CHAPTER 7

In Situ Tests

This chapter is devoted to the description of in situ tests and the test data that they generate. This chapter does not describe which soil properties can be inferred by correlation or other means from the test results; those correlations are discussed in Chapters 13, 14, and 15, dedicated to these soil properties. This chapter also does not describe the design methods that make use of in situ test results; this is covered in Chapters 17, 18, and 22, dedicated to design methods.

In situ tests are tests conducted on or in the soil at the site. They have been developed over the years as a complement to laboratory testing. Indeed, the drawbacks of laboratory tests are typically balanced by the advantages of in situ tests and vice versa (Table 6.1). Therefore, the best site investigation program uses a combination of in situ tests and laboratory tests. The most commonly used in situ tests are the standard penetration test, the field vane tests, the cone penetration test, the pressuremeter test, and the dilatometer test. Many other tests also exist, as shown in Figure 7.1 (Mayne et al. 2009).

7.1 STANDARD PENETRATION TEST

The standard penetration test (SPT) is the oldest of the in situ tests, and can be credited to Charles Gow in the United States who started developing it in 1902. After several decades of use, it was standardized in the mid-1930s (ASTM D1586). Today, the SPT (Figure 7.2) consists of driving a split spoon sampler into the soil using a standard 623N hammer falling from a height of 0.76 m onto an anvil at the top of the rods. The oldest hammer was the donut hammer, followed by the safety hammer and more recently the automatic hammer (Figure 7.3). For the donut hammer and the safety hammer, a person would raise the hammer with a rope. The rope would be wrapped around a cathead system (rotating drum) and the person would pull and release the rope to raise and drop the hammer in rhythm at about one blow per second. In the case of the automatic hammer, the hammer is raised automatically by a hydraulic jack. The rated energy of each blow is 623N × 0.76 m or 473 joules.

The rope-and-cathead system for the donut hammer and the safety hammer generate friction and other energy losses that decrease the amount of energy delivered to the split spoon sampler. Measurements have indicated that the mean energy actually delivered by these systems is around 285 J, or 60% of the maximum energy (ASTM D1586). Thus, the blow count N is often referred to as N_{60}. Because so much experience has been accumulated with these older systems, most correlations refer to N_{60}. However, automatic hammers may have much lower losses, so one should be careful about using the blow count N obtained with an automatic hammer without paying attention to this difference. The impact of the hammer on the anvil creates a compression wave in the steel rods which propagates at some 21,000 km/h (this, by the way, approaches the speed of the space shuttle in free space). The number of blows N_a necessary to drive the split spoon sampler 0.15 m into the soil is recorded. The SPT test continues and the number of blows N_b necessary to drive the sampler another 0.15 m is recorded. The SPT test continues and the number of blows N_c needed to drive the sampler yet another 0.15 m is recorded. The SPT blow count N (blows/0.3 m) is the sum of $N_b + N_c$, as N_a is considered to be a set of seating blows. A typical profile of SPT results is shown in Figure 7.4.

For design purposes, the N value is often corrected to account for influencing factors such as the energy level, the stress level, and the presence of silt (Table 7.1). Additional correction factors take into account the length of the rods, the diameter of the borehole, and whether or not the sampler has a liner. As explained earlier, the maximum energy that can be delivered by an SPT hammer system is 473 J (623 $N \times$ 0.76 m). If N_{measured} is the field value (N_b plus N_c, as explained earlier), N_{measured} corresponds to the energy (E) measured in the field, E_{measured}. To obtain the N_{60} value corresponding to 60% of the maximum energy that can be delivered by the system (0.6×435 J = 285 J), a linear interpolation is done as follows:

$$N_{60} = N_{\text{measured}} \left(\frac{E_{\text{measured}}\ (\text{J})}{285\ \text{J}} \right) \tag{7.1}$$

