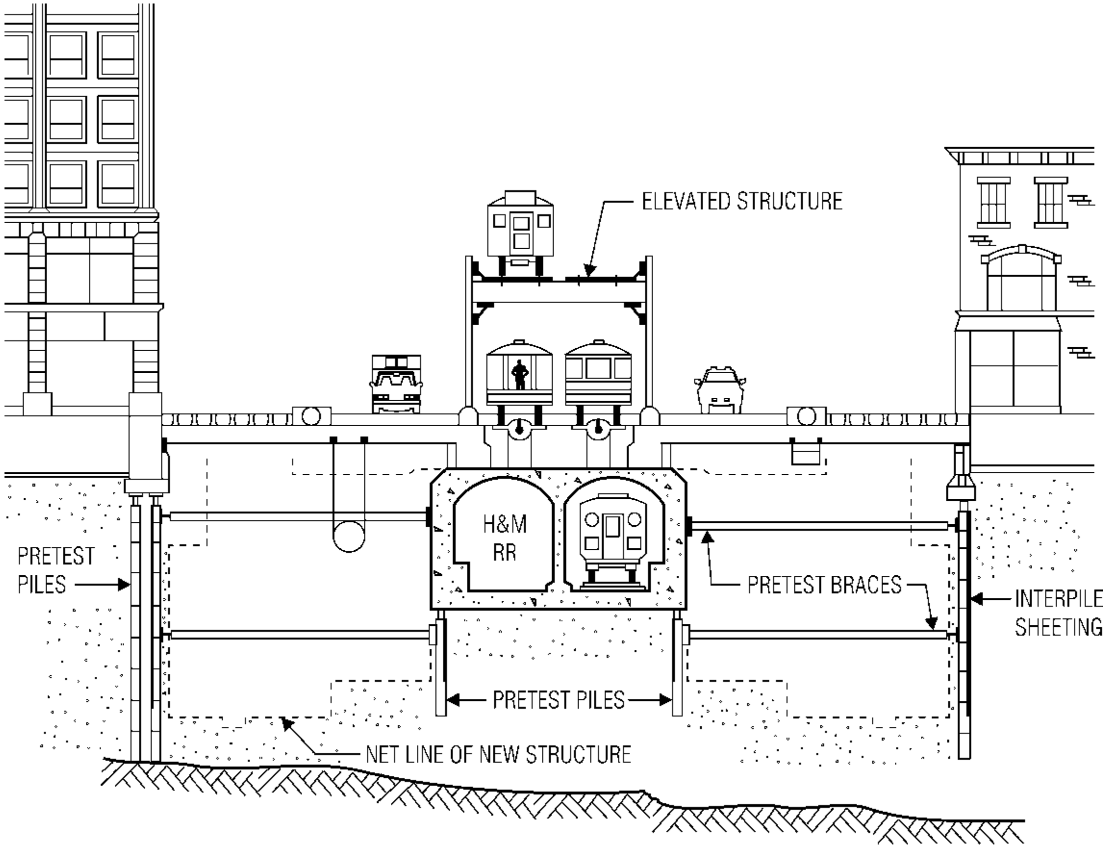

Fig. 7.55 Pretest piles support existing buildings, old elevated railway, and a tunnel during construction of a subway. (*Spencer, White & Prentis, Inc.*)

Ground Improvement

Soil for foundations can be altered to conform to desired characteristics. Whether this should be done depends on the cost of alternatives.

Investigations of soil and groundwater conditions on a site should indicate whether soil improvement, or stabilization, is needed. Tests may be necessary to determine which of several applicable techniques may be feasible and economical. Table 7.19 lists some conditions for which soil improvement should be considered and the methods that may be used.

As indicated in the table, ground improvement may increase strength, increase or decrease permeability, reduce compressibility, improve stability, or decrease heave due to frost or swelling. The main techniques used are constructed fills, replacement of unsuitable soils, surcharges, reinforcement, mechanical stabilization, thermal stabilization, and chemical stabilization. (Federal Highway Administration "Ground Improvement Technical Summaries" Publication No. FHWA-SA-98-086, December 1999).

7.37 Mechanical Stabilization of Soils

This comprises a variety of techniques for rearranging, adding, or removing soil particles. The objective usually is to increase soil density, decrease water content, or improve gradation. Particles may be rearranged by blending the layers of a stratified soil, remolding an undisturbed soil, or densifying a soil. Sometimes, the desired improvement can be obtained by drainage alone.

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Table 7.19 Where Soil Improvement May Be Economical

Soil Deficiency	Probable Type of Failure	Probable Cause	Possible Remedies
Slope instability	Slides on slope	Pore-water pressure	Drain; flatten slope; freeze
		Loose granular soil	Compact
		Weak soil	Mix or replace with select material
	Mud flow	Excessive water	Exclude water
	Slides—movement at toe	Toe instability	Place toe fill, and drain
Low bearing capacity	Excessive settlement	Saturated clay	Consolidate with surcharge, and drain
		Loose granular soil	Compact; drain; increase footing depth; mix with chemicals
		Weak soil	Superimpose thick fill; mix or replace with select material; inject or mix with chemicals; freeze (if saturated); fuse with heat (if unsaturated)
Heave	Excessive rise	Frost	For buildings: place foundations below frost line; insulate refrigeration-room floors; refrigerate to keep ground frozen For roads: Remove fines from gravel; replace with nonsusceptible soil
		Expansion of clay	Exclude water; replace with granular soil
Excessive Permeability	Seepage	Pervious soil or fissured rock	Mix or replace soil with select material; inject or mix soil with chemicals; construct cutoff wall with grout; enclose with sheetpiles and drain
"Quick" bottom	Loss of strength	Flow under	Add berm against cofferdam inner face; increase width of cofferdam between lines of sheeting; drain with wellpoints outside the cofferdam

Often, however, compactive effort plus water control is needed.

7.37.1 Embankments

Earth often has to be placed over the existing ground surface to level or raise it. Such constructed fills may create undesirable conditions because of improper compaction, volume changes, and unexpected settlement under the weight of the fill. To prevent such conditions, fill materials and their gradation, placement, degree of compaction, and thickness should be suitable for properly supporting the expected loads.

Fills may be either placed dry with conventional earthmoving equipment and techniques or wet by hydraulic dredges. Wet fills are used mainly for filling behind bulkheads or for large fills.

A variety of soils and grain sizes are suitable for topping fills for most purposes. Inclusion of organic matter or refuse should, however, be prohibited. Economics usually require that the source

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of fill material be as close as possible to the site. For most fills, soil particles in the 18 in below foundations, slabs, or the ground surface should not be larger than 3 in in any dimension.

For determining the suitability of a soil as fill and for providing a standard for compaction, the moisture-density relationship test, or Proctor test (ASTM D698 and D1557), is often used. Several of these laboratory tests should be performed on the borrowed material, to establish moisture-density curves. The peak of a curve indicates the maximum density achievable in the laboratory by the test method as well as the optimum moisture content. ASTM D1557 should be used as the standard when high bearing capacity and low compressibility are required; ASTM D698 should be used when requirements are lower, for example, for fills under parking lots.

The two ASTM tests represent different levels of compactive effort. But much higher compactive effort may be employed in the field than that used in the laboratory. Thus, a different moisture-density relationship may be produced on the site. Proctor test results, therefore, should not be considered an inherent property of the soil. Nevertheless, the test results indicate the proposed fill material's sensitivity to moisture content and the degree of field control that may be required to obtain the specified density.

See also Art. 7.40.

7.37.2 Fill Compaction

The degree of compaction required for a fill is usually specified as a minimum percentage of the maximum dry density obtained in the laboratory tests. This compaction is required to be accomplished within a specific moisture range. Minimum densities of 90 to 95% of the maximum density are suitable for most fills. Under roadways, footings, or other highly loaded areas, however, 100% compaction is often required. In addition, moisture content within 2 to 4% of the optimum moisture content usually is specified.

