3

Soil Exploration

3–1 INTRODUCTION

In Chapter 2, various engineering properties of soils were presented. An evaluation of these properties is absolutely necessary in any rational design of structures resting on, in, or against soil. To evaluate these properties, it is imperative that geotechnical engineers visit proposed construction sites and collect and test soil samples in order to evaluate and record results in a useful and meaningful form.

Chapter 3 deals with evaluation of soil properties, including reconnaissance, steps of soil exploration (boring, sampling, and testing), and the record of field exploration. Although different types of soil tests are discussed in this chapter, detailed test methods are outside the scope of this book. For specific step-by-step procedures, the reader is referred to *Soil Properties: Testing, Measurement, and Evaluation*, 5th edition, by Liu and Evett (Prentice Hall, 2003).

3–2 RECONNAISSANCE

A reconnaissance is a preliminary examination or survey of a job site. Usually, some useful information on the area (e.g., maps or aerial photographs) will already be available, and an astute person can learn much about surface conditions and get a general idea of subsurface conditions by simply visiting the site, observing thoroughly and carefully, and properly interpreting what is seen.

The first step in the preliminary soil survey of an area should be to collect and study any pertinent information that is already available. This could include general geologic and topographical information available in the form of geologic and topographic maps, obtainable from federal, state, and local governmental agencies (e.g., U.S. Geological Survey, Soil Conservation Service of the U.S. Department of Agriculture, and various state geologic surveys).

Aerial photographs can provide geologic information over large areas. Proper interpretation of these photographs may reveal land patterns, sinkhole cavities, landslides, surface drainage patterns, and the like. Such information can usually be obtained on a more widespread and thorough basis by aerial photography than by visiting the project site. Specific details on this subject are, however, beyond the scope of this book. For more information, the reader is referred to the many books available on aerial photo interpretation.

After carefully collecting and studying available pertinent information, the geotechnical engineer should then visit the site in person, observe thoroughly and carefully, and interpret what is seen. The ability to do this successfully requires considerable practice and experience; however, a few generalizations are given next.

To begin with, significant details on surface conditions and general information about subsurface conditions in an area may be obtained by observing general topographical characteristics at the proposed job site and at nearby locations where soil was cut or eroded (such as railroad and highway cuts, ditch and stream erosion, and quarries), thereby exposing subsurface soil strata.

The general topographical characteristics of an area can be of significance. Any unusual conditions (e.g., swampy areas or dump areas, such as sanitary landfills) deserve particular attention in soil exploration.

Because the presence of water is often a major consideration in working with soil and associated structures, several observations regarding water may be made during reconnaissance. Groundwater tables may be noted by observing existing wells. Historical high watermarks may be recorded on buildings, trees, and so on.

Often, valuable information can be obtained by talking with local inhabitants of an area. Such information could include the flooding history, erosion patterns, mud slides, soil conditions, depths of overburden, groundwater levels, and the like.

One final consideration is that the reconnoiterer should take numerous photographs of the proposed construction site, exposed subsurface strata, adjacent structures, and so on. These can be invaluable in subsequent analysis and design processes and in later comparisons of conditions before and after construction.

The authors hope the preceding discussion in this section has made the reader aware of the importance of reconnaissance with regard to soil exploration at a proposed construction site. In addition to providing important information, the results of reconnaissance help determine the necessary scope of subsequent soil exploration.

At some point prior to beginning any subsurface exploration (Section 3–3), it is important that underground utilities (water mains, sewer lines, etc.) be located to assist in planning and carrying out subsequent subsurface exploration.

3–3 STEPS OF SOIL EXPLORATION

After all possible preliminary information is obtained as indicated in the preceding section, the next step is the actual subsurface soil exploration. It should be done by experienced personnel, using appropriate equipment. Much of geotechnical engineering practice can be successful only if one has long experience with which to compare each new problem.

Soil exploration may be thought of as consisting of three steps—boring, sampling, and testing. *Boring* refers to drilling or advancing a hole in the ground, *sampling* refers to removing soil from the hole, and *testing* refers to determining characteristics or properties of the soil. These three steps appear simple in concept but are quite difficult in good practice and are discussed in detail in the remainder of this section.

Boring

Some of the more common types of borings are *auger borings, test pits,* and *core borings.*

An *auger* (see Figure 3–1) is a screwlike tool used to bore a hole. Some augers are operated by hand; others are power operated. As the hole is bored a short distance, the auger may be lifted to remove soil. Removed soil can be used for field classification and laboratory testing, but it must not be considered as an undisturbed soil sample. It is difficult to use augers in either very soft clay or coarse sand because the hole tends to refill when the auger is removed. Also, it may be difficult or impossible to use an auger below the water table because most saturated soils will not cling sufficiently to the auger for lifting. Hand augers may be used for boring to a depth of about 20 ft (6 m); power augers may be used to bore much deeper and quicker.

Test pits are excavations into the earth that permit a direct, visual inspection of the soil along the sides of the pit. As depicted in Figure 3–2, they may be large enough to allow a person to enter them and make inspections by viewing the exposed walls, taking color photographs of the soil in its natural condition, testing *in situ,* and taking undisturbed samples. Clearly, the soil strata (including thicknesses and stiffnesses of strata), texture and grain size of the soil, along with visual classification of soils, soil moisture content, detection of fissures or cracks in the soil, and location of groundwater, among others, can be easily and accurately determined throughout the depth of the test pit. Soil samples can be obtained by carving an undisturbed sample from the pit's sides or bottom or by pushing a thin-walled

FIGURE 3–1 Auger boring (McCarthy, 2002).
Source: Courtesy of Acker Drill Co.

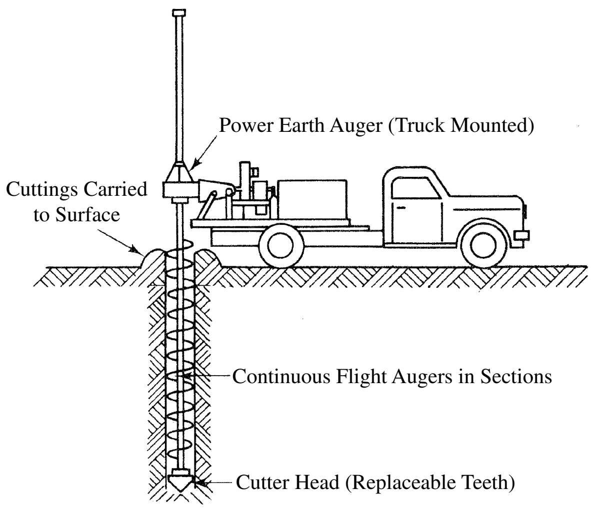
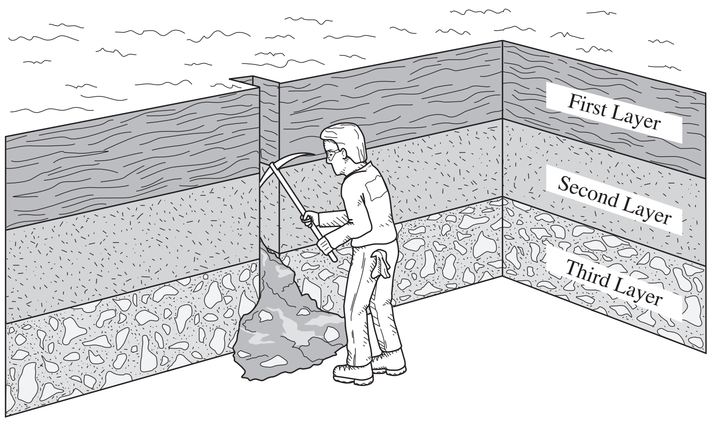


FIGURE 3–2 Test Pit.



steel tube into the pit's sides or bottom and extracting a sample by pulling the tube out. (Undisturbed samples should be preserved with wax to prevent moisture loss while the samples are transported to the laboratory.)

Test pits are excavated either manually or by power equipment, such as a backhoe or bulldozer (see Figure 3–3). For deeper pits, the excavation may need to be shored to protect persons entering the pits.

Soil inspection using test pits has several advantages. They are relatively rapid and inexpensive, and they provide a clear picture of the variation in soil properties with increasing depth. They also permit easy and reliable *in situ* testing and sampling. Another advantage of test pits is that they allow the detection and removal of larger soil particles (gravel or rocks, for example) for identification and testing; this may not be possible with boring samplers. On the other hand, test pits are generally limited by practical considerations as to depth; they generally do not extend deeper than 10 to 15 ft, whereas auger boring samplers can extend to much greater depths. Also, a high water table may preclude or limit the use of test pits.

Oftentimes, the presence of subsurface rock at a construction site can be important. Many times, construction projects have been delayed at considerable expense upon encountering unexpected rock in an excavation area. On the other hand, the presence of rock may be desirable if it can be used to support the load of an overlying structure. For these and other reasons, an investigation of subsurface rock in a project area is an important part of soil investigation.

Core borings are commonly used to drill into and through rock formations. Because rock is invariably harder than sandy and clayey soils, the sampling tools used for drilling in soil are usually not adequate for investigating subsurface rock. Core borings are performed using a core barrel, a hardened steel or steel alloy tube with a hard cutting bit containing tungsten carbide or commercial diamond chips (see Figure 3–4). Core barrels are typically 5 to 10 cm (2 to 4 in.) in diameter and 60 to 300 cm (2 to 10 ft) long.

Core borings are performed by attaching the core barrel and cutting bit to rods and rotating them with a drill, while water or air, serving as a coolant, is pushed (pumped) through the rods and barrel, emerging at the bit. The core remains in the

FIGURE 3–3 Backhoe (Permission granted by Caterpillar, Inc.).

core barrel and may be removed for examination by bringing the barrel to the surface. The rock specimen can be removed from the barrel, placed in the core box (see Figure 3–5), and sent to the laboratory for testing and analysis. The (empty) core barrel can then be used for another boring.

A wealth of information can be obtained from the laboratory testing and analysis of a rock core boring. The type of rock (such as granite, sandstone), its texture (coarse-grained or fine-grained, or some mixture of the two), degree of stratification (such as laminations), orientation of rock formation (bedding planes vertical, horizontal, or in between), and the presence of weathering, fractures, fissures, faults, or seams can be observed. Also, compression tests can be performed on core samples to determine the rock's compressive strength, and permeability tests can be done to see how underground water flow might be affected. All of the foregoing information can be invaluable in the design process and to prevent costly "surprises" that may be encountered during excavations.

FIGURE 3–4 Cutting bit for rock coring.

FIGURE 3–5 Core box containing rock core samples.

Core recovery is the length of core obtained divided by the distance drilled. For example, a laminated shale stratum with a number of clay seams would likely exhibit a relatively small percentage of core recovery because the clayey soil originally located between laminations may have been washed or blown away by the water or air, respectively, during the drilling process. On the other hand, a larger percentage of core recovery would be expected in the case of granite.

Preceding paragraphs have discussed some of the more common types of borings. Once a means of boring has been decided upon, the question arises as to how many borings should be made. Obviously, the more borings made, the better the analysis of subsurface conditions should be. Borings are expensive, however, and a balance must be made between the cost of additional borings and the value of information gained from them.

As a rough guide for initial spacing of borings, the following are offered: for multistory buildings, 50 to 100 ft (15 to 30 m); for one-story buildings, earthen dams, and borrow pits, 100 to 200 ft (30 to 60 m); and for highways (subgrade), 500 to 1000 ft (150 to 300 m). These spacings may be increased if soil conditions are found to be relatively uniform and must be decreased if found to be nonuniform.