Geotechnical Engineering: Unsaturated and Saturated Soils Jean-Louis Briaud

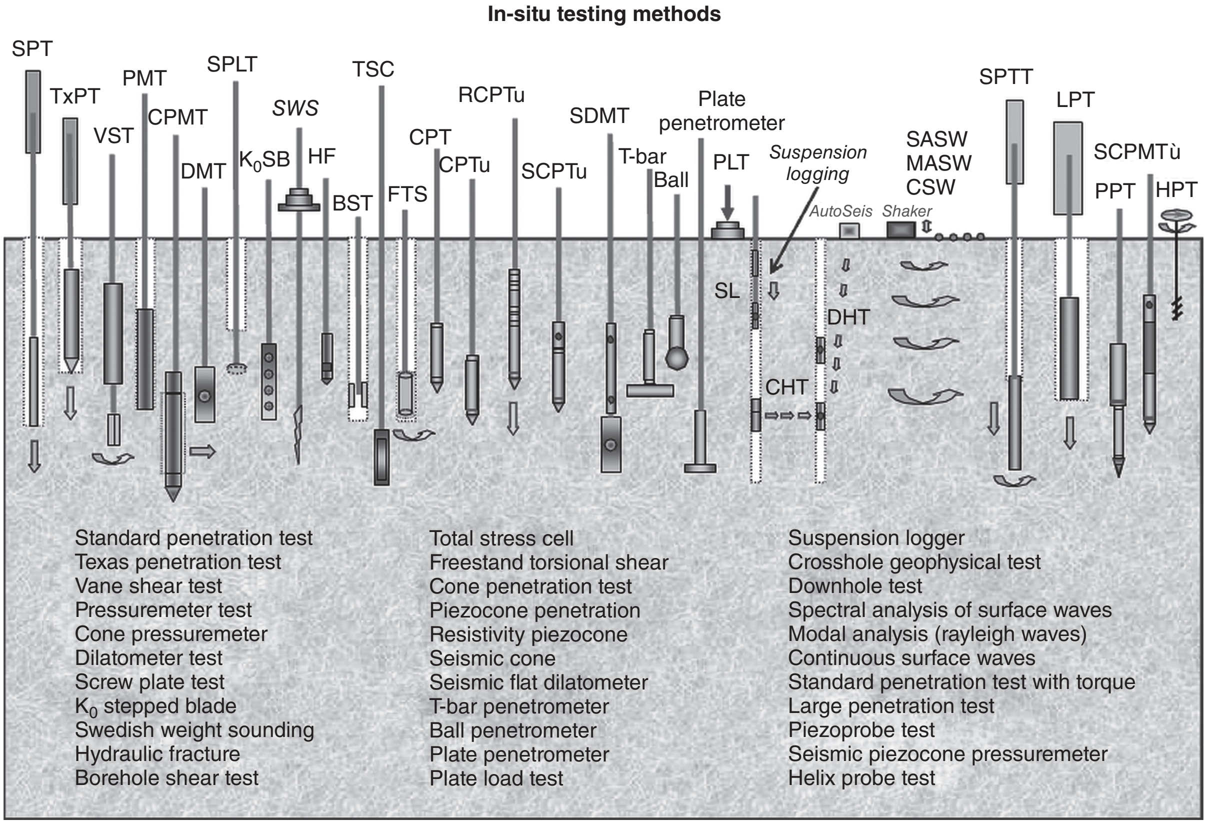


Figure 7.1 In situ tests. (Courtesy of Professor Paul Mayne, Georgia Institute of Technology, USA.)

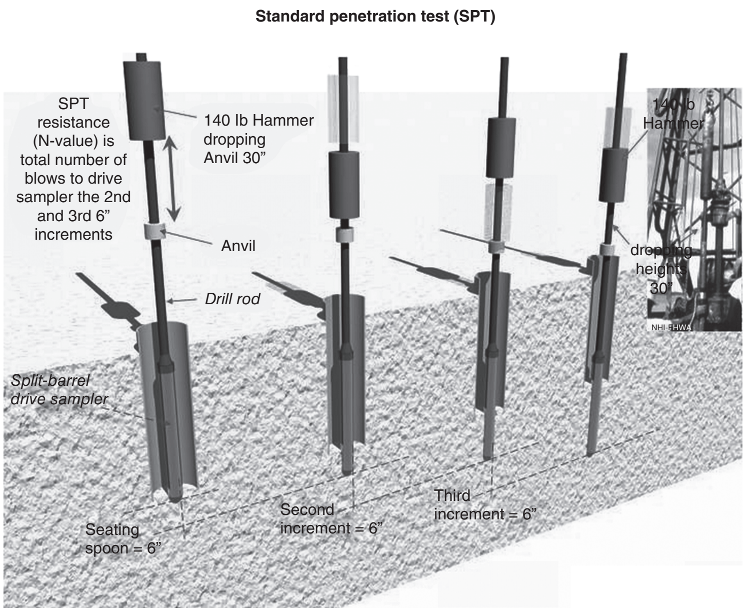


Figure 7.2 Standard penetration testing sequence. (Courtesy of Professor Kamal Tawfiq, Florida State University, USA.)