Field densities can be greater than 100% of the maximum density obtained in the laboratory test. Also, with greater compactive effort, such densities can be achieved with moisture contents that do not lie on the curves plotted from laboratory results. (Fine-grained soils should not be overcompacted on the dry side of optimum because when they get wet, they may swell and soften significantly.)

For most projects, lift thickness should be restricted to 8 to 12 in, each lift being compacted before the next lift is placed. On large projects where heavy compaction equipment is used, a lift thickness of 18 to 24 in is appropriate.

Compaction achieved in the field should be determined by performing field density tests on each lift. For that purpose, wet density and moisture content should be measured and the dry density computed. Field densities may be ascertained by the sand-cone (ASTM D1556) or balloon volume-meter (ASTM D2167) method, from an undisturbed sample, or with a nuclear moisture-density meter. Generally, one field density test for each 4000 to 10,000 ft^2 of lift surface is adequate.

Hydraulically placed fills composed of dredged soils normally need not be compacted during placement. Although segregation of the silt and clay fractions of the soils may occur, it usually is not harmful. But accumulation of the fine-grained material in pockets at bulkheads or under structures should be prevented. For the purpose, internal dikes, weirs, or decanting techniques may be used.

7.37.3 Soil Replacement or Blending

When materials at or near grade are unsuitable, it may be economical to remove them and substitute a fill of suitable soil, as described in Art. 7.37.1. When this is not economical, consideration should be given to improving the soil by other methods, such as densification or addition or removal of soil particles.

Mixing an existing soil with select materials or removing selected sizes of particles from an existing soil can change its properties considerably. Adding clay to a cohesionless soil in a nonfrost region, for example, may make the soil suitable as a base course for a road (if drainage is not too greatly impaired). Adding clay to a pervious soil may reduce its permeability sufficiently to permit its use as a reservoir bottom. Washing particles finer than 0.02 mm from gravel makes the soil less susceptible to frost heave (desirable upper limit for this fraction is 3%).

7.37.4 Surcharges

Where good soils are underlain by soft, compressible clays that would permit unacceptable settlement, the site often can be made usable by

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surcharging, or preloading, the surface. The objective is to use the weight of the surcharge to consolidate the underlying clays. This offsets the settlement of the completed structure that would otherwise occur. A concurrent objective may be to increase the strength of the underlying clays.

If the soft clay is overlain by soils with adequate bearing capacity, the area to be improved may be loaded with loose, dumped earth, until the weight of the surcharge is equivalent to the load that later will be imposed by the completed structure. (If highly plastic clays or thick layers with little internal drainage are present, it may be necessary to insert vertical drains to achieve consolidation within a reasonable time.) During and after placement of the surcharge, settlement of the original ground surface and the clay layer should be closely monitored. The surcharge may be removed after little or no settlement is observed. If surcharging has been properly executed, the completed structure should experience no further settlement due to primary consolidation. Potential settlement due to secondary compression, however, should be evaluated, especially if the soft soils are highly organic.

7.37.5 Densification

Any of a variety of techniques, most involving some form of vibration, may be used for soil densification. The density achieved with a specific technique, however, depends on the grain size of the soil. Consequently, grain-size distribution must be taken into account when selecting a densification method.

Compaction of clean sands to depths of about 6 ft usually can be achieved by rolling the surface with a heavy, vibratory, steel-drum roller. Although the vibration frequency is to some extent adjustable, the frequencies most effective are in the range of 25 to 30 Hz. Bear in mind, however, that little densification will be achieved below a depth of 6 ft, and the soil within about 1 ft of the surface may actually be loosened. Compactive effort in the field may be measured by the number of passes made with a specific machine of given weight and at a given speed. For a given compactive effort, density varies with moisture content. For a given moisture content, increasing the compactive effort increases the soil density and reduces the permeability.

Compaction piles also may be used to densify sands. For that purpose, the piles usually are made of wood. Densification of the surrounding soils results from soil displacement during driving of the pile or shell and from the vibration produced during pile driving. The foundations to be constructed need not bear directly on the compaction piles but may be seated anywhere on the densified mass.

Vibroflotation and Terra-Probe are alternative methods that increase sand density by multiple insertions of vibrating probes. These form cylindrical voids, which are then filled with offsite sand, stone, or blast-furnace slag. The probes usually are inserted in clusters, with typical spacing of about 4½ ft, where footings will be placed. Relative densities of 85% or more can be achieved throughout the depth of insertion, which may exceed 40 ft. Use of vibrating probes may not be effective, however, if the fines content of the soil exceeds about 15% or if organic matter is present in colloidal form in quantities exceeding about 5% by weight.

Another technique for densification is dynamic compaction, which in effect subjects the site to numerous mini-earthquakes. In saturated soils, densification by this method also results from partial liquefaction, and the elevated pore pressures produced must be dissipated between each application of compactive energy if the following application is to be effective. As developed by Techniques Louis Menard, dynamic compaction is achieved by dropping weights ranging from 10 to 40 tons from heights up to 100 ft onto the ground surface. Impact spacings range up to 60 ft. Multiple drops are made at each location to be densified. This technique is applicable to densification of large areas and a wide range of grain sizes and materials.

7.37.6 Drainage

This is effective in soil stabilization because strength of a soil generally decreases with an increase in amount and pressure of pore water. Drainage may be accomplished by gravity, pumping, compression with an external load on the soil, electro-osmosis, heating, or freezing.

Pumping often is used for draining the bottom of excavations (Art. 7.30). For slopes, however, advantage must be taken of gravity flow to attain permanent stabilization. Vertical wells may be used to relieve artesian pressures. Usually,

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intercepting drains, laid approximately along contours, suffice.

Where mud flows may occur, water must be excluded from the area. Surface and subsurface flow must be intercepted and conducted away at the top of the area. Also, cover, such as heavy mulching and planting, should be placed over the entire surface to prevent water from percolating downward into the soil. See also Art. 7.40.

Electrical drainage adapts the principle that water flows to the cathode when a direct current passes through saturated soil. The water may be pumped out at the cathode. Electro-osmosis is relatively expensive and therefore usually is limited to special conditions, such as drainage of silts, which ordinarily are hard to drain by other methods.

Vertical drains, or piles, may be used to compact loose, saturated cohesionless soils or to consolidate saturated cohesive soils. They provide an escape channel for water squeezed out of the soil by an external load. A surcharge of pervious material placed over the ground surface also serves as part of the drainage system as well as part of the fill, or external load. Usually, the surcharge is placed before the vertical drains are installed, to support equipment, such as pile drivers, over the soft soil. Fill should be placed in thin layers to avoid formation of mud flows, which might shear the sand drains and cause mud waves. Analyses should be made of embankment stability at various stages of construction.

7.38 Thermal Stabilization of Soils

Thermal stabilization generally is costly and is restricted to conditions for which other methods are not suitable. Heat may be used to strengthen nonsaturated loess and to decrease the compressibility of cohesive soils. One technique is to burn liquid or gas fuel in a borehole.

Freezing a wet soil converts it into a rigid material with considerable strength, but it must be kept frozen. The method is excellent for a limited excavation area, for example, freezing the ground to sink a shaft. For the purpose, a network of pipes is placed in the ground and a liquid, usually brine, at low temperature is circulated through the pipes. Care must be taken that the freezing does not spread beyond the area to be stabilized and cause heaving damage.

7.39 Chemical Stabilization of Soils

Utilizing, portland cement, bitumens, or other cementitous materials, chemical stabilization meets many needs. In surface treatments, it supplements mechanical stabilization to make the effects more lasting. In subsurface treatments, chemicals may be used to improve bearing capacity or decrease permeability.