Once the means of boring and the spacing have been decided upon, a final question arises as to how deep the borings should be. In general, borings should extend through any unsuitable foundation strata (unconsolidated fill, organic soils, compressible layers such as soft, fine-grained soils, etc.) until soil of acceptable bearing capacity (hard or compact soil) is reached. If soil of acceptable bearing capacity is encountered at shallow depths, one or more borings should extend to a sufficient depth to ensure that an underlying weaker layer, if found, will have a negligible effect on surface stability and settlement. In compressible fine-grained strata, borings should extend to a depth at which stress from the superimposed load is so small that surface settlement is negligible. In the case of very heavy structures, including tall buildings, borings in most cases should extend to bedrock. In all cases, it is advisable to investigate drilling at least one boring to bedrock.

The preceding discussion presented some general considerations regarding boring depths. A more definitive criterion for determining required minimum depths of test borings in cohesive soils is to carry borings to a depth where the increase in stress due to foundation loading (i.e., weight of the structure) is less than 10% of the effective overburden pressure. Figures 3–6, 3–7, and 3–8 were developed (Barksdale and Schreiber, 1979) to determine minimum depths of borings based on the 10% increase in stress criterion for cohesive soils. Figure 3–6 is for a continuous footing (such as a wall footing). Figure 3–7 is for a square footing with a design pressure between 1000 and 9000 lb/ft^2, and Figure 3–8 is for a square footing with a design pressure between 100 and 1000 lb/ft^2. If the groundwater table is at the footing's base, the buoyant weight (submerged unit weight) of the soil should be used in these figures. If the groundwater table is lower than distance B below the footing (B is the footing's width), the wet unit weight should be used. For intermediate conditions, an interpolation can be made between required depths of boring for shallow and deep groundwater conditions, or the groundwater table can be conservatively assumed to be at the footing's base. It should be noted that on the left sides of Figures 3–6 through 3–8 two scales are given for footing width and minimum test boring depth. In each figure, footing widths given on one side of the width scale correspond with boring depths given on the same side of the boring depth scale (Barksdale and Schreiber, 1979).

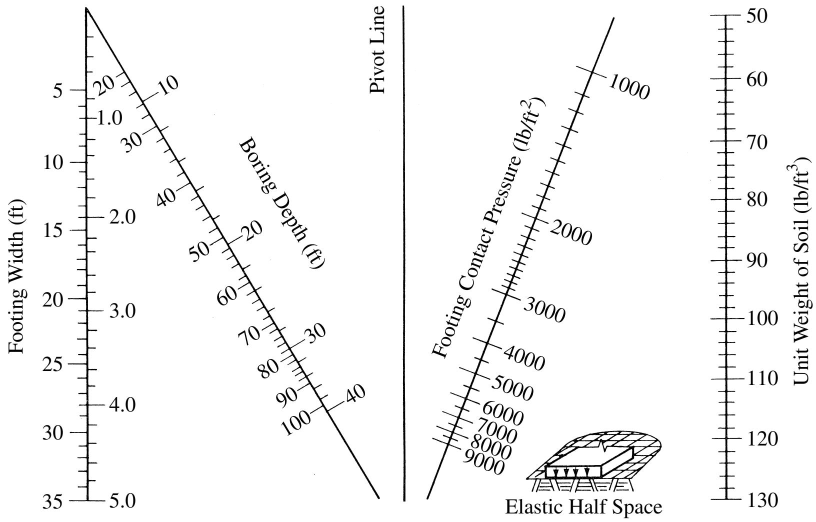


FIGURE 3–6 Infinite strip loading—Boussinesq-type solid (1 ft = 0.3048 m; 1 lb/ft^2 = 47.88 N/m^2; 1 lb/ft^3 = 0.1571 kN/m^3).
Source: R. D. Barksdale and M. O. Schreiber, "Calculating Test-Boring Depths," *Civil Eng.*, ASCE, 49(8) 74–75 (1979). Reprinted by permission.

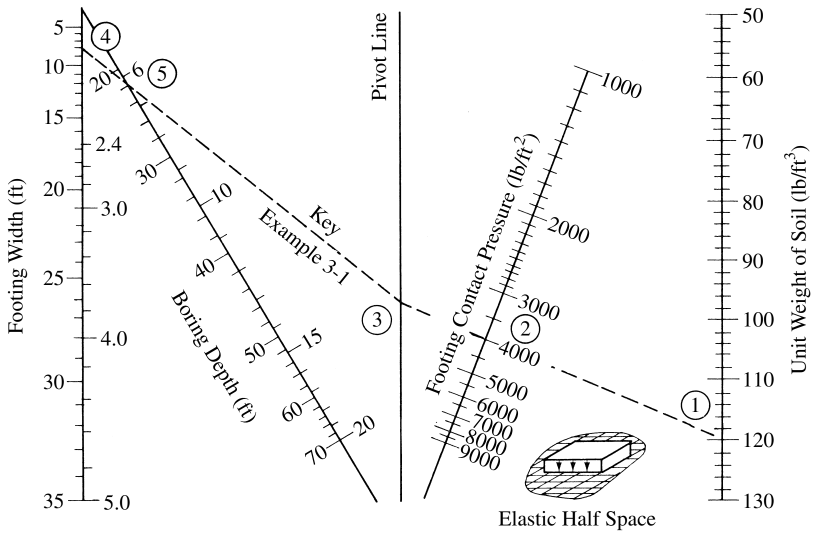


FIGURE 3–7 Square loading—Boussinesq-type solid (1 ft = 0.3048 m; 1 lb/ft^2 = 47.88 N/m^2; 1 lb/ft^3 = 0.1571 kN/m^3).
Source: R. D. Barksdale and M. O. Schreiber, "Calculating Test-Boring Depths," *Civil Eng., ASCE*, 49(8) 74–75 (1979). Reprinted by permission.

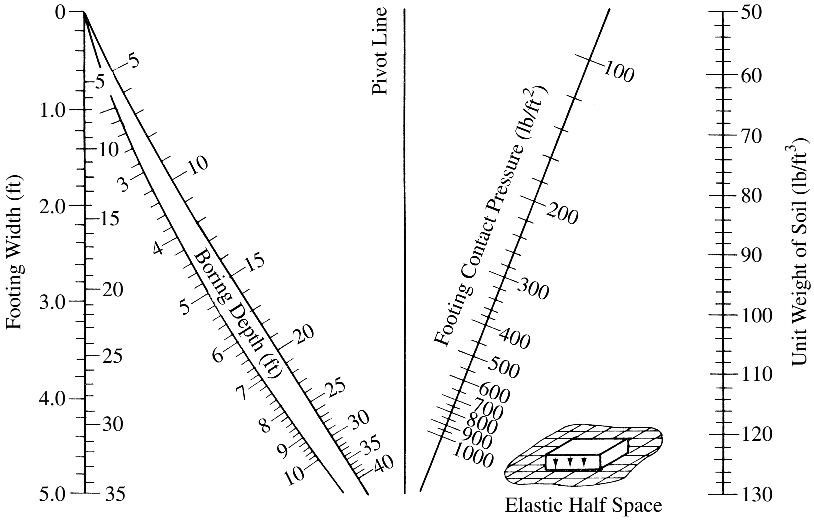


FIGURE 3–8 Square loading (low-pressure)—Boussinesq-type solid (1 ft = 0.3048 m; 1 lb/ft^2 = 47.88 N/m^2; 1 lb/ft^3 = 0.1571 kN/m^3).
Source: R. D. Barksdale and M. O. Schreiber, "Calculating Test-Boring Depths," *Civil Eng., ASCE*, 49(8) 74–75 (1979). Reprinted by permission.

EXAMPLE 3–1

Given

1. An 8-ft square footing is subjected to a contact pressure of 4000 lb/ft^2.
2. The wet unit weight of the soil supporting the footing is estimated to be 120 lb/ft^3.
3. The water table is estimated to be 30 ft beneath the footing.

Required

The minimum depth of test boring.

Solution

Because the water table is estimated to be 30 ft beneath the footing and the footing's width is 8 ft, the soil's wet unit weight should be used. From Figure 3–7, with a wet unit weight of 120 lb/ft^3, contact pressure between footing and soil equal to 4000 lb/ft^2, and width of footing equal to 8 ft, the minimum depth of test boring is determined to be 22 ft.

Figures 3–6 through 3–8 are quite useful for estimating minimum required test boring depths in cohesive soils. In the final analysis, however, the depth of a specific boring should be determined by the engineer based on his or her expertise, experience, judgment, and general knowledge of the specific area. Also, in some cases, the depth (and spacing) of borings may be specified by local codes or company policy.

Sampling

Sampling refers to the taking of soil or rock from bored holes. Samples may be classified as either *disturbed* or *undisturbed*.

As mentioned previously in this section, in auger borings soil is brought to the ground surface, where samples can be collected. Such samples are obviously disturbed samples, and thus some of their characteristics are changed. (Split-spoon samplers, described in Section 3–5, also provide disturbed samples.) Disturbed samples should be placed in an airtight container (plastic bag or airtight jar, for example) and should, of course, be properly labeled as to date, location, borehole number, sampling depth, and so on. Disturbed samples are generally used for soil grain-size analysis, determination of liquid and plastic limits and specific gravity of soil, and other tests, such as the compaction and CBR (California bearing ratio) tests.

For determination of certain other properties of soils, such as strength, compressibility, and permeability, it is necessary that the collected soil sample be exactly the same as it was when it existed in place within the ground. Such a soil sample is referred to as an undisturbed sample. It should be realized, however, that such a sample can never be completely undisturbed (i.e., be exactly the same as it was when it existed in place within the ground).

Undisturbed samples may be collected by several methods. If a test pit is available in clay soil, an undisturbed sample may be obtained by simply carving a sample very carefully out of the side of the test pit. Such a sample should then be coated with paraffin wax and placed in an airtight container. This method is often too tedious, time consuming, and expensive to be done on a large scale, however.

A more common method of obtaining an undisturbed sample is to push a thin tube into the soil, thereby trapping the (undisturbed) sample inside the tube, and then to remove the tube and sample intact. The ends of the tube should be sealed with paraffin wax immediately after the tube containing the sample is brought to the ground surface. The sealed tube should then be sent to the laboratory, where subsequent tests can be made on the sample, with the assumption that such test results are indicative of the properties of the soil as it existed in place within the ground. The thin-tube sampler is called a *Shelby tube*. It is a 2- to 3-in. (51- to 76-mm)-diameter 16-gauge seamless steel tube (see Figure 3–9).

When using a thin-tube sampler, the engineer should minimize the disturbance of the soil. Pushing the sampler into the soil quickly and with constant speed causes the least disturbance; driving the sampler into the soil by blows of a hammer produces the most.

Normally, samples (both disturbed and undisturbed) are collected at least every 5 ft (1.5 m) in depth of the boring hole. When, however, any change in soil characteristics is noted within 5-ft intervals, additional samples should be taken.

The importance of properly and accurately identifying and labeling each sample cannot be overemphasized.

FIGURE 3–9 Shelby tube.

After a boring has been made and samples taken, an estimate of the groundwater table can be made. It is common practice to cover the hole (e.g., with a small piece of plywood) for safety reasons, mark it for identification, leave it overnight, and return the next day to record the groundwater level. The hole should then be filled in to avoid subsequent injury to people or animals (see Section 3–4).

Testing

A large number of tests can be performed to evaluate various soil properties. These include both laboratory and field tests. Some of the most common tests are listed in Table 3–1. As indicated at the beginning of this chapter, the reader is referred to *Soil Properties: Testing, Measurement, and Evaluation*, 5th edition, by Liu and Evett (Prentice Hall, 2003) for specific step-by-step procedures involving these tests. Three field tests—the standard penetration test, cone penetration test, and vane test—are described in some detail in Sections 3–5 through 3–7.