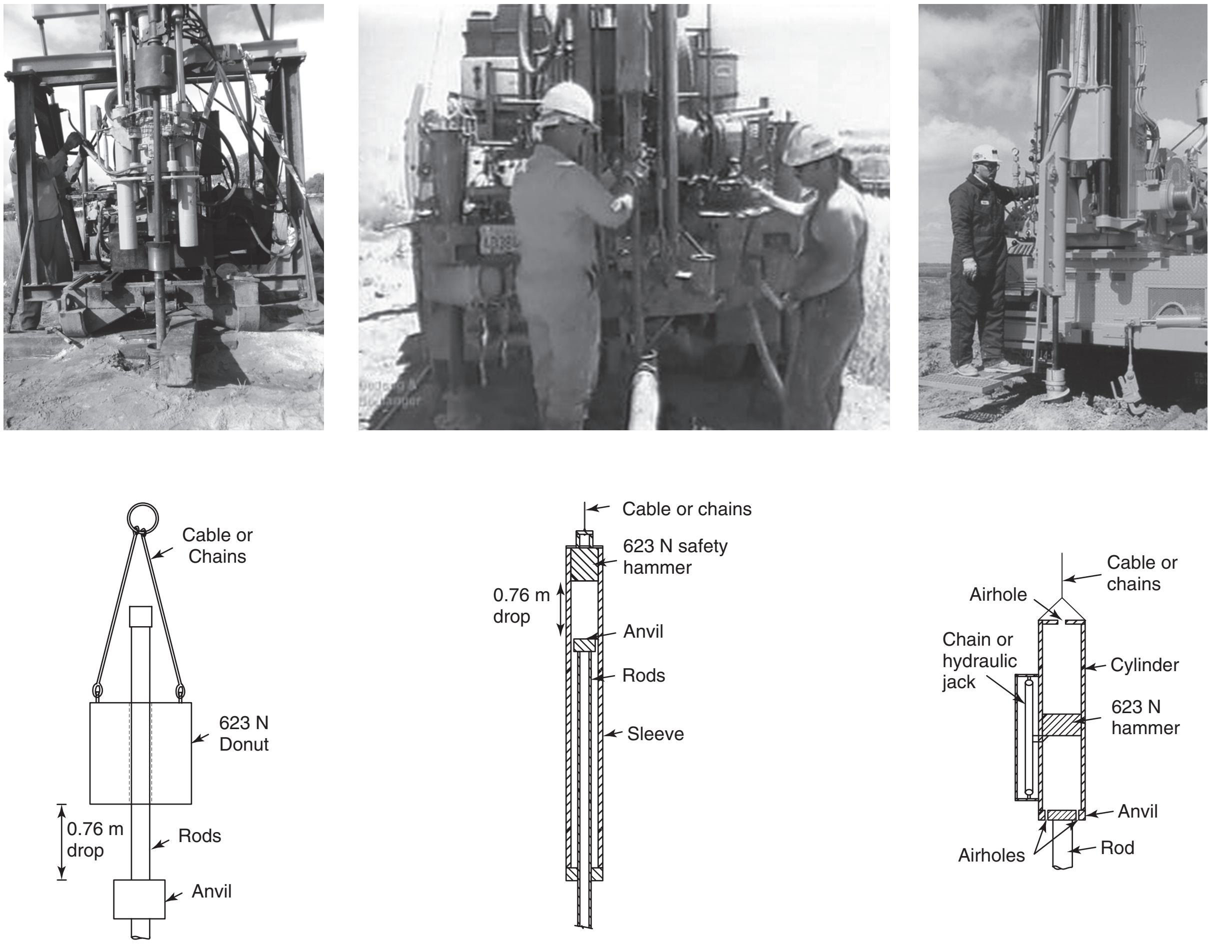


Figure 7.3 Standard penetration test hammers. (a: Courtesy of Fugro, b: Photo from Bray et al. 2001. Used by permission. c: Central Mine Equipment Co.)

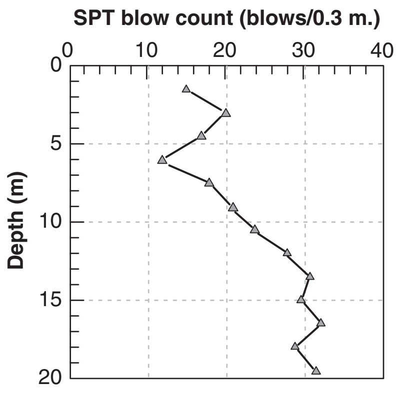


Figure 7.4 Example of SPT sounding result.