Soil-cement, a mixture of portland cement and soil, is suitable for subgrades, base courses, and pavements of roads not carrying heavy traffic ("Essentials of Soil-Cement Construction," Portland Cement Association). Bitumen-soil mixtures are extensively used in road and airfield construction and sometimes as a seal for earth dikes ("Guide Specifications for Highway Construction," American Association of State Highway and Transportation Officials, 444 North Capitol St., N.W., Washington, DC 20001 (www.ashto.org)). Hydrated, or slaked, lime may be used alone as a soil stabilizer, or with fly ash, portland cement, or bitumen ("Lime Stabilization of Roads," National Lime Association, 200 North Globe Road, Suite 800 Arlington, VA 22203 (www.lime.org)). Calcium or sodium chloride is used as a dust palliative and an additive in construction of granular base and wearing courses for roads ("Calcium Chloride for Stabilization of Bases and Wearing Courses," Calcium Chloride Institute).

Grouting, with portland cement or other chemicals, often is used to fill rock fissures, decrease soil permeability, form underground cutoff walls to eliminate seepage, and stabilize soils at considerable depth. The chemicals may be used to fill the voids in the soil, to cement the particles, or to form a rocklike material.

(K. Terzaghi and R. B. Peck, "Soil Mechanics in Engineering Practice," John Wiley & Sons, Inc., New York (www.wiley.com); G. P. Tschebotarioff, "Soil Mechanics, Foundations, and Earth Structures," McGraw-Hill Book Company, New York (books.mcgraw-hill.com); H. Y. Fang, "Foundation Engineering Handbook," 2nd ed., Van Nostrand Reinhold Company, New York.)

7.40 Geosynthetics

In the past, many different materials have been used for soil separation or reinforcement, including

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grasses, rushes, wood logs, wood boards, metal mats, cotton, and jute. Because they deteriorated in a relatively short time, required maintenance frequently, or were costly, however, use of more efficient, more permanent materials was desirable. Synthetic fabrics, grids, nets, and other structures are now used as an alternative.

Types of geosynthetics, polymer compositions generally used, and properties important for specifying materials to achieve desired performance are described in Art. 5.29. Principal applications of geosynthetics, functions of geosynthetics in those applications, recommended structures for each case, and design methods are discussed in the following. Table 7.20 summarizes the primary functions of geosynthetics in applications often used and indicates the type of geosynthetic generally recommended by the manufacturers of these materials for the applications. ("Geosynthetic Design and Construction Guidelines" Federal Highway Administration Publication No. FHWA-HI-95-038, May, 1995).

7.40.1 Design Methods for Geosynthetics

The most commonly used design methods for geosynthetics in geotechnical applications are the empirical (design by experience), specification, and rational (design by function) methods.

The empirical design process employs a selection process based on the experience of the project geo-technical engineer or of others, such as designers of projects reported in engineering literature, manufacturers of geosynthetics, and professional associations.

Design by specification is often used for routine applications. Standard specifications for specific applications may be available from geosynthetics manufacturers or developed by an engineering organization or a government agency for its own use or by an association or group of associations, such as the joint committee established by the American Association of State Highway and Transportation Officials, Associated General Contractors, and American Road and Transportation Builders Association (Art. 5.29).

When using the rational design method, designers evaluate the performance, construction methods required, and durability under service conditions of geosynthetics that are suitable for the planned application. This method can be used for all site conditions to augment the preceding methods. It is necessary for applications not covered by standard specifications. It also is required for projects of such nature that large property damage or personal injury would result if a failure should occur. The method requires the following:

A decision as to the primary function of a geosynthetic in the application under consideration

Table 7.20 Primary Function of Geosynthetics in Geotechnical Applications

Application	Function	Geosynthetic
Subgrade stabilization	Reinforcement, separation, filtration	Geotextile or geogrid
Railway trackbed stabilization	Drainage, separation, filtration	Geotextile
Sedimentation-control silt fence	Sediment retention, filtration, separation	Geotextile
Asphalt overlays	Stress-relieving layer and waterproofing	Geotextile
Soil reinforcement:		
Embankments	Reinforcement	Geotextile or geogrid
Steep slopes	Reinforcement	Geotextile or geogrid
Retaining walls	Reinforcement	Geotextile or geogrid
Erosion control:		
Reinforcement	Reinforcement, separation	Geocomposite
Riprap	Filtration and separation	Geotextile
Mats	Filtration and separation	Geotextile
Subsurface drainage filter	Filtration	Geotextile
Geomembrane protection	Protection and cushion	Geotextile
Subsurface drainage	Fluid transmission and filtration	Prefabricated drainage composite

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Estimates or calculations to establish the required properties (design values) of the material for the primary function

Determination of the allowable properties, such as minimum tensile or tear strength or permittivity, of the material by tests or other reliable means

Calculation of the safety factor as the ratio of allowable to design values

Assessment of this result to ascertain that it is sufficiently high for site conditions

("A Design Primer: Geotextiles and Related Materials," Industrial Fabric Association International, 345 Cedar St., Suite 800, St. Paul, MN 55101; "Geotextile Testing and the Design Engineer," STP 952, ASTM; R. M. Koerner, "Designing with Geosynthetics," 2nd ed., Prentice-Hall, Englewood Cliffs, N. J.)

7.40.2 Geosynthetics Nomenclature

Following are some of the terms generally used in design and construction with geosynthetics:

Apparent Opening Size (AOS). A property designated O_{95} applicable to a specific geotextile that indicates the approximate diameter of the largest particle that would pass through the geotextile. A minimum of 95% of the openings are the same size or smaller than that particle, as measured by the dry sieve test specified in ASTM D4751.

Blinding. Blocking by soil particles of openings in a geotextile, as a result of which its hydraulic conductivity is reduced.

Chemical Stability. Resistance of a geosynthetic to degradation from chemicals and chemical reactions, including those catalyzed by light.

Clogging. Retention of soil particles in the voids of a geotextile, with consequent reduction in the hydraulic conductivity of the fabric.

Cross-Machine Direction. The direction within the plane of a fabric perpendicular to the direction of manufacture. Generally, tensile strength of the fabric is lower in this direction than in the machine direction.

Denier. Mass, g, of a 9000-m length of yarn.

Fabric. Polymer fibers or yarn formed into a sheet with thickness so small relative to dimensions in the plane of the sheet that it cannot resist compressive forces acting in the plane. A needle-punched fabric has staple fibers or filaments mechanically bonded with the use of barbed needles to form a compact structure. A spun-bonded fabric is formed with continuous filaments that have been spun (extruded), drawn, laid into a web, and bonded together in a continuous process, chemically, mechanically, or thermally. A woven fabric is produced by interlacing orthogonally two or more sets of elements, such as yarns, fibers, rovings, or filaments, with one set of elements in the machine direction. A monofilament woven fabric is made with single continuous filaments, whereas a multifilament woven fabric is composed of bundles of continuous filaments. A split-film woven fabric is constructed of yarns formed by splitting longitudinally a polymeric film to form a slit-tape yarn. A nonwoven fabric is produced by bonding or interlocking of fibers, or both.

Fiber. Basic element of a woven or knitted fabric with a length-diameter or length-width ratio of at least 100 and that can be spun into yarn or otherwise made into a fabric.

Filament. Variety of fiber of extreme length, not readily measured.

Filtration. Removal of particles from a fluid or retention of soil particles in place by a geosynthetic, which allows water or other fluids to pass through.

Geocomposite. Manufactured laminated or composite material composed of geotextiles, geomembranes, or geogrids, and sometimes also natural materials, or a combination.

Geogrid. Orthogonally arranged fibers, strands, or rods connected at intersections, intended for use primarily as tensile reinforcement of soil or rock.

Geomembrane. Geosynthetic, impermeable or nearly so, intended for use in geotechnical applications.

Geosynthetics. Materials composed of polymers used in geotechnical applications.