3–4 GROUNDWATER TABLE

The term *groundwater table* (or just *water table*) has been mentioned several times earlier in this chapter. Section 3–4 presents more detailed information about this important phenomenon as it relates to the study of soils.

The location of the water table is a matter of importance to engineers, particularly when it is near the ground surface. For example, a soil's bearing capacity (see Chapter 9) can be reduced when the water table is at or near a footing. The location of the water table is not fixed at a particular site; it tends to rise and fall during periods of wet and dry weather, respectively. Fluctuations of the water table may result in reduction of foundation stability; in extreme cases, structures may float out of the ground. Accordingly, foundation design and/or methods of construction may

TABLE 3–1
Common Types of Testing

Property of Soil	Type of Test	ASTM Designa-tion	AASHTO Designa-tion
	(a) *Laboratory testing of soils*		
Grain-size distribution	Mechanical analysis	D422	T88
Consistency	Liquid limit (LL)	D4318	T89
	Plastic limit (PL)	D4318	T90
	Plasticity index (PI)	D4318	T90
Unit weight	Specific gravity	D854	T100
Moisture	Natural water content		
	Conventional oven method	D2216	T93
	Microwave oven method	D4643	
Shear strength	Unconfined compression	D2166	T208
	Direct shear	D3080	T236
	Triaxial	D2850	T234
Volume change	Shrinkage factors	D427	T92
Compressibility	Consolidation	D2435	T216
Permeability	Permeability	D2434	T215
Compaction	Standard Proctor	D698	T99
characteristics	Modified Proctor	D1557	T180
California bearing ratio (CBR)		D1883	T193
	(b) *Field testing of soils*		
Compaction	Moisture–density relations	D698	T99, T180
control	In-place density (Sand-cone Method)	D1556	T191
	In-place density (Nuclear Method)	D2922	T205
Shear strength (soft clay)	Vane test	D2573	T223
Relative density (granular soil)	Penetration test	D1586	T206
Permeability	Pumping test		
Bearing capacity			
Pavement	CBR	D1883	T193
Piles (vertical load)	Pile load test	D1143	T222

be affected by the location of the water table. Knowing the position of the water table is also very important when sites are being chosen for hazardous waste and sanitary landfills, to avoid contaminating groundwater.

The water table can be located by measuring down to the water level in existing wells in an area. It can also be determined from boring holes. The level to which groundwater rises in a boring hole is the groundwater elevation in that area. If adjacent soil is pervious, the water level in a boring hole will stabilize in a short period of time; if the soil is relatively impervious, it may take much longer for this to happen. General practice in soil surveying is to cover the boring hole (e.g., with a small piece of plywood) for safety reasons, leave it for at least 24 hours to allow the water level to rise in the hole and stabilize, and return the next day to locate and record the groundwater table. The hole should then be filled to avoid subsequent injury to people or animals.

3–5 STANDARD PENETRATION TEST (ASTM D 1586)

The standard penetration test (SPT) is widely used in the United States. Relatively simple and inexpensive to perform, it is useful in determining certain properties of soils, particularly of cohesionless soils, for which undisturbed samples are not easily obtained.

The SPT utilizes a *split-spoon sampler* (see Figure 3–10). It is a 2-in. (51-mm)-O.D. 1 3/8-in. (35-mm)-I.D. tube, 18 to 24 in. (457 to 610 mm) long, that is split longitudinally down the middle. The split-spoon sampler is attached to the bottom of a drilling rod and driven into the soil with a drop hammer. Specifically, a 140-lb (623-N) hammer falling 30 in. (762 mm) is used to drive the split-spoon sampler 18 in. (457 mm) into the soil.

As a sampler is driven the 18 in. (457 mm) into the soil, the number of blows required to penetrate each of the three 6-in. (152-mm) increments is recorded separately. The standard penetration resistance value (or *N*-value) is the number of blows required to penetrate the last 12 in. (305 mm). Thus, the *N*-value represents the number of blows per foot (305 mm). After blow counts have been obtained, the split-spoon sampler can be removed and opened (along the longitudinal split) to obtain a disturbed sample for subsequent examination and testing.

SPT results (i.e., *N*-values) are influenced by overburden pressure (effective weight of overlying soil) at locations where blow counts are made. Several methods have been proposed to correct *N*-values to reflect the influence of overburden pressure. Two methods are presented here.

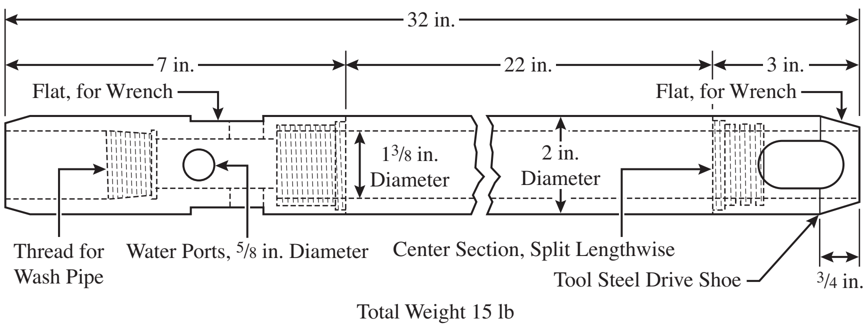


FIGURE 3–10 Split-spoon sampler for the standard penetration test.

One method (Peck et al., 1974) utilizes the following equations to evaluate C_N, a correction factor to be applied to the N-value determined in the field:

$$C_N = 0.77 \log_{10} \frac{20}{p_0} \quad (p_0 \text{ in tons/ft}^2) \tag{3–1}$$

$$C_N = 0.77 \log_{10} \frac{1915}{p_0} \quad (p_0 \text{ in kN/m}^2) \tag{3–2}$$

where p_0 is the effective overburden pressure at the elevation of the SPT. These equations are not valid if p_0 is less than 0.25 ton/ft^2 (24 kN/m^2). Figure 3–11 gives a graphic relationship, based in part on Eq. (3–1), for determining a correction factor to be applied to the N-value recorded in the field. If p_0 is greater than or equal to 0.25 ton/ft^2, the correction factor may be determined using either Eq. (3–1) or Figure 3–11. If p_0 is less than 0.25 ton/ft^2, the correction factor should be taken from the figure.

A second method for correcting N-values to reflect the influence of overburden pressure (Terzaghi et al., 1996 and Liao and Whitman, 1986) utilizes the following equation*:

$$N = N' \times (100/p_0)^{1/2} \tag{3–3}$$

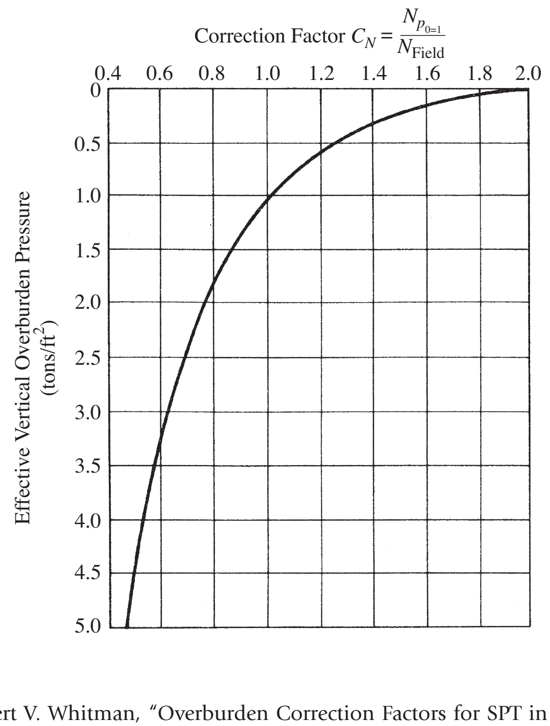


FIGURE 3–11 Chart for correction of N-values in sand for influence of overburden pressure (reference value of effective overburden pressure, 1 ton/ft^2).
Note: 1 ton/ft^2 = 95.76 kN/m^2.
Source: R. B. Peck, W. E. Hansen, T. H. Thornburn, *Foundation Engineering*, 2nd ed., John Wiley & Sons, Inc. New York, 1974. Copyright ©1974 by John Wiley & Sons, Inc. Reprinted by permission of John Wiley & Sons, Inc.

*From Samson, S., C. Liao and Robert V. Whitman, "Overburden Correction Factors for SPT in Sand," *J. Geotech. Eng. Div. ASCE,* 112(3), 373–377 (1986). Reproduced by permission of ASCE.

where N = corrected N-value
N' = N-value determined in the field
p_0 = effective overburden pressure

These methods give comparable results. It should be noted that the first method [Eqs. (3–1) and (3–2)] results in no adjustment of the N-value at a depth where the effective overburden pressure is 1 ton/ft^2 (96 kN/m^2). The second method [Eq. (3–3)] yields no adjustment at a depth where the effective overburden pressure is 100 kN/m^2 (1.04 tons/ft^2).

EXAMPLE 3–2

Given

An SPT was performed at a depth of 20 ft in sand of unit weight 135 lb/ft^3. The blow count was 40.

Required

The corrected N-value by each of the methods presented previously.

Solution

1. By Eq. (3–1),

$$C_N = 0.77 \log_{10} \frac{20}{p_0} \tag{3–1}$$

$$p_0 = \frac{(20\text{ ft})(135\text{ lb/ft}^3)}{2000\text{ lb/ton}} = 1.35\text{ tons/ft}^2$$

$$C_N = 0.77 \log_{10} \frac{20}{1.35\text{ tons/ft}^2} = 0.901$$

(This value of 0.901 for C_N can also be obtained using Figure 3–11 by locating 1.35 tons/ft^2 along the ordinate, moving horizontally to the curved line, and then moving upward to obtain the correction factor, C_N.) Therefore,

$$N_{\text{corrected}} = (40)(0.901) = 36$$

2. By Eq. (3–3),

$$N = N' \times (100/p_0)^{1/2} \tag{3–3}$$

$$p_0 = (1.35\text{ tons/ft}^2)\left(\frac{95.76\text{ kN/m}^2}{1\text{ ton/ft}^2}\right) = 129.3\text{ kN/m}^2$$

$$N = (40) \times (100/129.3\text{ kN/m}^2)^{1/2}$$

$$N_{\text{corrected}} = 35$$

EXAMPLE 3–3

Given

An SPT test was performed at a depth of 8.5 m in sand of unit weight 20.04 kN/m^3. The blow count was 38.

Required

The corrected N-value by each of the methods presented previously.

Solution

1. By Eq. (3–2),

$$C_N = 0.77 \log_{10} \frac{1915}{p_0} \tag{3–2}$$

$$p_0 = (8.5\text{ m})(20.04\text{ kN/m}^3) = 170.3\text{ kN/m}^2$$

$$C_N = 0.77 \log_{10} \frac{1915}{170.3\text{ kN/m}^2} = 0.809$$

Therefore,

$$N_{\text{corrected}} = (38)(0.809) = 31$$

2. By Eq. (3–3),

$$N = N' \times (100/p_0)^{1/2} \tag{3–3}$$

$$N = (38) \times (100/170.3\text{ kN/m}^2)^{1/2}$$

$$N_{\text{corrected}} = 29$$

EXAMPLE 3–4

Given

Same data as given in Example 3–2, except that the water table is located 5 ft below the ground surface.

Required

The corrected N-value by the first method presented previously.