N_{measured} also corresponds to the vertical effective stress at rest σ'_{vo} at the depth of the test. To obtain the N_1 value corresponding to a reference value of σ'_{vo} equal to 100 kPa, a power law interpolation is used:

$$N_1 = N_{\text{measured}}\left(\frac{100}{\sigma'_{vo}\,(\text{kPa})}\right)^{0.5} \tag{7.2}$$

Table 7.1 Correction of the SPT Blow Count Value *N*

Energy level	$N_{60} = N_{\text{measured}} \times \left(\frac{E_{\text{measured}}{}^{*}}{285\,\text{J}}\right)$
Stress level	$N_1 = N_{\text{measured}} \times \left(\frac{100\,\text{kPa}}{\sigma'_{vo}{}^{**}}\right)^{0.5}$ kPa
High silt content and effect of capillary	$N' = 15 + \left(\frac{N_{\text{measured}} - 15}{2}\right)$

$^{*}E_{\text{measured}}$ must be in joules
$^{**}\sigma'_{\text{vo}}$ must be in kPa

$\text{N}_{\text{measured}}$ is sometimes corrected for silt content as follows:

$$N' = 15 + \left(\frac{N_{\text{measured}} - 15}{2}\right) \tag{7.3}$$

Note that the decision to correct or not correct the N value requires engineering judgment. In general, N_{60} should always be used as a standardizing method, but this requires to measure the actual energy which is rarely done. If one needs to evaluate the friction angle φ of the soil, then N_1 should be used because N includes the effect of stress level, while φ does not. However, if one uses N in a direct design such as an ultimate bearing capacity equation of the form $p_u = kN + \gamma D$, then N should not be corrected for stress level, as the stress level is part of the soil resistance in both the SPT and the foundation capacity. Liquefaction charts do include most of the correction factors for N.

In the United States, the number N is used extensively in the design of structures over sand and gravel, but it is not used with silts and clays because it is felt that a better approach is possible, such as taking undisturbed samples. Some other countries, like Brazil, extend use of the SPT to silts and clays. Applications include settlement and ultimate bearing pressure for shallow and deep foundations, soil properties such as shear strength parameters and modulus values, and liquefaction potential. The advantages of the SPT include that it is a rugged test which can nearly always be performed and give results; that it is performed with the same drill rig used to collect samples; that it has been used for a long time and thus is well known and understood; and that it yields both an evaluation of strength and a sample for identification purposes at the same time. A primary drawback is that the amount of energy reaching the sample can vary quite a bit.

7.2 CONE PENETRATION TEST

The development of the cone penetration or penetrometer test (CPT) started in the early 1930s in the Netherlands, and can be credited to Pieter Barentsen, who performed the first CPT in 1932. At that time, a mechanical cone was used (Briaud and Miran 1992a; ASTM D3441), but in the mid-1950s electronic cones came into use (Mayne 2007a, b; ASTM D5778). Today, the CPT (Figure 7.5) consists of pushing a 35.6 mm diameter instrumented rod into the soil at 20 mm/s. A drill rig, or more commonly a truck, weighing as much as 200 kN provides the vertical reaction (Figure 7.6). At the bottom of the rods is the instrumented cone tip (Figure 7.7), which can be equipped with different sensors to make many measurements. The two primary measurements are the tip resistance q_c at the point of the cone and the sleeve friction f_s on a sleeve right behind the point. Note that the measured tip resistance i should be corrected for the influence of water pressure inside the cone to obtain the total cone tip resistance q_t (Mayne 2007a). Examples of continuous profiles obtained with the CPT are shown in Figure 7.8. Other possible measurements include water pressure measurements, shear wave velocity, electrical resistivity, inclination, sound level, lateral stress, camera, radio isotope, and temperature. The CPT can also be equipped with a soil and water sampler. The most common location for water pressure measurement is right behind the cone point (Figure 7.7); the measurement is made through a saturated porous element behind which a transducer senses the compression in the water as the cone advances. The shear wave velocity is typically measured between the surface and a geophone located in the rods and sensing the arrival of a shear wave generated at the ground surface (Figure 7.9). The electrical resistivity is measured between two electrodes mounted on the rods and separated by a nonconducting material to force the electrical current to go through the soil instead of through the rods.