Geotextile. Fabric composed of a polymer and used in geotechnical applications.

Grab Tensile Strength. Tensile strength determined in accordance with ASTM D4632 and typically found from a test on a 4-in-wide strip of fabric, with the tensile load applied at the midpoint of the fabric width through 1-in-wide jaw faces.

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Gradient Ratio. As measured in a constant-head permittivity test on a geotextile, the ratio of the average hydraulic gradient across the fabric plus 1 in of soil adjoining the fabric to the average hydraulic gradient of the 2 in of soil between 1 and 3 in above the fabric.

Machine Direction. The direction in the plane of the fabric parallel to the direction of manufacture. Generally, the tensile strength of the fabric is largest in this direction.

Monofilament. Single filament, usually of a denier higher than 15.

Mullen Burst Strength. Hydraulic bursting strength of a geotextile as determined in accordance with ASTM D3786.

Permeability (Hydraulic Conductivity). A measure of the capacity of a geosynthetic to allow a fluid to move through its voids or interstices, as represented by the amount of fluid that passes through the material in a unit time per unit surface area under a unit pressure gradient. Accordingly, permeability is directly proportional to thickness of the geosynthetic.

Permittivity. Like permeability, a measure of the capacity of a geosynthetic to allow a fluid to move through its voids or interstices, as represented by the amount of fluid that passes through a unit surface area of the material in a unit time per unit thickness under a unit pressure gradient, with laminar flow in the direction of the thickness of the material. For evaluation of geotextiles, use of permittivity, being independent of thickness, is preferred to permeability.

Puncture Strength. Ability of a geotextile to resist puncture as measured in accordance with ASTM D3787.

Separation. Function of a geosynthetic to prevent mixing of two adjoining materials.

Soil-Fabric Friction. Resistance of soil by friction to sliding of a fabric embedded in it, exclusive of resistance from cohesion. It is usually expressed as a friction angle.

Staple Fibers. As usually used in geotextiles, very short fibers, typically 1 to 3 in long.

Survivability. Ability of geosynthetics to perform intended functions without impairment.

Tearing Strength. Force required either to start or continue propagation of a tear in a fabric as determined in accordance with ASTM D4533.

Tenacity. Fiber strength, grams per denier.

Tex. Denier divided by 9.

Transmissivity. Amount of fluid that passes in unit time under unit pressure gradient with laminar flow per unit thickness through a geosynthetic in the in-plane direction.

Yarn. Continuous strand composed of textile fibers, filaments, or material in a form suitable for knitting, weaving, or otherwise intertwining to form a geotextile.

7.40.3 Geosynthetic Reinforcement of Steep Slopes

Geotextiles and geogrids are used to reinforce soils to permit slopes much steeper than the shearing resistance of the soils will permit. (**Angle of repose**, the angle between the horizontal and the maximum slope that a soil assumes through natural processes, is sometimes used as a measure of the limiting slopes for unconfined or unreinforced cuts and fills, but it is not always relevant. For dry, cohesionless soils, the effect of height of slope on this angle is negligible. For cohesive soils, in contrast, the height effect is so large that angle of repose is meaningless.) When geosynthetic reinforcement is used, it is placed in the fill in horizontal layers. Vertical spacing, embedment length, and tensile strength of the geosynthetic are critical in establishing a stable soil mass.

For evaluation of slope stability, potential failure surfaces are assumed, usually circular or wedge-shaped but other shapes also are possible. Figure 7.56*a* shows a slope for which a circular failure surface starting at the bottom of the slope and extending to the ground surface at the top is assumed. An additional circular failure surface is indicated in Fig. 7.56*b*. Fig. 7.56*c* shows a wedge-shaped failure surface. An infinite number of such failure surfaces are possible. For design of the reinforcement, the surfaces are assumed to pass through a layer of reinforcement at various levels and apply tensile forces to the reinforcement, which must have sufficient tensile strength to resist them. Sufficient reinforcement

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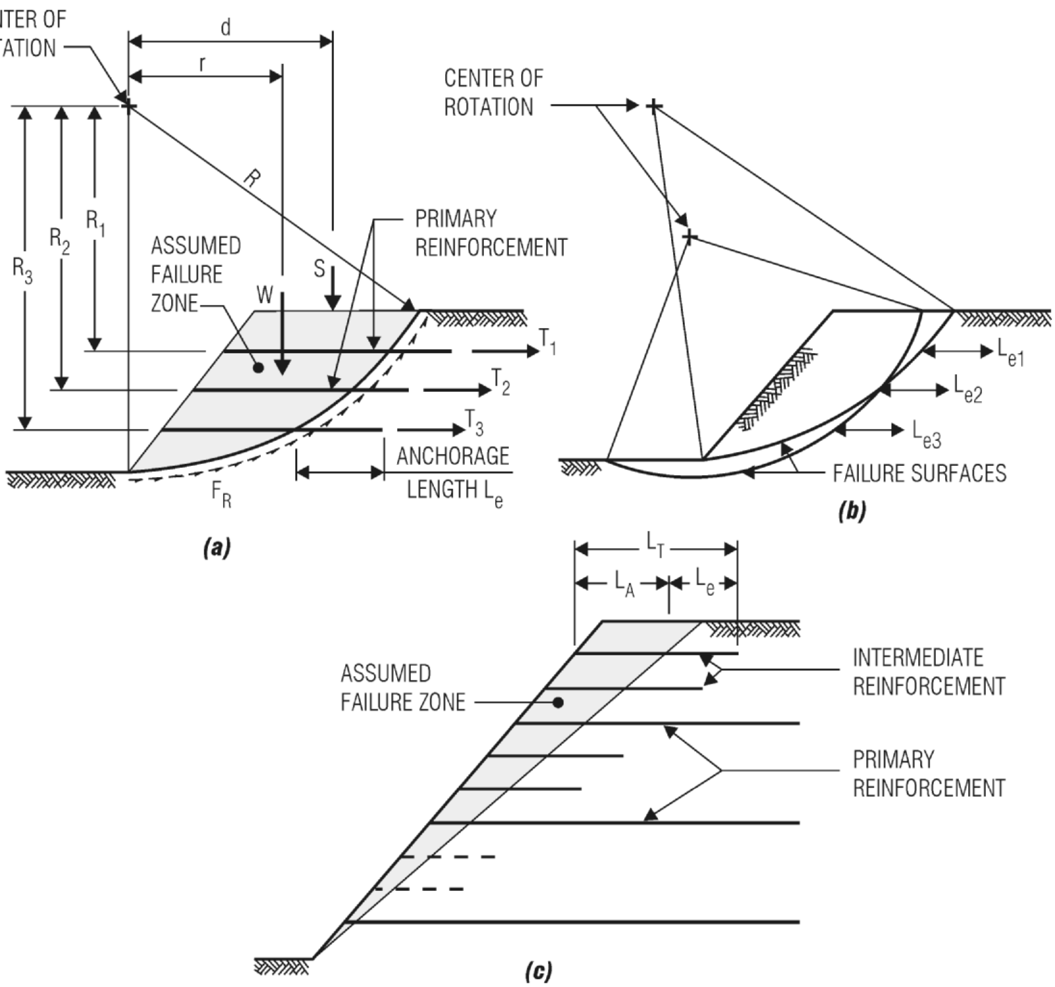

Fig. 7.56 Stabilization of a steep slope with horizontal layers of geosynthetic reinforcement. (*a*) Primary reinforcement for a circular failure surface. (*b*) Embedment lengths of reinforcement extend from critical failure surfaces into the backfill. (*c*) Intermediate reinforcement for shallow failure surfaces.

embedment lengths extending into stable soil behind the surfaces must be provided to ensure that the geosynthetic will not pull out at design loads.