Solution

By Eq. (3–1),

$$C_N = 0.77 \log_{10} \frac{20}{p_0} \tag{3–1}$$

$$p_0 = \frac{(5\text{ ft})(135\text{ lb/ft}^3) + (15\text{ ft})(135\text{ lb/ft}^3 - 62.4\text{ lb/ft}^3)}{2000\text{ lb/ton}}$$
$$= 0.882\text{ ton/ft}^2$$
$$C_N = 0.77\log_{10}\frac{20}{0.882\text{ ton/ft}^2} = 1.04$$

Therefore,

$$N_{\text{corrected}} = (40)(1.04) = 42$$

In addition to the effect of overburden pressure, SPT results (N-values) are influenced by (1) drill rod lengths, (2) whether or not liners are present in the sampler, and (3) borehole diameters. Table 3–2 gives some corrections that can be applied to measured N-values to adjust for these three influences.

Through empirical testing, correlations between (corrected) SPT N-values and several soil parameters have been established. These are particularly useful for cohesionless soils but are less reliable for cohesive soils. Table 3–3 gives correlations of the relative density of sands with SPT N-values; Table 3–4 gives correlations of the consistency of clays and unconfined compressive strength (q_u). Figure 3–12 gives a graphic relationship between the angle of internal friction of cohesionless soil and SPT N-values. Figure 3–12 also gives graphic relationships between certain bearing capacity factors for cohesionless soil and SPT N-values. These relationships will be utilized in Chapter 9.

The reader is cautioned that, although the standard penetration test is widely used in the United States, results are highly variable and thus difficult to interpret.

TABLE 3–2
Approximate Corrections to Measured *N*-Values

Influence	Correction Size	Factor
Rod length	>10 m	1.0
	6–10 m	0.95
	4–6 m	0.85
	3–4 m	0.75
Standard sampler	—	1.0
U.S. sampler without liners	—	1.2
Borehole diameter	65–115 mm	1.0
	150 mm	1.05
	200 mm	1.15

Source: A. W. Skempton, "Standard Penetration Test Procedures and the Effects in Sands of Overburden Pressure, Relative Density, Particle Size, Ageing, and Overconsolidation," *Geotechnique*, *36*(3), 425–447 (1986). Reprinted by permission; and K. Terzaghi, R. B. Peck, and G. Mesri, *Soil Mechanics in Engineering Practice*, 3rd ed., John Wiley & Sons, Inc. New York, 1996. Copyright © 1996 by John Wiley & Sons, Inc. Reprinted by permission of John Wiley & Sons, Inc.

TABLE 3–3
Relative Density of Sands According to Results of Standard Penetration Text

SPT *N*-Value	Relative Density
0–4	Very loose
4–10	Loose
10–30	Medium
30–50	Dense
Over 50	Very dense

Source: K. Terzaghi, R. B. Peck, and G. Mesri, *Soil Mechanics in Engineering Practice*, 3rd ed., John Wiley & Sons, Inc. New York, 1996. Copyright © 1996 by John Wiley & Sons, Inc. Reprinted by permission of John Wiley & Sons, Inc.

TABLE 3–4
Relation of Consistency of Clay, SPT *N*-Value, and Unconfined Compressive Strength (q_u)

	q_u (kN/m²)					
Consistency:	*Very Soft*	*Soft*	*Medium*	*Stiff*	*Very Stiff*	*Hard*
SPT *N*-value	<2	2–4	4–8	8–15	15–30	>30
q_u	<25	25–50	50–100	100–200	200–400	>400

Source: K. Terzaghi, R. B. Peck, and G. Mesri, *Soil Mechanics in Engineering Practice*, 3rd ed., John Wiley & Sons, Inc. New York, 1996. Copyright © 1996 by John Wiley & Sons, Inc. Reprinted by permission of John Wiley & Sons, Inc.

Nevertheless, it is a useful guide in foundation analysis. Much experience is necessary to properly apply the results obtained. Outside the United States, other techniques are used. For example, in Europe the cone penetration test (Section 3–6) is often preferred.

3–6 CONE PENETRATION TEST (ASTM D 3441 AND D 5778)

The cone penetration test (CPT) has been widely used in Europe for many years but is now gaining favor in the United States. It has the advantage of accomplishing subsurface exploration rapidly without taking soil samples.

In simple terms, a *cone penetrometer* is a slender metal rod containing a 35.7-mm-diameter, cone-shaped tip with a 60° apex angle; a *friction-cone penetrometer* contains a 133.7-mm-long cylindrical sleeve in addition to a cone-shaped tip. A penetrometer is advanced into and through the soil, and its resistance to being advanced through the soil is measured as a function of the depth of soil penetrated. Correlations between such resistance and soil types can give valuable information regarding soil type as a function of depth. Cone penetrometers can be categorized as

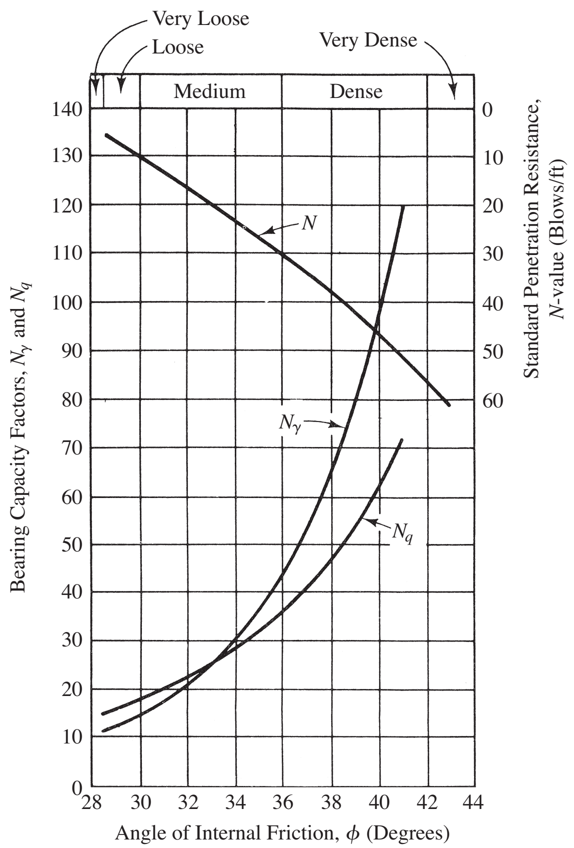
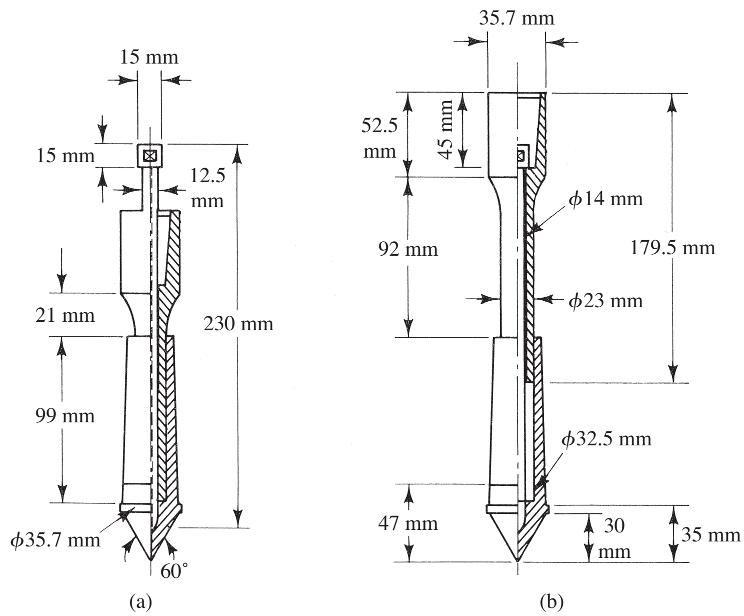


FIGURE 3–12 Curves showing the relationship between bearing capacity factors and φ, as determined by theory, and the rough empirical relationship between bearing capacity factors or φ and values of standard penetration resistance, N. *Source:* R. B. Peck, W. E. Hansen, T. H. Thornburn, *Foundation Engineering*, 2nd ed., John Wiley & Sons, Inc. New York, 1974. Copyright © 1974 by John Wiley & Sons, Inc. Reprinted by permission of John Wiley & Sons, Inc.

mechanical cone penetrometers (ASTM D 3441) and electric friction-cone penetrometers (ASTM D 5778).

There are two types of mechanical cone penetrometers—the *mechanical cone penetrometer* (Figure 3–13) and the *mechanical friction-cone penetrometer* (Figure 3–14). The main difference between the two is that in addition to cone resistance, the friction-cone penetrometer also allows for determination of side (sleeve) resistance as the penetrometer is advanced through the soil. Mechanical cone penetrometers are either pushed (by a hydraulic jack, for example) or driven (such as by blows of a drop hammer) into and through soil. When penetrometers are pushed, the test is known as a *static cone test* (sometimes referred to as a *Dutch cone test*); when they are driven, the test is called a *dynamic cone test*. The static test is sensitive to small differences in soil consistency. Because the penetrometer is pushed (rather than driven) in a static test, the procedure probably tends not to alter soil structure significantly for loose sands and sensitive clays. The dynamic test covers a wider range of soil consistencies, and because the penetrometer is driven, penetrations of gravel and soft rock are possible.

FIGURE 3–13 Mechanical cone penetrometer tip (Dutch mantle cone): (a) collapsed; (b) extended.
Source: Annual Book of ASTM Standards, ASTM, Philadelphia, 2002. Copyright American Society for Testing and Materials. Reprinted with permission.



While mechanical penetrometers have relatively low initial cost, they are relatively slow in use, labor intensive, and somewhat limited in accuracy. They have been supplanted to some extent by *electric friction-cone penetrometers,* which are more expensive but operate faster, are less labor intensive, and provide higher accuracy. The two types of penetrometers differ in their operation. Mechanical penetrometers are advanced through the soil in stages and measure cone resistance and friction resistance at intervals of around 20 cm; electric penetrometers include built-in strain gauges, which make continuous measurements of cone resistance and friction resistance with increasing depth. Figure 3–15 illustrates an electric friction-cone penetrometer. *Piezocone penetrometers* (Figure 3–16) are essentially electric friction-cone penetrometers that contain pressure sensors for measuring pore water pressure that develops during a test. They have been useful in saturated clays.