The cone penetrometer point resistance q_t is influenced by the stress level surrounding the point where the test is performed and by the properties of the soil in the vicinity of that location. To use a cone parameter that is dependent only on an intrinsic soil property, it is desirable to

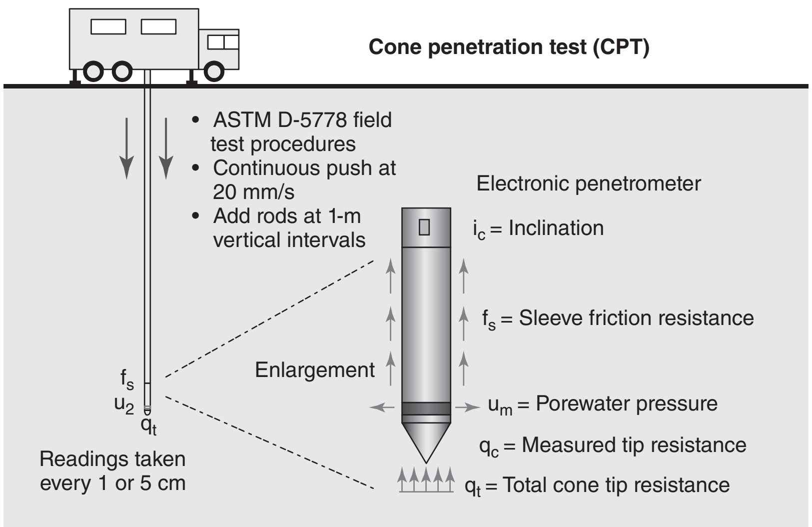


Figure 7.5 Cone penetrometer test. (From Mayne 2007a. Courtesy of Professor Paul Mayne, Georgia Institute of Technology)

Figure 7.6 Cone penetrometer truck. (From Mayne 2007a. Courtesy of Professor Paul Mayne, Georgia Institute of Technology.)

correct the q_t value for the stress level, as was done for the SPT (equation 7.2). The following corrections may be recommended.

For sands:

$$q_{t1} = \left(\frac{q_t}{\sigma_a}\right)\left(\frac{\sigma_a}{\sigma'_{vo}}\right)^{0.5} = \frac{q_t}{(\sigma'_{vo}\sigma_a)^{0.5}} \qquad (7.4)$$

For clays:

$$q_{t1} = \frac{q_t - \sigma_{vo}}{\sigma'_{vo}} \qquad (7.5)$$

where q_{t1} is the dimensionless corrected normalized CPT point resistance, q_t the total CPT point resistance, σ'_{vo} and σ_{vo} the vertical effective stress and vertical total stress at the depth of the cone respectively, and σ_a the atmospheric pressure used to nondimensionalize equation 7.4. The reason for not using Eq. 7.4 for clays is that the undrained shear strength of clays has been shown to be linearly proportional to the vertical effective stress. Alternatively, a progressive transition between the two soil types can be used through equation 7.6, which also includes a fine content influence factor K_c useful in liquefaction studies.

$$q_{t1} = \left(\frac{q_t}{\sigma_a}\right)\left(\frac{\sigma_a}{\sigma'_{vo}}\right)^{n} K_c \qquad (7.6)$$

where n is 0.5 for sand, 0.7 for silty sand, 0.8 for silt, and 1 for clay, and K_c is a fine content factor gradually varying from 1 to 1.5 for clean sands, 1.5 to 3.5 for silty sands, and 3.5 to 6 for silts.

The most useful application of the CPT is stratigraphy, because the CPT penetration resistance profile gives the engineer a continuous display of the strength of the deposit. Note that the scale of the cone influences the thickness of the layer that can be detected, as well as the strength of that layer. If a layer is smaller than about 10 times the diameter of the cone, the tip resistance will not reach the value that would be obtained if the layer were infinitely deep. Associated with stratigraphy is the ability to classify the soil on the basis of the friction ratio, that is, the ratio of the sleeve friction over the