Pull-out is resisted by geotextiles mainly by friction or adhesion—and by geogrids, which have significant open areas, also by soil-particle *strike-through*. The soil-fabric interaction is determined in laboratory pull-out tests on site-specific soils and the geosynthetic to be used, but long-term load-transfer effects may have to be estimated. Design of the reinforcement requires calculation of the embedment required to develop the reinforcement fully and of the total resisting force (number of layers and design strength) to be provided by the reinforcement. The design should be based on safety factors equal to or greater than those required by local design codes. In the absence of local code requirements, the values given in Table 7.21 may be used. A stability analysis should be performed to investigate, at a minimum, circular and wedge-shaped failure surfaces through the toe (Fig. 7.56*a*), face (Fig. 7.56*c*), and deep seated below the toe (Fig. 7.56*b*). The total resisting moment for a

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Table 7.21 Minimum Safety Factors K for Slope Reinforcement

External Stability		Internal Stability	
Condition	K	Condition	K
Sliding	1.5	Slope stability	1.3
Deep seated (overall stability)	1.3	Design tensile strength T_d	*
Dynamic loading	1.1	Allowable geosynthetic strength T_a†	
		Creep	4
		Construction	1.1 to 1.3
		Durability	1.1 to 1.2
		Pull-out resistance	
		Cohesionless soils	1.5‡
		Cohesive soils	2

* T_d at 5% strain should be less than T_a.

† $T_a = T_L/K_cK_d$, where T_L is the creep limit strength, K_c is the safety factor for construction, and K_d is the safety factor for durability. In the absence of creep tests or other pertinent data, the following may be used: $T_a = T_u/10.4$ or $T_u \geq 10.4T_d$, where T_u is the ultimate tensile strength of the geosynthetic.

‡ For a 3-ft minimum embedment.

circular slip surface can be determined from Fig. 7.56*b* as

$$M_R = RF_r + \sum_{i=1}^{i=n} R_iT_i \tag{7.90}$$

where R = radius of failure circle

F_R = soil shearing resistance along the slip surface = $\tau_f L_{sp}$

τ_f = soil shear strength

L_{sp} = length of slip surface

R_i = radius of slip surface at layer i

T_i = strength of the reinforcing required for layer i

The driving moment, or moment of the forces causing slip, is

$$M_D = Wr + Sd \tag{7.91}$$

where W = weight of soil included in assumed failure zone (Fig. 7.56*a*)

r = moment arm of W with respect to the center of rotation (Fig. 7.56*a*)

S = surcharge

d = moment arm of S with respect to the center of rotation (Fig. 7.56*a*)

The safety factor for the assumed circular failure surface then is

$$K_D = \frac{M_R}{M_D} \tag{7.92}$$

A safety factor should be computed for each potential failure surface. If a safety factor is less than the required minimum safety factor for prevention of failure of the soil unreinforced, either a stronger reinforcement is required or the number of layers of reinforcement should be increased. This procedure may also be used to determine the reinforcement needed at any level to prevent failure above that layer.

The next step is calculation of length L_e of reinforcement required for anchorage to prevent pullout.

$$L_e = \frac{KF_D}{2\sigma_o \tan \phi_{sr}} \tag{7.93}$$

where F_D = required pull-out strength

K = minimum safety factor: 1.5 for cohesionless soils; 2 for cohesive soils

σ_o = overburden pressure above the reinforcing level = wh

w = density of the soil

h = depth of overburden

ϕ_{sr} = soil-reinforcement interaction angle, determined from pull-out tests

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Embedment length L_e should be at least 3 ft. The total length of a reinforcement layer then is L_e plus the distance from the face of the slope to the failure circle (Fig. 7.56*b*). The total length of the reinforcement at the toe should be checked to ascertain that it is sufficient to resist sliding of the soil mass above the base of the slope.

Among the family of potential failure surfaces that should be investigated is the wedge shape, such as the one shown in Fig. 7.56*c*. To reinforce the failure zones close to the face of the slope, layers of reinforcement are required in addition to those provided for the deep failure zones, as indicated in Fig. 7.56*c*. Such face reinforcement should have a maximum vertical spacing of 18 in and a minimum length of 4 ft. Inasmuch as the tension in this reinforcement is limited by the short embedment, a geosynthetic with a smaller design allowable tension than that required for deep-failure reinforcement may be used. In construction of the reinforced slope, fill materials should be placed so that at least 4 in of cover will be between the geosynthetic reinforcement and vehicles or equipment operating on a lift. Backfill should not incorporate particles larger than 3 in. Turning of vehicles on the first lift above the geosynthetic should not be permitted. Also, end dumping of fill directly on the geosynthetic should not be allowed.

7.40.4 Geosynthetics in Retaining-Wall Construction

Geotextiles and geogrids are used to form retaining walls (Fig. 7.57*a*) or to reinforce the backfill of a retaining wall to create a stable soil mass (Fig. 7.57*b*). In the latter application, the reinforcement reduces the potential for lateral displacement of the wall under the horizontal pressure of the backfill.

As in the reinforcement of steep slopes discussed in Art. 7.40.3, the reinforcement layers must intersect all critical failure surfaces. For cohesionless backfills, the failure surface may be assumed to be wedge shaped, as indicated in Fig. 7.56*c*, with the sloping plane of the wedge at an angle of $45^\circ + \phi/2$ with the horizontal. If the backfill is not homogeneous, a general stability analysis should be carried out as described in Art. 7.40.3.

The design process for cohesionless soils can be simplified by use of a constant vertical spacing s_v for the reinforcement layers. This spacing would be approximately

$$S_v = \frac{T_a}{KK_a wH} \tag{7.94}$$

where T_a = allowable tension in the reinforcement

K = safety factor as specified in a local code or as given in Table 7.21

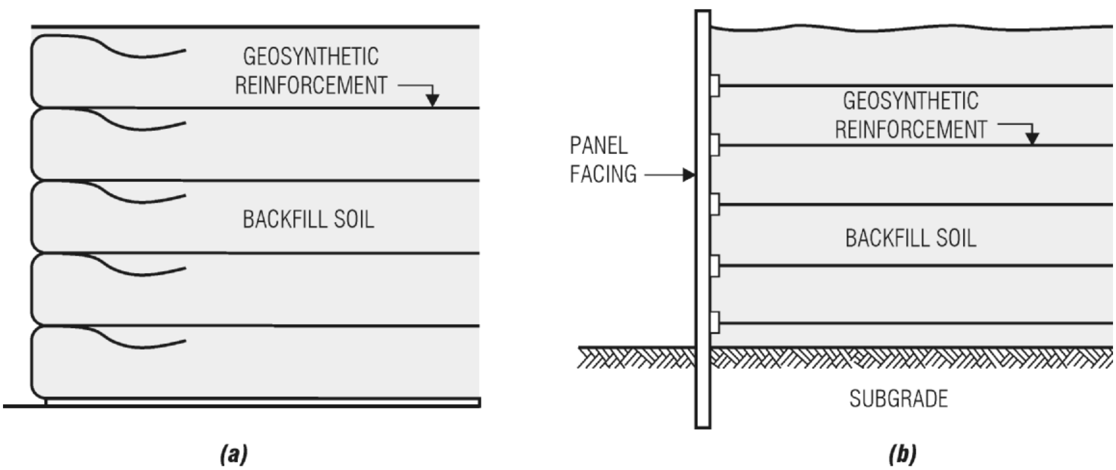

Fig. 7.57 Geosynthetic applications with retaining walls: (*a*) Reinforced earth forms a retaining wall. (*b*) Retaining wall anchored into backfill.

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K_a = coefficient of active earth pressure (Art. 7.27)

w = density of backfill

H = average height of embankment

If Eq. (7.94) yields a value for s_v less than the minimum thickness of a lift in placement of the backfill, a stronger geosynthetic should be chosen. The minimum embedment length L_e may be computed from Eq. (7.93). Although the total reinforcement length thus computed may vary from layer to layer, a constant reinforcement length would be convenient in construction.