In all cases, the penetrometer's resistance to being pushed through the soil is measured and recorded as a function of depth of soil penetrated. The cone resistance (q_c) is the total force acting on the penetrometer divided by its projected area (i.e., the area of a 35.7-mm-diameter circle, or 10 cm^2). The friction resistance (f_s) is the total friction force acting on the friction sleeve divided by its surface area (i.e., the side area of a 35.7-mm-diameter, 133.7-mm-long cylinder, or 150 cm^2). The ratio of friction resistance to cone resistance is known as the friction ratio and is denoted by F_r (i.e., $F_r = f_s/q_c$). CPT data are ordinarily presented as plots of cone resistance, friction resistance, and friction ratio versus depth (see Figures 3–17 and 3–18). In general, the ratio of sleeve resistance to cone resistance is higher in cohesive soils than in cohesionless soils; hence, this ratio together with cone resistance can be

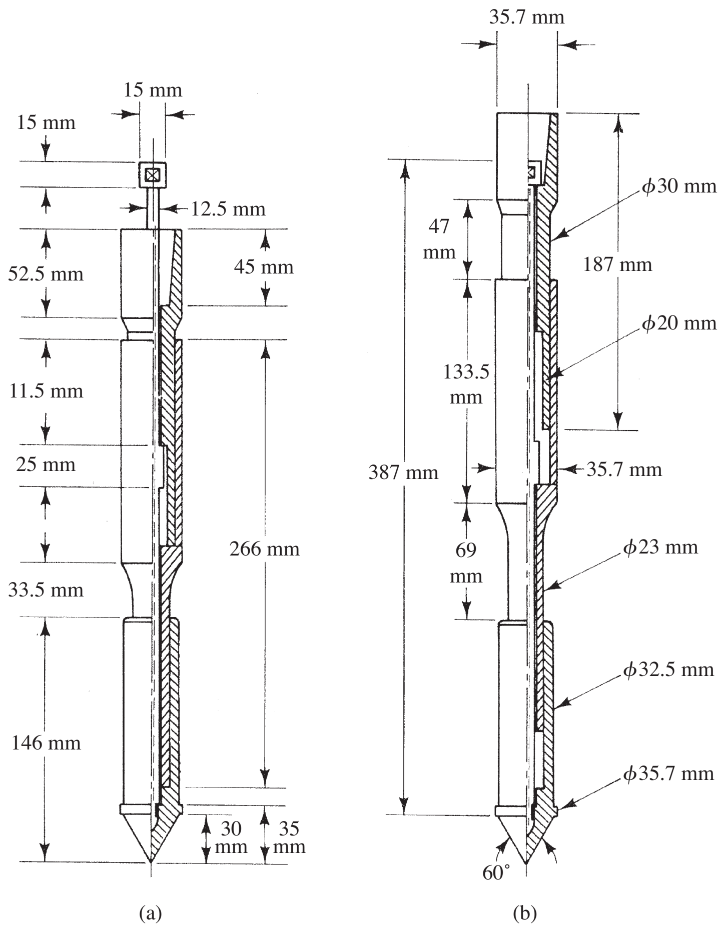


FIGURE 3–14 Mechanical friction-cone penetrometer tip (Begemann friction cone): (a) collapsed; (b) extended. *Source: Annual Book of ASTM Standards*, ASTM, Philadelphia, 2002. Copyright American Society for Testing and Materials. Reprinted with permission.

used to estimate the type of soil being penetrated. For example, Figure 3–19 classifies soils based on cone resistance and friction ratio for mechanical cone penetrometers. Similarly, Figure 3–20 classifies soils for electric friction-cone penetrometers. Using the results of a CPT test (such as Figures 3–17 and3–18) and the correlations of Figures 3–19 and 3–20, one can prepare a chart of soil type as a function of depth at the test site.

3–7 VANE TEST

The field vane test is a fairly simple test that can be used to determine in-place shear strength for soft clay soils—particularly those clay soils that lose part of their strength when disturbed (sensitive clays)—without taking an undisturbed sample. A vane tester (see Figure 3–21) is made up of two thin metal blades attached to a vertical shaft. The test is carried out by pushing the vane tester into the soil and then

FIGURE 3–15 Fugro electric friction cone.
Source: K. Terzaghi, R. B. Peck, and G. Mesri, *Soil Mechanics in Engineering Practice*, 3rd ed., John Wiley & Sons, Inc. New York, 1996, fig. 11–13(b), p. 49. Copyright © 1996 by John Wiley & Sons, Inc. Reprinted by permission of John Wiley & Sons, Inc.

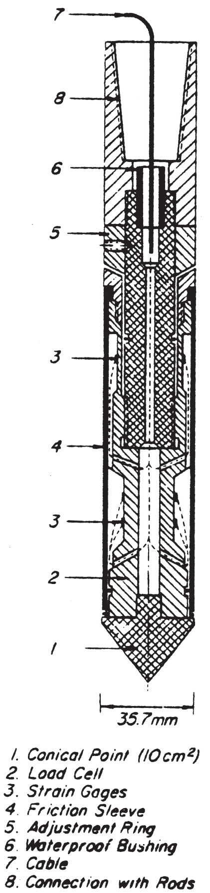
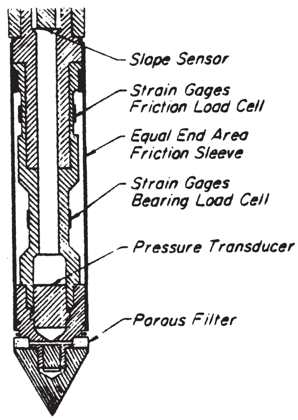


FIGURE 3–16 Piezocone.
Source: K. Terzaghi, R. B. Peck, and G. Mesri, *Soil Mechanics in Engineering* Practice, 3rd ed., John Wiley & Sons, Inc. New York, 1996, fig. 11–13(c), p. 49. Copyright © 1996 by John Wiley & Sons, Inc. Reprinted by permission of John Wiley & Sons, Inc.



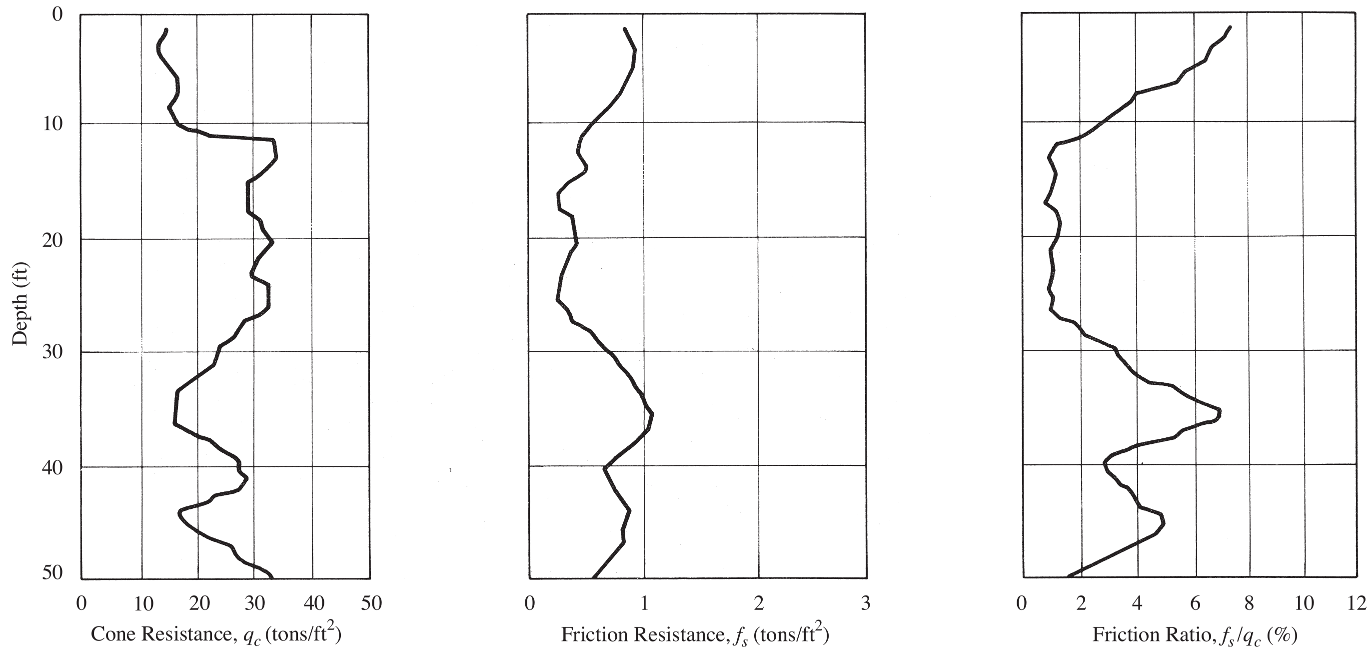


FIGURE 3–17 Sample CPT test results. These results were obtained from a mechanical friction cone.

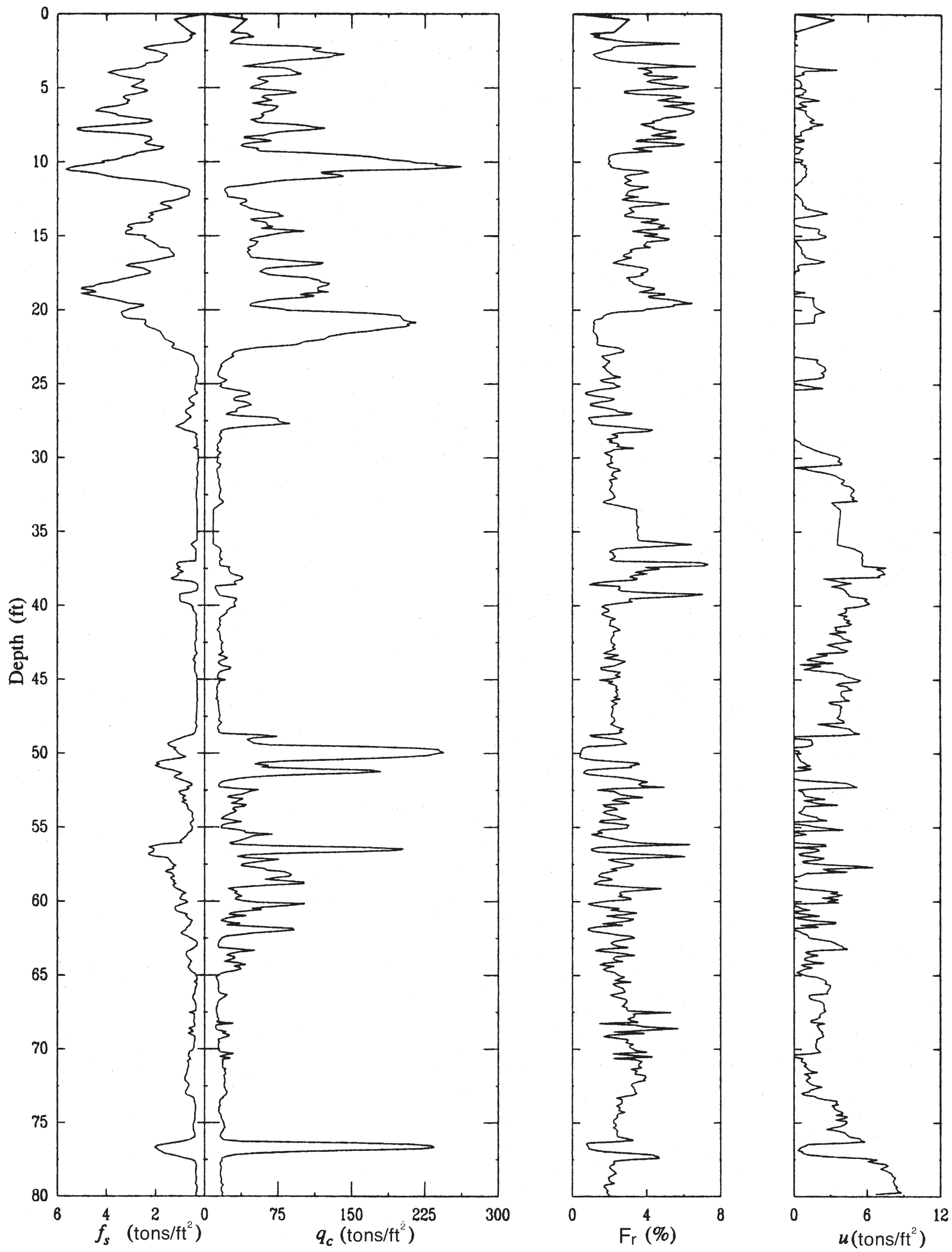


FIGURE 3–18 Sample CPT test results. These results were obtained from a piezocone and thus also include a plot of pore water pressure, u, vs. depth. All stresses and pressures are expressed in tons per square foot (tsf). *Source:* D. P. Coduto, *Geotechnical Engineering Principles and Practice*, fig. 3–29, p. 78 (1999).