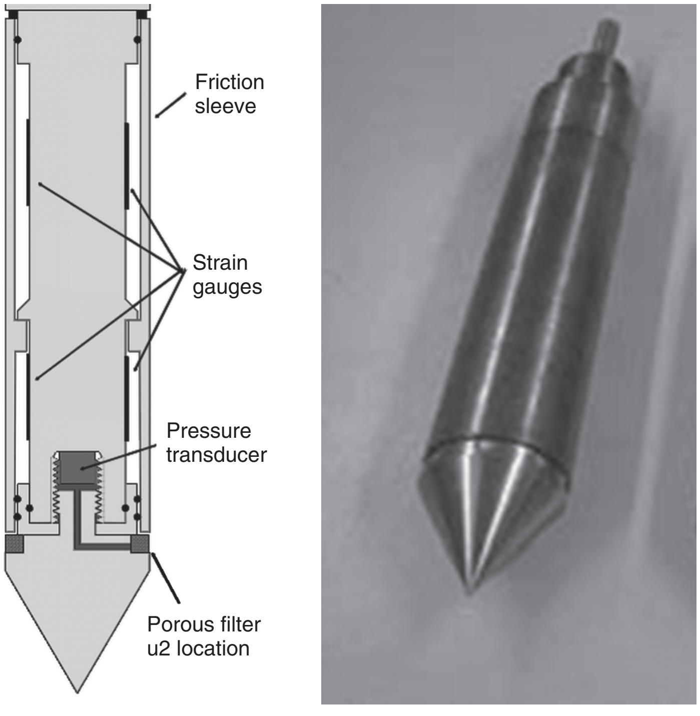


Figure 7.7 Cone penetrometers. (From Mayne 2007a. Courtesy of Professor Paul Mayne, Georgia Institute of Technology.)

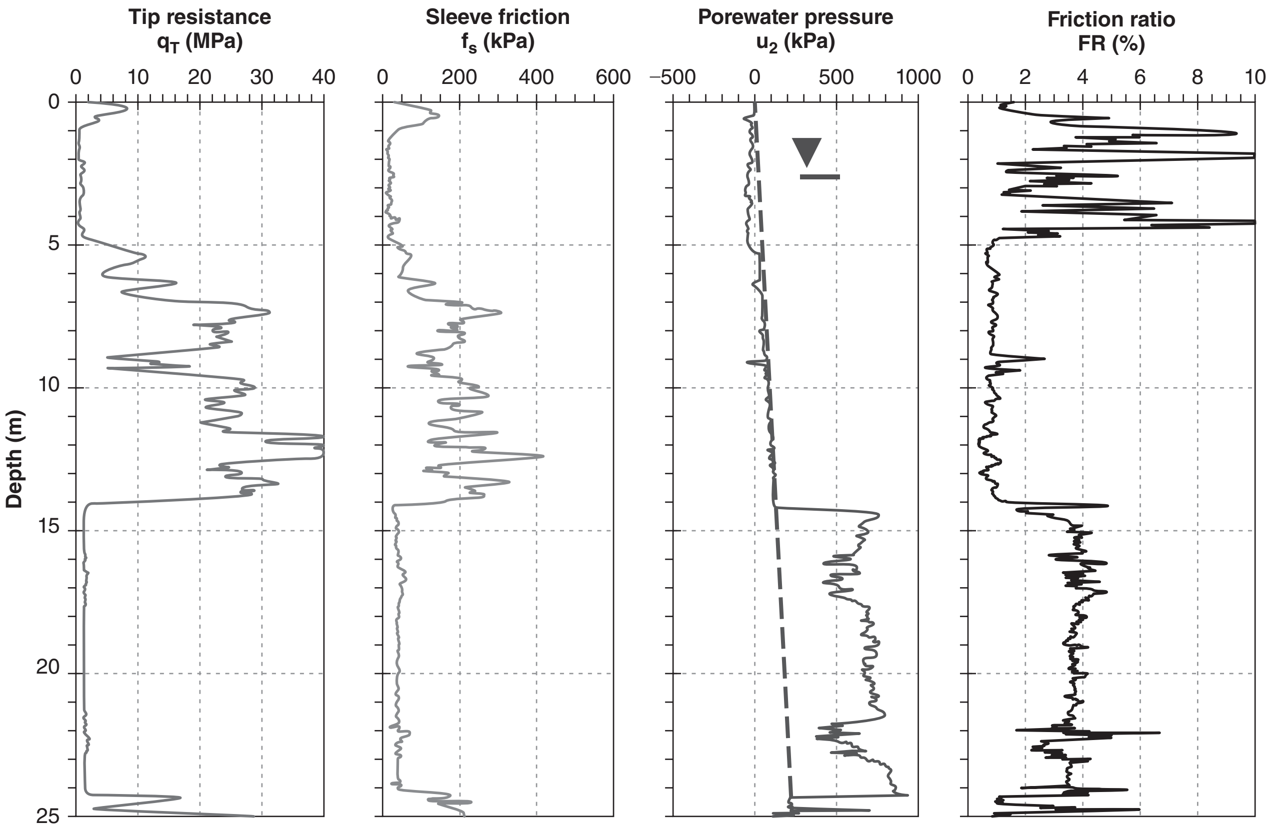


Figure 7.8 Examples of CPT profiles. (From Mayne 2007a. Courtesy of Professor Paul Mayne, Georgia Institute of Technology.)