When the earth adjoining the backfill is a random soil with lower strength than that of the backfill, the random soil exerts a horizontal pressure on the backfill that is transmitted to the wall (Fig. 7.58). This may lead to a sliding failure of the reinforced zone. The reinforcement at the base should be sufficiently long to prevent this type of failure. The total horizontal sliding force on the base is, from Fig. 7.58,

$$P = P_b + P_s + P_v \tag{7.95}$$

where $P_b = K_a w_b H^2/2$

wb = density of soil adjoining the reinforcement zone

$P_s = K_a w_s h H$

$w_s h$ = weight of uniform surcharge

P_v = force due to live load V as determined by the Boussinesq method (Art. 7.11)

The horizontal resisting force is

$$F_H = [(w_s h + w_r H)\tan\phi_{sr} + c]L \tag{7.96}$$

where $w_r H$ = weight of soil in the reinforcement zone

ϕ_{sr} = soil-reinforcement interaction angle

c = undrained shear strength of the backfill

L = length of the reinforcement zone base

The safety factor for sliding resistance then is

$$K_{sl} = \frac{F_H}{P} \tag{7.97}$$

and should be 1.5 or larger. A reinforcement length about 0.8H generally will provide base resistance sufficient to provide a safety factor of about 1.5.

The most economical retaining wall is one in which the reinforcement is turned upward and backward at the face of the wall and also serves as the face (Fig. 7.58*a*) The backward embedment should be at least 4 ft. If desired for esthetic reasons

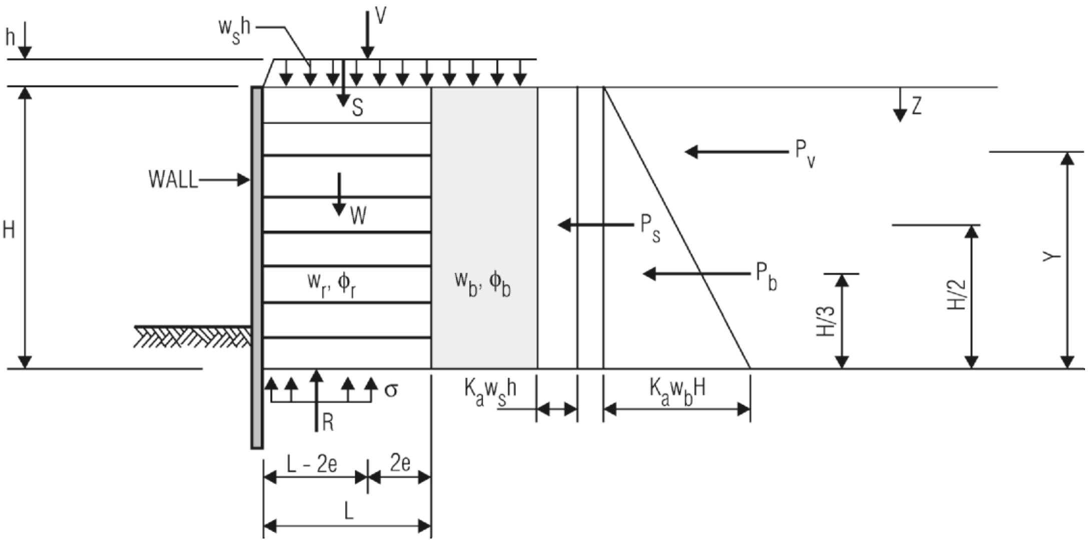

Fig. 7.58 Retaining wall anchored with geosynthetic reinforcement is subjected to pressure from random-soil backfill, sand backfill, surcharge, and live load. Assumed pressure distribution diagrams are rectangular and triangular.

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or to protect the geosynthetic from damage or deterioration from exposure to ultraviolet light, sprayed concrete may be applied to the wall face.

As an alternative, the wall may be composed of concrete block or precast concrete panels that are anchored to soil reinforcement. The reinforcement should be installed taut to limit lateral movement of the wall during construction.

See Art. 7.40.3 for other precautions to be taken during construction.

7.40.5 Geosynthetic Reinforcement for Embankments

Geosynthetics placed in horizontal layers may be used to reinforce embankments in a manner similar to that used to reinforce steep slopes (Art. 7.40.3). The reinforcement may permit greater embankment height and a larger safety factor in embankment design than would an unreinforced embankment. Also, displacements during construction may be smaller, thus reducing fill requirements. Furthermore, reinforcement properly designed and installed can prevent excessive horizontal displacements along the base that can cause an embankment failure when the underlying soil is weak. Moreover, reinforcement may decrease horizontal and vertical displacements of the underlying soil and thus limit differential settlement. Reinforcement, however, will reduce neither long-term consolidation of the underlying weak soil nor secondary settlement.

Either geotextiles or geogrids may be used as reinforcement. If the soils have very low bearing capacity, it may be necessary to use a geotextile separator with geogrids for filtration purposes and to prevent the movement of the underlying soil into the embankment fill.

Figure 7.59 illustrates reinforcement of an embankment completely underlain by a weak soil. Without reinforcement, horizontal earth pressure within the embankment would cause it to spread laterally and lead to embankment failure, in the absence of sufficient resistance from the soil. Reinforcement is usually laid horizontally in the direction of major stress; that is, with strong axis normal to the longitudinal axis of the embankment. Reinforcement with strong axis placed parallel to the longitudinal axis of the embankment may also be required at the ends of the embankment. Seams should be avoided in the high-stress direction.

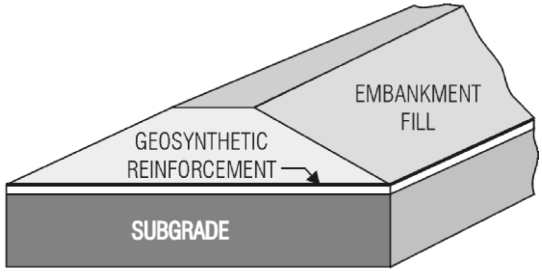

Fig. 7.59 Geosynthetic reinforcement for an embankment on weak soil is placed directly on the subgrade.

Design of the reinforcement is similar to that required for steep slopes (Art. 7.40.3).

For an embankment underlain by locally weak areas of soil or voids, reinforcement may be incorporated at the base of the embankment to bridge them.

7.40.6 Soil Stabilization with Geosynthetics

Woven or nonwoven geotextiles are used to improve the load-carrying capacity of roads over weak soils and to reduce rutting. Acting primarily as a separation barrier, the geosynthetic prevents the subgrade and aggregate base from mixing. The geosynthetic may also serve secondary functions. Acting as a filter, it prevents fines from migrating into the aggregate due to high water pressure. Also, the geotextile may facilitate drainage by allowing pore water to pass through and dissipate into the underlying soil. In addition, acting as reinforcement, the geotextile can serve as a membrane support of wheel loads and provide lateral restraint of the base and subgrade through friction between the fabric, aggregate, and soil.

Installation techniques to be used depend on the application. Usually, geosynthetics are laid directly on the subgrade (Fig. 7.60*a*). Aggregate then is placed on top to desired depth and compacted.