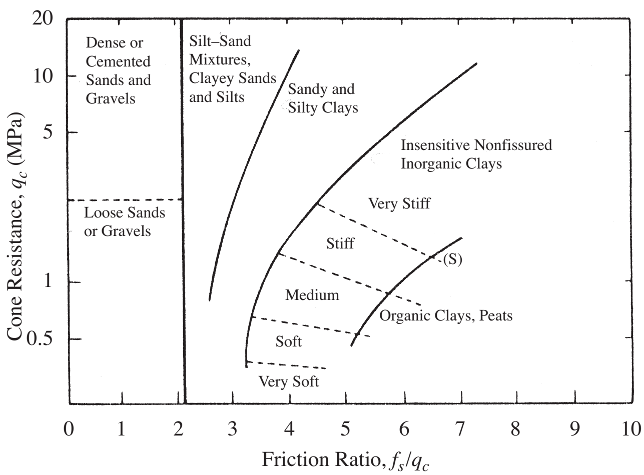


FIGURE 3–19 Soil classification based on Begemann cone penetrometer tests (mechanical cone).
Source: From K. Terzaghi, R. B. Peck, and G. Mesri, *Soil Mechanics in Engineering Practice*, 3rd ed., John Wiley & Sons, Inc., New York, 1996. Copyright © 1996, by John Wiley & Sons, Inc. Reprinted by permission of John Wiley & Sons, Inc.

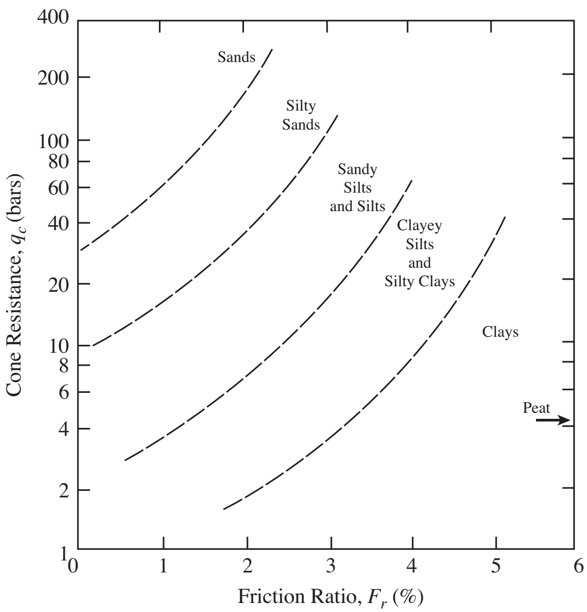


FIGURE 3–20 Simplified soil classification chart for standard electric friction cone.
Note: 1 bar = 100 kPa
Source: P. K. Robertson and R. G. Campanella, *Interpretation of Cone Penetration Test Part I*, 1983, fig. 2, p. 721. Reprinted by permission.

applying a torque to the vertical shaft. The clay's cohesion can be computed by using the following formula (Skempton and Bishop, 1950):

$$c = \frac{T}{\pi[(d^2h/2) + (d^3/6)]} \tag{3–4}$$

where c = cohesion of the clay, lb/ft^2 or N/m^2
T = torque required to shear the soil, ft-lb or m · N
d = diameter of vane tester, ft or m
h = height of vane tester, ft or m

FIGURE 3–21 Vane tester.

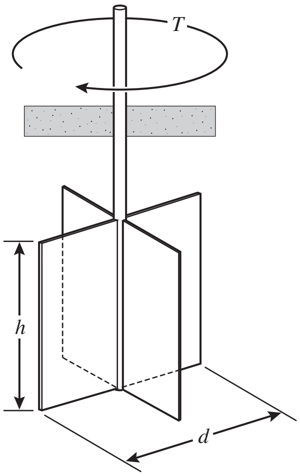
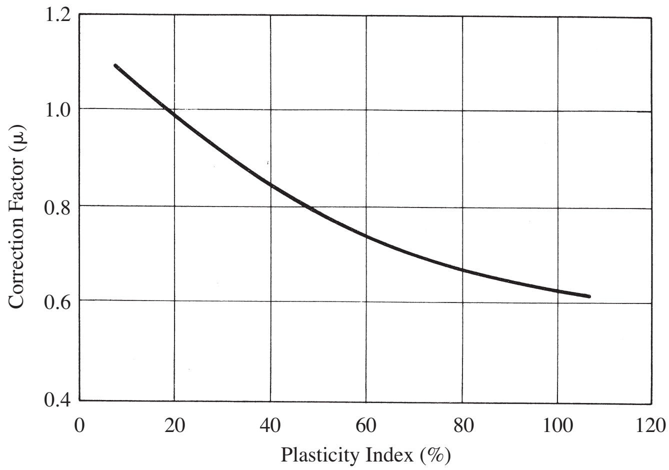


FIGURE 3–22 Correction factor for vane shear test. *Source:* L. Bjerrum, "Problems of Soil Mechanics and Construction on Soft Clays," *8th Int. Conf. SMFE*, Moscow, 1973. Reprinted in *Norwegian Geotechnical Institute Publication No. 100*, Oslo, 1974.



Bjerrum (1974) found a tendency of the vane test to overestimate cohesion in high plasticity clays and developed an empirical relationship for determining a correction factor. This relationship is shown in Figure 3–22, where a correction factor, μ, can be determined if the clay's plasticity index is known.

It should be emphasized that the field vane test is suitable for use only in soft or sensitive clays. Also, no soil sample is obtained for subsequent examination and testing when a field vane test is performed.

EXAMPLE 3–5

Given

A vane tester with diameter and height of 3.625 in. (0.3021 ft) and 7.25 in. (0.6042 ft), respectively, requires a torque of 17.0 ft-lb to shear a clayey soil, the plasticity index of which is 48%.

Required

This soil's cohesion.

Solution

By Eq. (3–4),

$$c = \frac{T}{\pi[(d^2h/2) + (d^3/6)]} \qquad \textbf{(3–4)}$$

$$c = \frac{17.0 \text{ ft-lb}}{\pi\left[\frac{(0.3021 \text{ ft})^2(0.6042 \text{ ft})}{2} + \frac{(0.3021 \text{ ft})^3}{6}\right]} = 168 \text{ lb/ft}^2$$

From Figure 3–22, with a plasticity index of 48%, a correction factor, μ, of 0.80 is obtained. Hence,

$$c_{\text{corrected}} = (0.80)(168 \text{ lb/ft}^2) = 134 \text{ lb/ft}^2$$

3–8 GEOPHYSICAL METHODS OF SOIL EXPLORATION

Borings and test pits (Section 3–3) afford definitive subsurface exploration. They can, however, be both time consuming and expensive. In addition, they give subsurface conditions only at boring or test pit locations, leaving vast areas in between for which conditions must be interpolated or estimated.

Geophysical methods, which are widely used in highway work and in other applications, can be implemented more quickly and less expensively and can cover greater areas more thoroughly. They tend, however, to yield less definitive results requiring more subjective interpretation by the user. Accordingly, a number of borings are still required to obtain soil samples from which accurate determinations of soil properties can be made in order to verify and complement results determined by geophysical methods.

Two particular geophysical methods—*seismic refraction* and *electrical resistivity*—are discussed in this section. In the former, resistance to flow of a seismic wave through soil is measured; in the latter, resistance of soil to movement of an electrical current is determined. Using values obtained therefrom, a specialist can interpret the depth to and thickness of different soil strata and estimate, with the aid of supplemental borings, some of the engineering properties of the subsurface material.

Seismic Refraction Method

The seismic refraction method is based on the fact that velocities of seismic waves traveling through soil and rock material are related to the material's density and elasticity. In general, the denser the material, the greater will be the velocity of seismic waves moving through it. In carrying out this method, seismic (sound or vibration) waves are created within the soil at a particular location. Ordinarily, these waves are produced either by exploding small charges of dynamite several feet below the ground surface or by striking a heavy hammer against a steel plate. A detector, known as a *geophone,* placed some known (or measurable) distance from the shock source, detects the presence of a wave, and a timing device measures the time required for the wave to travel from the point of impact to the point of detection.

In conducting a seismic refraction field survey, a series of geophone readings is obtained at different distances along a straight line from the point of impact. For detection points relatively close to the impact point, the first shock to reach the geophones travels from the impact point through more direct surface routes to the detection points (see Figure 3–23).

When a harder layer, say rock, underlies the surficial soil layer, a seismic wave traveling downward from the point of impact into the rock layer is refracted to travel longitudinally through the upper part of the rock layer and eventually back to the ground surface (through the surficial layer) to be recorded by the geophones (Figure 3–23). Because seismic wave velocity is much greater through the rock layer than through the surficial soil, for geophones located relatively far from the impact point, the refracted wave will reach the geophone more quickly than the direct wave. The time required for the first shock to reach each geophone is plotted as a function of distance from the shock source, as in Figure 3–24. The wave to the first few geophones closer to the shock source travels directly through the surficial layer; therefore, the slope of the time versus distance graph is inversely equivalent to velocity—that is,

$$v_1 = \frac{L_2 - L_1}{t_2 - t_1} \tag{3–5}$$

where v_1 = wave's velocity through the surficial soil layer (i.e., reciprocal of the slope of line 1 as shown in Figure 3–24)
L_1 and L_2 = distances from shock source to geophones Nos. 1 and 2, respectively (Figure 3–23)
t_1 and t_2 = times required for the first shock wave to reach geophones Nos. 1 and 2, respectively

Similarly, v_2 is the reciprocal of the slope of line 2 as shown in Figure 3–24. The thickness of stratum H_1 is given by

$$H_1 = \frac{L}{2}\sqrt{\frac{v_2 - v_1}{v_2 + v_1}} \tag{3–6}$$

where H_1 = depth of the upper layer (Figure 3–23)
L = distance taken from the time versus distance graph where the two slopes intersect (Figure 3–24)

As indicated in Table 3–5, wave velocities range from about 800 ft/s (244 m/s) in loose sand above the water table to 20,000 ft/s (6096 m/s) in granite and unweathered gneiss. This wide range makes possible a general assessment of the characteristics of material encountered.