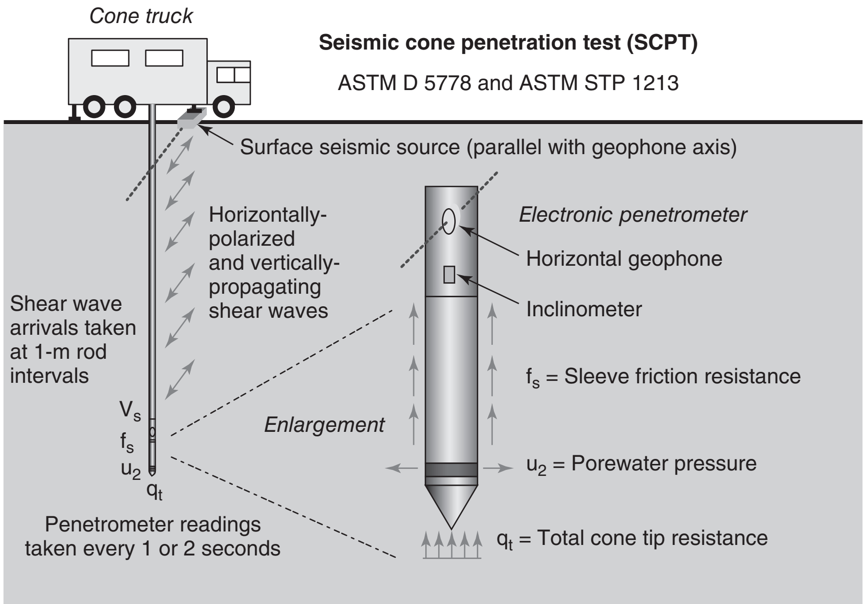


Figure 7.9 Seismic cone penetrometer test. (From Mayne 2007a. Courtesy of Professor Paul Mayne, Georgia Institute of Technology.)

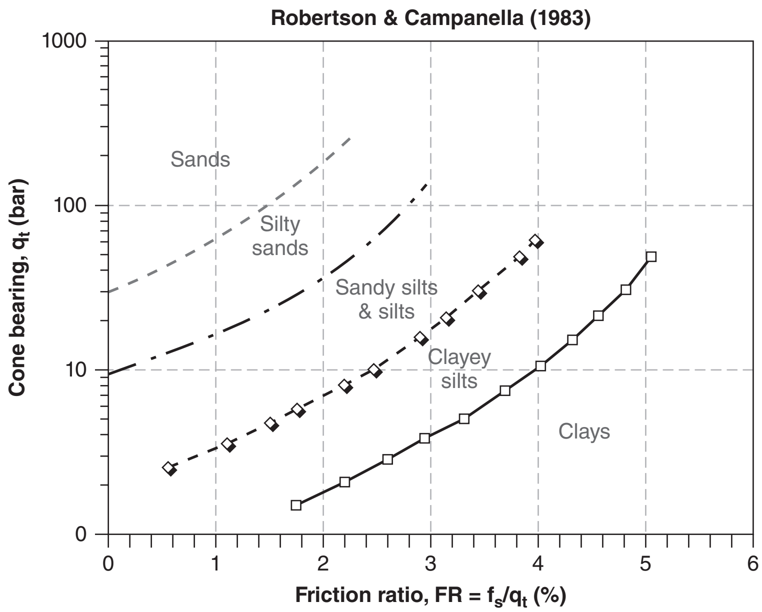


Figure 7.10 Robertson & Campanella (1983) soil classification using CPT results. (From Mayne 2007a. Courtesy of Professor Paul Mayne, Georgia Institute of Technology, USA.)