Design of permanent roads and highways consists of the following steps: If the CBR ≤ 3, need for a geotextile is indicated. The pavement is designed by usual methods with no allowance for structural support from the geotextile. If a thicker subbase than that required for structural support would have to be specified because of the sus-

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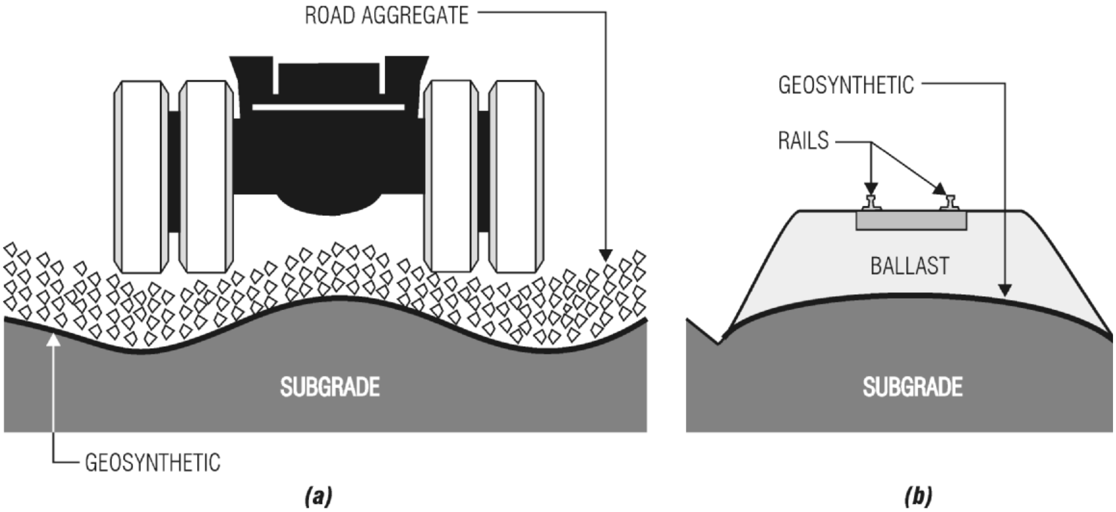

Fig. 7.60 Geosynthetic is used (*a*) to reinforce a road, (*b*) to reinforce a railway roadbed.

ceptibility of the underlying soil to pumping and subbase intrusion, the subbase may be reduced 50% and a geotextile selected for installation at the subbase-subgrade interface. For stabilization of the subgrade during construction, an additional determination of thickness of subbase assisted by a geotextile is made by conventional methods (bearing capacity N_c about 3.0 without geotextiles and about 5.5 with) to limit rutting to a maximum of 3 in under construction vehicle loads. The thicker subbase thus computed is selected. Then, the geotextile strength requirements for survivability and filtration characteristics are checked. (Details for this are given in B. R. Christopher and R. D. Holtz, "Geotextile Design and Construction Guidelines," FHWA DTFH61-86-C-00102, National Highway Institute, Federal Highway Administration, Washington, DC 20590 (www.fhwa.dot.gov).)

Geosynthetics are also used under railway tracks for separation of subgrade and subballast or subballast and ballast (Fig. 7.60*b*). They also are used for roadbed filtration, lateral permeability, and strength and modulus improvement.

7.40.7 Geosynthetics in Erosion Control

For purposes of erosion control, geosynthetics are used as turf reinforcement, as separators and filters under riprap, or armor stone, and as an alternative to riprap. Different types of geosynthetics are used for each of these applications.

Turf Control ▪ To establish a reinforced turf in ditches and water channels and on slopes, three-dimensional erosion-control mats often are used. Entangling with the root and stem network of vegetation, they greatly increase resistance to flow of water down slopes and thus retard erosion.

Mats used for turf reinforcement should have a strong, stable structure. They should be capable of retaining the underlying soil but have sufficient porosity to allow roots and stem to grow through them. Installation generally requires pinning the mat to the ground and burying mat edges and ends. Topsoil cover may be used to reduce erosion even more and promote rapid growth of vegetation.

When a geosynthetic is placed on a slope, it should be rolled in the direction of the slope. Horizontal joints should not be permitted. Vertical joints should be shingled downstream. Ditch and channel bottoms should be lined by rolling the geosynthetic longitudinally. Joints transverse to water flow should have a 3-ft overlap and be shingled downstream. Roll edges should be overlapped 2 to 4 in. They should be staked at intervals not exceeding 5 ft to prevent relative movement.

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Where highly erodible soils are encountered, a geotextile filter should be installed under the turf reinforcement and staked or otherwise bonded to the mats. For stability and seeding purposes, wood chips may be used to infill the turf reinforcement.

Use of Geosynthetics with Riprap ▪ Large armor stones are often used to protect soil against erosion and wave attack. Graded-aggregate filter generally is placed between the soil and the riprap to prevent erosion of the soil through the armoring layer. As a more economical alternative, geotextiles may be used instead of aggregate. They also offer greater control during construction, especially in underwater applications. Geosynthetics typically used are nonwoven fabrics, monofilament nonwoven geotextiles, and multifilament or fibrillated woven fabrics.

The geosynthetics should have sufficient permeability to permit passage of water to relieve hydrostatic pressure behind the riprap. Also, the geosynthetic should be capable of retaining the underlying soil. Conventional filter criteria can be used for design of.the geosynthetic, although some modifications may be required to compensate for properties of the geosynthetic.

Installation precautions that should be observed include the following: Riprap should be installed with care to avoid tearing the geosynthetic inasmuch as holes would decrease its strength. Stone placement, including drop heights, should be tested in field trials to develop techniques that will not damage the geosynthetic. As a general guide, for material protected by a sand cushion and material with properties exceeding that required for unprotected applications, drop height for stones weighing less than 250 lb should not exceed 3 ft; without a cushion, 1 ft. Stone weighing more than 250 lb should be placed without free fall, unless field tests determine a safe drop height. Stone weighing more than 100 lb should not be permitted to roll along the geosynthetic. Installation of the armor layer should begin at the base of slopes and at the center of the zone covered by the geosynthetic. After the stones have been placed, they should not be graded.

Special construction procedures are required for slopes greater than 2.5:1. These include increase in overlap, slope benching, elimination of pins at overlaps, toe berms for reaction against slippage, and laying of the geosynthetic sufficiently loose to allow for downstream movement, but folds and wrinkles should not be permitted.

The geosynthetic should be rolled out with its strong direction (machine direction for geotextiles) up and down the slope. Adjoining rolls should be seamed or shingle overlapped in the downslope or downstream direction. Joints should be stapled or pinned to the ground. Recommended pin spacing is 2 ft for slopes up to 3:1, 3 ft for slopes between 3:1 and 4:1, 5 ft for 4:1 slopes, and 6 ft for slopes steeper than 4:1. For streambanks and slopes exposed to wave action, the geosynthetic should be anchored at the base of the slope by burial around the perimeter of a stone-filled key trench. It should also be keyed at the top of the slope if the armor-geosynthetic system does not extend several feet above high water.

Riprap Replacement ▪ Instead of the riprap generally used for erosion control, concrete mats may be used. For this purpose, the concrete conventionally has been cast in wood or steel forms. Use of expandable fabric forms, however, may be more economical. Such forms are made by joining two fabric sheets at discrete points. After the sheets are placed over the area to be protected, grout is pumped into the space between the sheets to form a mattress that initially will conform to the shape of the ground and later harden. Thickness of the mattress is controlled by internal spacer threads. Filter points and bands are formed in the mattress to dissipate pore water from the subsoil. The fabric forms may be grouted underwater, even in flowing water, and in hazardous-liquid conditions. The fabric usually used is a woven geotextile.

7.40.8 Uses of Geosynthetics in Subsurface Drainage

Subsurface drainage is required for many construction projects and geotextiles find many uses in such applications. Their primary function is to serve, with graded granular filter media, as a permeable separator to exclude soil from the drainage media but to permit water to pass freely. Nonwoven geotextiles are usually used for this purpose because of their high flow capacity and small pore size. Generally, fabric strength is not a

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primary consideration for subsurface drainage applications, except during installation.

Following are brief descriptions of typical applications of geotextiles in subsurface drainage:

Permeable separators wrapped around trench or edge drains

Drains for retaining walls and bridge abutments with the geotextile enclosing the backfill

Encirclement of slotted or jointed drains and wall pipes to prevent filter particles from entering the drains while permitting passage of water

Wraps for interceptor, toe, and surface drains on slopes to assist stabilization by dissipating excess pore-water pressures and retarding erosion

Seepage control with chimney and toe drains for earth dams and levees with the geotextile laid along the upstream face and anchored by a berm.