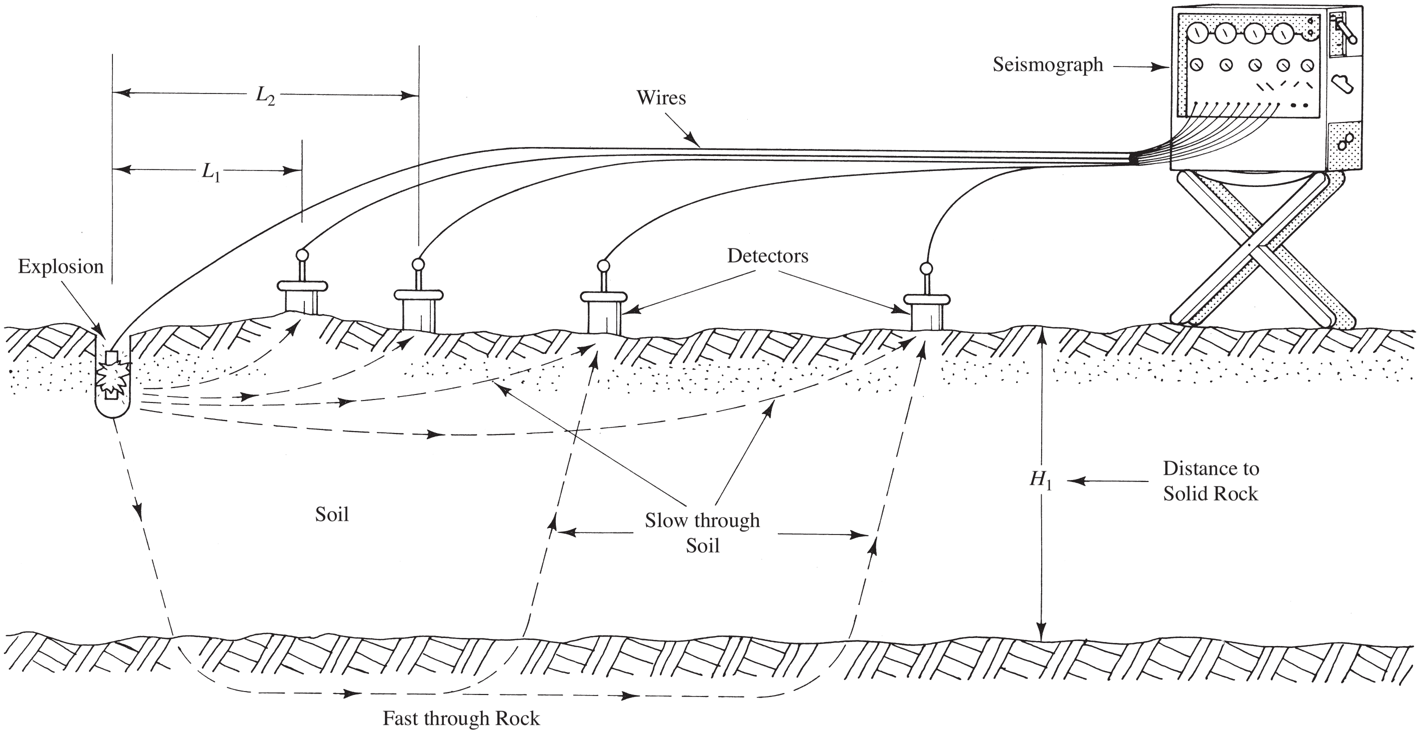
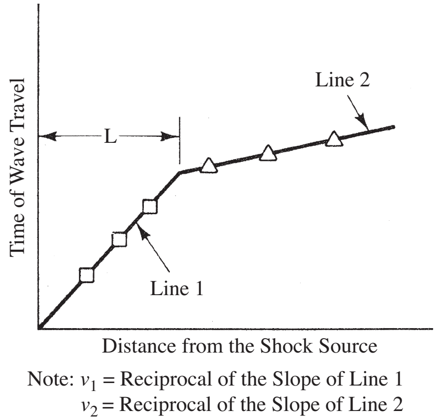


FIGURE 3–23 Seismic refraction test.
Source: R. W. Moore, "Geophysics Efficient in Exploring the Subsurface," *J. Soil Mech. Found. Div. Proc. ASCE*, SM3 (June 1961).

FIGURE 3–24 Time of wave travel as a function of distance from shock source in seismic refraction method.



Seismic refraction can be used to estimate depths to successively harder strata, but it will not determine softer strata below harder strata. It can also be used to find the depth to groundwater and to locate sinkholes. However, where boundaries are irregular or poorly defined, interpretation of the results of seismic refraction may be questionable.

Electrical Resistivity Method

As indicated initially in this section, resistance to movement of an electrical current through soil is determined in the electrical resistivity method. The premise for using this technique in subsurface investigations is that electrical resistance varies significantly enough among different types of soil and rock materials to allow identification of specific types if their resistivities are known.

A soil's resistivity generally varies inversely with its water content and dissolved ion concentration. Because clayey soils exhibit high dissolved ion concentrations, wet clayey soils have the lowest resistivities of all soil materials—as low as 5 ohm-ft (1.5 ohm · m). Coarse, dry sand and gravel deposits and massive bedded and hard bedrocks have the highest resistivities—over 8000 ohm-ft (2438 ohm · m). Table 3–6 gives the resistivity correlation for various types of soil materials.

One specific procedure for conducting an electrical resistivity field survey utilizes four equally spaced electrodes. (This is known as the *Wenner method.*) The four electrodes are placed in a straight line spaced distance D apart, as illustrated in Figure 3–25. An electrical current is supplied (by a battery or small generator) through the outer electrodes (Figure 3–25); its value is measured by an ammeter. The voltage drop in the soil material within the zone created by the electrodes' electric field is measured between the two inner electrodes by a voltmeter (Figure 3–25). The soil material's electrical resistivity can be computed by using the following equation:

$$\rho = 2\pi D \frac{V}{I} = 2\pi DR \tag{3–7}$$

TABLE 3–5
Representative Velocity Values[1–2]

Unconsolidated materials	ft/s	m/s
Most unconsolidated materials	Below 3000	Below 914
Soil		
Normal	800–1500	244–457
Hard packed	1500–2000	457–610
Water	5000	1524
Loose sand		
Above water table	800–2000	244–610
Below water table	1500–4000	457–1219
Loose mixed sand and gravel, wet	1500–3500	457–1067
Loose gravel, wet	1500–3000	457–914
Consolidated materials		
Most hard rocks	Above 8000	Above 2438
Coal	3000–5000	914–1524
Clay	3000–6000	914–1829
Shale		
Soft	4000–7000	1219–2134
Hard	6000–10,000	1829–3048
Sandstone		
Soft	5000–7000	1524–2134
Hard	6000–10,000	1829–3048
Limestone		
Weathered	As low as 4000	As low as 1219
Hard	8000–18,000	2438–5486
Basalt	8000–13,000	2438–3962
Granite and unweathered gneiss	10,000–20,000	3048–6096
Compacted glacial tills, hardpan, cemented gravels	4000–7000	1219–2134
Frozen soil	4000–7000	1219–2134
Pure ice	10,000–12,000	3048–3658

[1]Courtesy of Soiltest, Inc.
[2]Occasional formations may yield velocities that lie outside these ranges.

where ρ = resistivity of the soil material, ohm-ft or ohm · m
D = electrode spacing, ft or m
V = voltage drop between the two inner electrodes, volts
I = current supplied through the outer electrodes, amperes
R = resistance, ohms

The zone created by the electrodes' electrical field extends downward to a depth approximately equal to the electrode spacing (i.e., D in Figure 3–25).

TABLE 3–6
Resistivity Correlation[1]

Ohm-ft	2π Ohm · cm	Types of Materials
5–10	1000–2000	Wet to moist clayey soils
10–50	3000–15,000	Wet to moist silty clay and silty soils
50–500	15,000–75,000	Moist to dry silty and sandy soils
500–1000	30,000–100,000	Well-fractured to slightly fractured bedrock with moist-soil–filled cracks
1000	100,000	Sand and gravel with silt
1000–8000	100,000–300,000	Slightly fractured bedrock with dry-soil–filled cracks; sand and gravel with layers of silt
8000 (plus)	300,000 (plus)	Massive bedded and hard bedrock; coarse, dry sand and gravel deposits

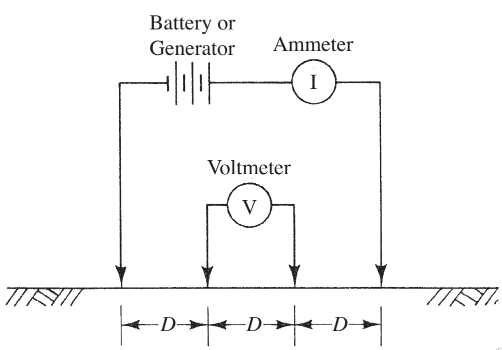
[1]Courtesy of Soiltest, Inc.

Consequently, the depth of subsurface material included in a given measurement is approximately equal to the spacing between electrodes. The resistivity determined by this method [computed by Eq. (3–7)] is actually a weighted mean value of all soil material within the zone.

A single application of the procedure just outlined would give an indication of the "average" type of subsurface material within the applicable zone. To determine depths of strata of different resistivities, the procedure is repeated for successively increasing electrode spacings (see Figure 3–26). Because the applicable zone's depth varies directly with electrode spacing, data obtained from successively increasing electrode spacings should indicate changes in resistivity with depth, which in turn serves to locate different soil strata.

Resistivity data can be analyzed by plotting $\Sigma\rho$ (summation of soil resistivity values) versus electrode spacing (D) for increasing electrode spacings. Such a plotting is illustrated in Figure 3–27. A straight-line plot indicates a constant soil resistivity

FIGURE 3–25 Electrode configuration for electrical resistivity test.



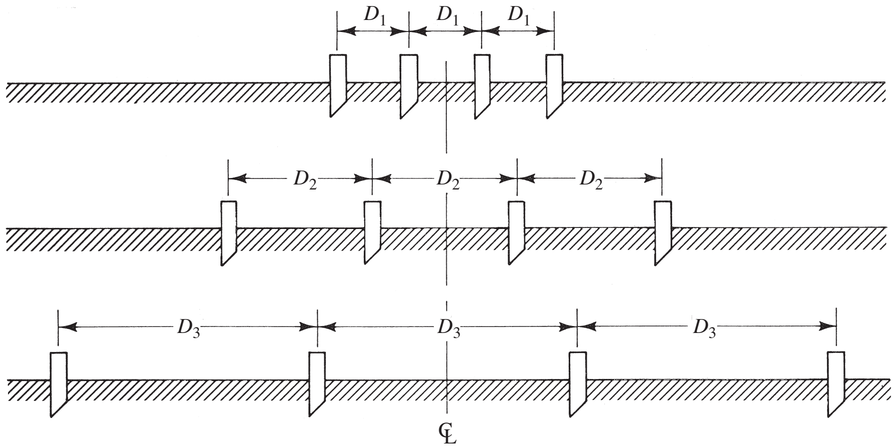


FIGURE 3–26 Representative electrode positions during a sequence of sounding measurements (the position of the center of the spread is fixed).
Source: Courtesy of Soiltest, Inc.

(and therefore the same soil type) within the depth range for which the plot is straight. Furthermore, the slope of the straight line is equal to ρ_1/D, and ρ_1 gives the resistivity in the upper layer. Using this value of resistivity, one can estimate the type of soil within this layer. If a different soil type is encountered as additional tests are performed at increasing electrode spacings, a second straight-line plot should result with a slope equal to ρ_2/D_1, with ρ_2 giving the resistivity of the lower layer, from which the type of soil can be evaluated. Furthermore, the intersection of the two straight lines gives the approximate depth of the boundary between the two layers (Figure 3–27).

The electrical resistivity method can be used to indicate subsurface variations where a hard layer underlies a soft layer; however, unlike the seismic refraction method, it can also be used where a soft layer underlies a hard layer. The electrical resistivity method can be used not only to estimate depth to strata of different resistivities but also to find depth to groundwater and to locate masses of dry sands, gravels, and rock. It should be realized that errors in interpretation can occur because soil resistivity varies with moisture content and identifies soil only indirectly. Hence, the electrical resistivity method should always be used with confirmatory drilling.

As related at the beginning of this section, geophysical methods afford relatively rapid and low-cost subsurface exploration as compared with test borings. However, dependable results from geophysical methods require experienced and skillful interpretation of test data. Geophysical methods have some disadvantages. The greatest is that, because of the subjectivity involved in analyzing, interpreting, and drawing conclusions from collected data, the resulting picture of the area's subsurface features may not be entirely accurate. Accordingly, geophysical methods should always be used in conjunction with test borings—either using

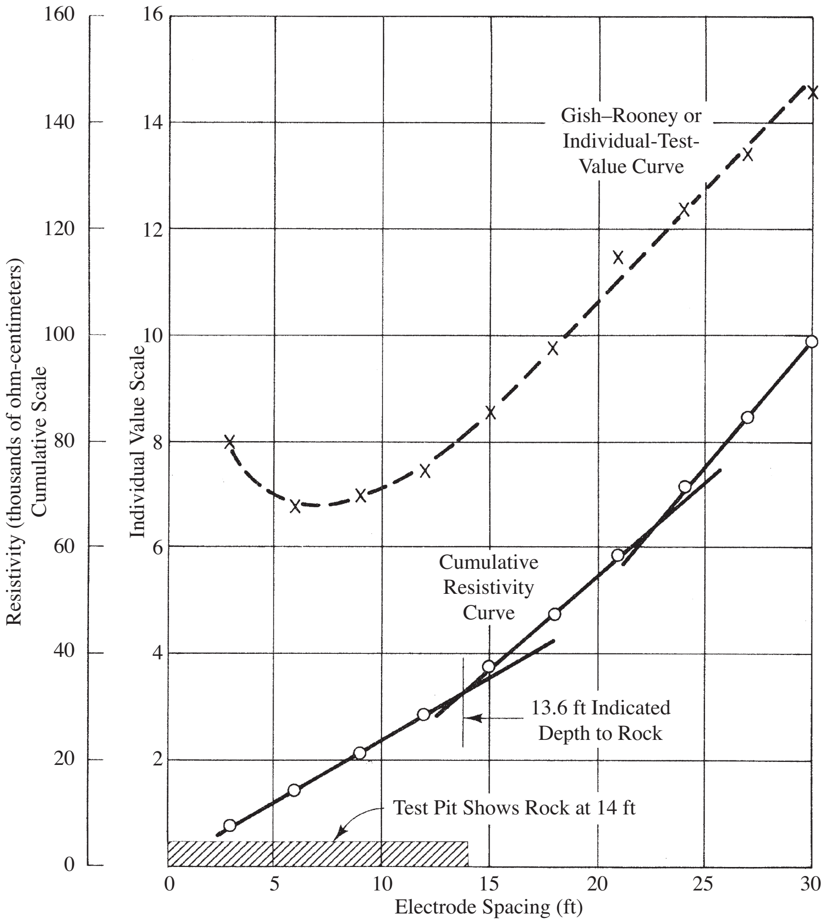


FIGURE 3–27 Typical resistivity data and method of analysis using the cumulative resistivity curve.
Source: R. W. Moore, "Geophysics Efficient in Exploring the Subsurface," *J. Soil Mech. Found. Div. Proc. ASCE*, SM3 (June 1961).

sufficient test borings to verify results of geophysical methods or using geophysical methods to provide intermediate subsurface information between adjacent test borings.

3–9 RECORD OF SOIL EXPLORATION

It is of utmost importance that complete and accurate records be kept of all data collected. Boring, sampling, and testing are often costly undertakings, and failure to keep good, accurate records not only is senseless, but also may be dangerous.

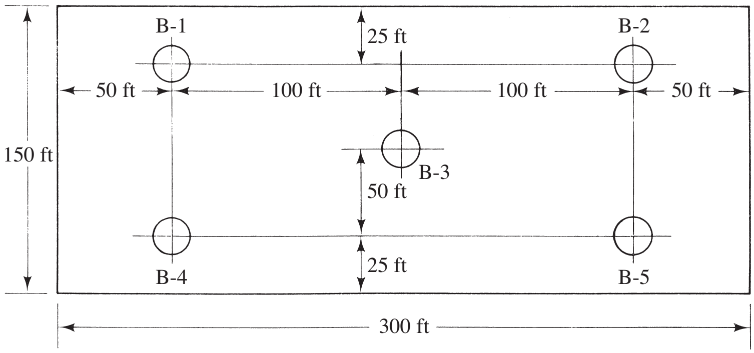


FIGURE 3–28 Example map showing boring locations on 150-ft by 300-ft construction site.

To begin with, a good map giving specific locations of all borings should be available. Each boring should be identified (by number, for example) and its location documented by measurement to permanent features. Such a map is illustrated in Figure 3–28.

For each boring, all pertinent data should be recorded in the field on a boring log sheet. Normally, these sheets are preprinted forms containing blanks for filling in appropriate data. An example of a boring log is given in Figure 3–29.

Soil data obtained from a series of test borings can best be presented by preparing a geologic profile, which shows the arrangement of various layers of soil as well as the groundwater table, existing and proposed structures, and soil properties data (SPT values, for example). Each borehole is identified and indicated on the geologic profile by a vertical line. An example of a geologic profile is shown in Figure 3–30.

A geologic profile is prepared by indicating on each borehole on the profile (i.e., each vertical line representing a borehole) the data obtained by boring, sampling, and testing. From these data, soil layers can be sketched in. Obviously, the more boreholes and the closer they are spaced, the more accurate the resulting geologic profile.

3–10 CONCLUSION

The subject of this chapter should be considered as one of the most important in this book. Analysis of soil and design of associated structures are of questionable value if the soil exploration data are not accurately determined and reported.

The authors hope this chapter will give the reader an effective introduction to actual soil exploration. However, learning to conduct soil exploration well requires much practice and varied experience under the guidance of experienced practitioners. Not only is it a complex science, it is a difficult art.

Sheet 1 of 1

ABC DRILLING COMPANY, INC.
NEWARK, NEW YORK

BORING NO. 5
ORD. ELEV. 372.4

PROJECT: Job No. 459
Name Eureka Warehouse
Address Illion, New York

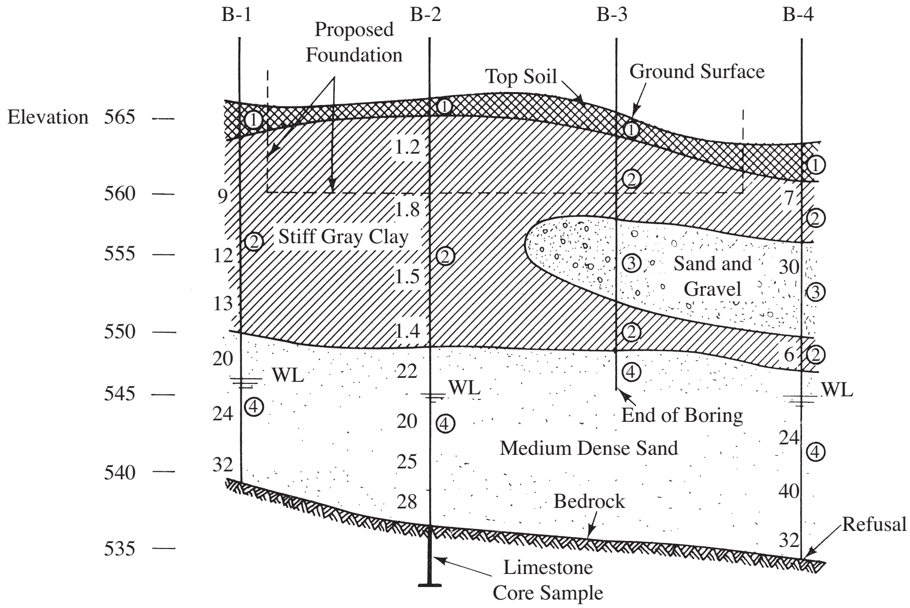
CASING (Size & Type) 2½" Drive Pipe
SAMPLE SPOON (Size & Type) 2" O.D.S.S.
HAMMER (Csg): Wt. 250 lb, Drop 24 in.
(Spoon): Wt. 140 lb, Drop 30 in.
DATE: Started 7/28/– Completed 7/29/– Driller Henry James

GROUND WATER OBSERVATIONS

Date	Time	Depth	Casing at
7/29/–	3:00 PM	18'3"	15'0"
"	4:00 PM	12'0"	10'0"
"	4:30 PM	8'0"	5'0"
7/30/–	8:30 PM	7'0"	Out

DEPTH FT.	BLOWS CSG.	BLOWS SPOON		Samples	
0–1	2				Black and grey moist FILL: cinders, brick, and silt
1–2	16	11 / 8			
2–3	9	4	12	S #1, 1'–2'6"	3'0"
3–4	3	1 / 1			Black PEAT
4–5	3	2	3	S #2, 3'–4'6"	
5–6	3	P / 1			6'0"
6–7	5	1	2	S #3, 5'–6'6"	Grey moist SILT with embedded fine gravel, trace of fine sand
7–8	6	3 / 6			
8–9	8	5	11	S #4, 7'–8'6"	
9–10	9				
10–11	3	4 / 8			
11–12	8	6	14	S #5, 10'–11'6"	12'6"
12–13	15	15			Weathered SHALE
13–14	32	18 / 21	39	S #6, 12'6"–14'	
14–15	78				TOP OF ROCK 15'0"
15–20				Core Boring Series M—double tube core barrel, 2-in.-diameter bit	Weathered grey SHALE Run #1, 15'0" – 20'0" Recovered 30" – 50% Lost water @ 16'6" 20'0"
20–25					SHALE and SANDSTONE Run #2, 20'0" – 25'0" Recovered 56" – 93% Steady resistance while drilling 25'0"

FIGURE 3–29 Boring log sheet.
Source: B. K. Hough, *Basic Soils Engineering*, 2nd ed., The Ronald Press Company, New York, 1969.



Note: 1 2 , . . . = Top Soil, Stiff Gray Clay, . . .

9, 12 , . . . = Standard Penetration Resistance (Number of Blows/ft)

1.2, 1.8 , . . . = Unconfined Compressive Strength (tons/ft^2)

FIGURE 3–30 Example of geologic profile.
Source: W. C. Teng, *Foundation Design*, 1962. Reprinted by permission of Pearson Education, Upper Saddle River, NJ.

3–11 PROBLEMS

3–1. A 4-ft square footing is subjected to a contact pressure of 6000 lb/ft^2. The wet unit weight of the cohesive soil supporting the footing is estimated to be 118 lb/ft^3, and groundwater is known to be at a great depth. Determine the minimum depth of test boring based on the criterion that test borings in cohesive soils should be carried at least to a depth where the increase in stress due to the foundation loading is less than 10% of the effective overburden pressure.

3–2. A standard penetration test (SPT) was performed at a depth of 10 ft in sand of unit weight 120 lb/ft^3. The *N*-value was found to be 26. Determine the corrected *N*-value by the two methods presented in this chapter.

3–3. Rework Problem 3–2 if groundwater is located 8 ft below the ground surface.

3–4. An SPT was performed at a depth of 7 m in sand of unit weight 20.40 kN/m^3. The *N*-value was found to be 22. Compute the corrected *N*-value by the two methods presented in this chapter.

3–5. Rework Problem 3–4 if groundwater is located 2 m below the ground surface.

3–6. A field vane test was performed in a soft, sensitive clay layer. The vane tester's diameter and height are 4 and 8 in., respectively. The torque required to shear the clay was 61 ft-lb. Determine the clayey soil's cohesion if its plasticity index is known to be 40%.

3–7. Soil exploration was conducted at a construction site by seismic refraction, with field readings obtained as listed next:

Distance (ft)	Time (ms)
20	21
40	42
60	62.25
80	83
100	86.75
120	88.25
140	89.25
160	90.75
180	93

Estimate the thickness and type of material of the first soil layer and the type of material in the underlying second layer.

3–8. Soil exploration was conducted at a construction site by the electrical resistivity method, with field data obtained as follows:

Electrode Spacing (ft)	Resistance Readings (ohms)
10	12.73
20	2.79
30	1.46
40	1.15
50	1.05
60	0.84
70	1.21
80	1.00
90	0.97
100	0.95

Estimate the thickness and type of material of the first soil layer and the type of material in the underlying second layer.