7.40.9 Geosynthetics as Pond Liners

Geomembranes, being impermeable, appear to be an ideal material for lining the bottom of a pond to retain water or other fluid. Used alone, however, they have some disadvantages. In particular, they are susceptible to damage from many sources and require a protective soil cover of at least 12 in. Also, for several reasons, it is advisable to lay a geotextile under the geomembrane. The geotextile provides a clean working area for making seams. It adds puncture resistance to the liner. It increases friction resistance at the interface with the soil, thus permitting steeper side slopes. And it permits lateral and upward escape of gases emitted from the soil. For this purpose, needle-punched non-woven geotextiles, geonets, or drainage composites with adequate transmissivity for passing the gases are needed. In addition, it is advantageous to cover the top surface of the geomembrane with another geotextile. Its purpose is to maintain stability of cover soil on side slopes and to prevent sharp stones that may be present in the cover soil from puncturing the liner. This type of construction is also applicable to secondary containment of underground storage tanks for prevention of leakage into groundwater.

In selection of a geosynthetic for use as a pond liner, consideration should be given to its chemical resistance with regard to the fluid to be contained and reactive chemicals in the soil. For determination of liner thickness, assumptions have to be made as to loads from equipment during installation and basin cleaning as well as to pressure from fluid to be contained.

7.40.10 Geosynthetics as Landfill Liners

Liners are used along the bottom and sides of landfills to prevent leachate formed by reaction of moisture with landfill materials from contaminating adjacent property and groundwater. Clay liners have been traditional for this purpose (Fig. 7.61*a*). They have the disadvantage of being thick, often in the range of 2 to 6 ft, and being subject to piping under certain circumstances, permitting leakage of leachate. Geomembranes, geotextiles, geonets, and geocomposites offer an alternative that prevents rather than just minimizes leachate migration from landfills.

The U. S. Environmental Protection Agency (EPA) requires that all new hazardous-waste landfills, surface impoundments, and waste piles have two or more liners with a leachate-collection system between the liners. This requirement may be satisfied by installation of a top liner constructed of materials that prevent migration of any constituent into the liner during the period such facility remains in operation and a lower liner with the same properties. In addition, primary leachate-collection and leak-detection systems must be installed with the double liners to satisfy the following criteria:

The primary leachate-collection system should be capable of keeping the leachate head from exceeding 12 in.

Collection and leak-detection systems should incorporate granular drainage layers at least 12 in thick that are chemically resistant to the waste and leachate. Hydraulic conductivity should be at least 0.02 ft/min. An equivalent drainage geosynthetic, such as a geonet, may be used instead of granular layers. Bottom slope should be at least 2%.

A granular filter or a geotextile filter should be installed in the primary system above the drainage layer to prevent clogging.

When gravel is used as a filter, pipe drains resistant to chemicals should be installed to collect leachate efficiently (Fig. 7.61*a*).

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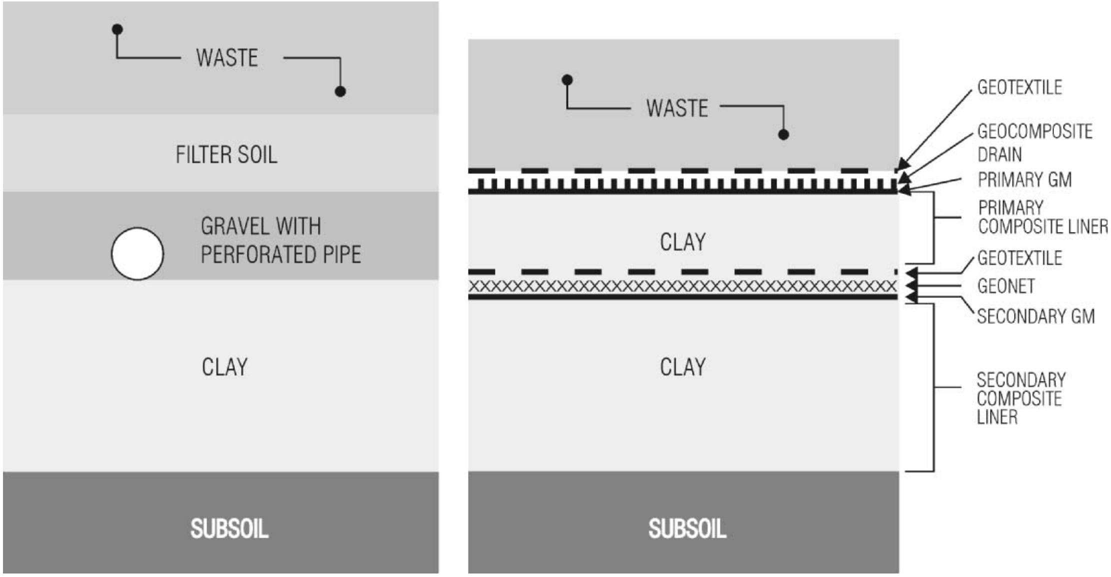

Fig. 7.61 Landfill liner systems: (*a*) with filter soil, gravel, drain pipe, and clay liner; (*b*) with geotextile separator-filter, geocomposite leachate drain, primary geomembrane and clay liner, geotextile filter, geonet for leak detection, and secondary geomembrane and clay liner.

Figure 7.61*b* illustrates a lining system that meets these criteria. Immediately underlying the wastes is a geotextile that functions as a filter. It overlies the geocomposite primary leachate drain. Below is the primary liner consisting of a geomembrane above a clay blanket. Next comes a geotextile filter and separator, followed underneath by a geonet that functions as a leak-detection drain. These are underlain by the secondary liner consisting of another geomembrane and clay blanket, which rests on the subsoil.

The EPA requires the thickness of a geomembrane liner for containment of hazardous materials to be at least 30 mils (0.75 mm) with timely cover or 45 mils (1.2 mm) without such cover. The secondary geomembrane liner should be the same thickness as the primary liner. Actual thickness required depends on pressures from the landfill and loads from construction equipment during installation of the liner system.

Terminals of the geosynthetics atop the side slopes generally consist of a short runout and a drop into an anchor trench, which, after insertion of the geosynthetics, is backfilled with soil and compacted. Side-slope stability of liner system and wastes needs special attention in design.

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American Rock Mechanics Association, www.armarocks.org

American Society of Civil Engineers, www.asce.org

American Society for Testing and Materials (ASTM), www.astm.org

Association of Engineering Geologists, www.aegweb.org

Association of Geotechnical & Geoenvironmental Specialists (AGS), www.ags.org.uk

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Canadian Geotechnical Society, www.cgs.ca

Deep Foundations Institute, www.dfi.org

Federal Highway Administration (FHWA), Bridge Technology, Geotechnical Section, www.fhwa.dot.gov/bridge/geo.htm

Geo-Institute of the American Society of Civil Engineers, www.geoinstitute.org

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Geosynthetic Institute (GSI), www.geosynthetic-institute.org

Geosynthetic Materials Association (GMA), www.gmanow.com

Geotechnical & Geoenvironmental Software Directory, www.ggsd.com

International Association of Foundation Drilling (ADSC), www.adsc-iafd.com

International Geosynthetic Society (IGS), http://mergrb.rmc.ca/

International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE), www.issmge.org

Mexican Society of Soil Mechanics, www.smms.org.mx

National Council for Geo-Engineering and Construction (Geo Council), www.geocouncil.org

National Geophysical Data Center (NGDC), www.ngdc.noaa.gov

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National Geotechnical Experimentation Sites (NGES) www.geocouncil.org/nges/nges.html

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