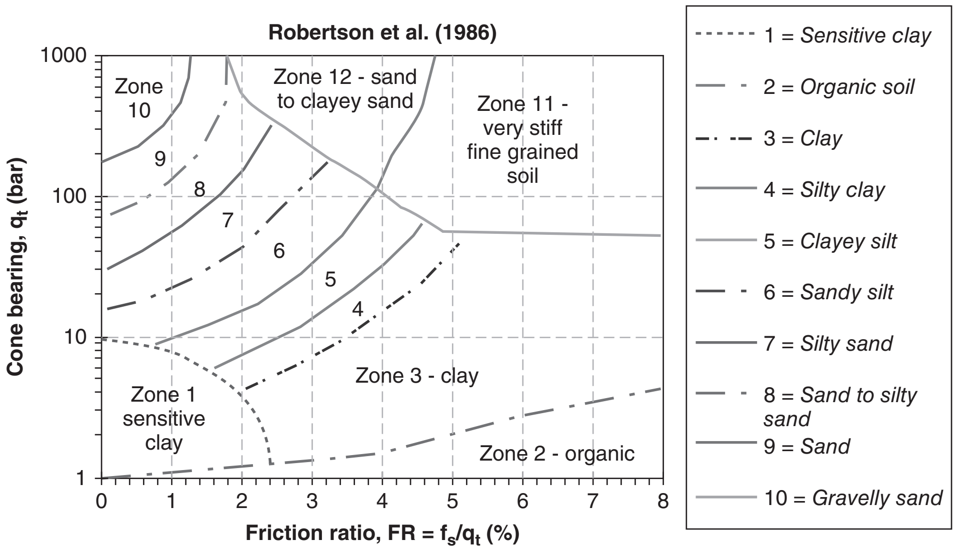


Figure 7.11 Robertson et al. (1986) soil classification using CPT results. (From Mayne 2007a. Courtesy of Professor Paul Mayne, Georgia Institute of Technology, USA.)

tip resistance (FR = f_s/q_t). Several classification schemes have been proposed; Figure 7.10 and 7.11 show two of them. The reason why it is possible to estimate the soil classification from the friction ratio is that the sleeve friction value does not change significantly between a sand and a clay, whereas the tip resistance changes dramatically. Maximum values of sleeve friction might be about 200 kPa for both sand and clay, whereas the maximum tip resistance may be 2000 kPa in a hard clay and 20,000 kPa in a dense sand. The friction ratio would be 10% for the clay and 1% for the sand.

The CPT parameters are used extensively in geotechnical engineering worldwide. Applications include obtaining soil properties such as shear strength parameters and modulus values, ultimate bearing pressure and settlement of shallow and deep foundations, and liquefaction potential. The advantages of the CPT include that it gives a rapid and continuous profile of soil strength; that it is much less operator dependent than other in situ tests; that it is relatively economical; that it does not create cuttings; and that it has a wide range of applications. For example, it is one of the best ways to obtain ultimate vertical pile capacity. One drawback is that the penetration depth is limited in stronger soils.

7.3 PRESSUREMETER TEST

There are three types of pressuremeters: the preboring pressuremeter, the self-boring pressuremeter, and the push-in or cone pressuremeter. In the preboring *pressuremeter test* (PMT), a borehole is drilled first, the drilling tool is removed, and the PMT probe is inserted in the open hole. For the self-boring PMT, the probe is equipped with its own drilling equipment and bores itself into the soil to avoid decompression of the soil due to preboring. For the push-in PMT, the probe is pushed into the soil and full displacement takes place during the insertion, as in the cone penetration test. This section discusses the preboring PMT, which is the most common of the three.

The pressuremeter test was developed in France in the late 1950s, and can be credited to Louis Menard, who conceived it as part of his university graduation project in 1957. The PMT (Figure 7.12; Briaud 1992; ASTM D4719) consists of boring a hole of a given diameter (e.g., 75 mm) down to the selected testing depth, withdrawing the drilling tool, lowering a cylindrical probe to the testing depth, and inflating the cylinder while recording the pressure necessary to do so and the corresponding increase in radius. The test result (Figure 7.13) is an in situ stress-strain curve that gives a number of useful soil parameters: the modulus E_o, called the first load modulus; the pressure p_{oh}, found at the beginning of the curve where the horizontal soil pressure is being reestablished; a yield pressure p_y; and a soil strength called the limit pressure p_L. Often an unload-reload loop is performed near p_y and a reload modulus E_r is determined. Typical profiles resulting from a PMT program are shown in Figure 7.14.

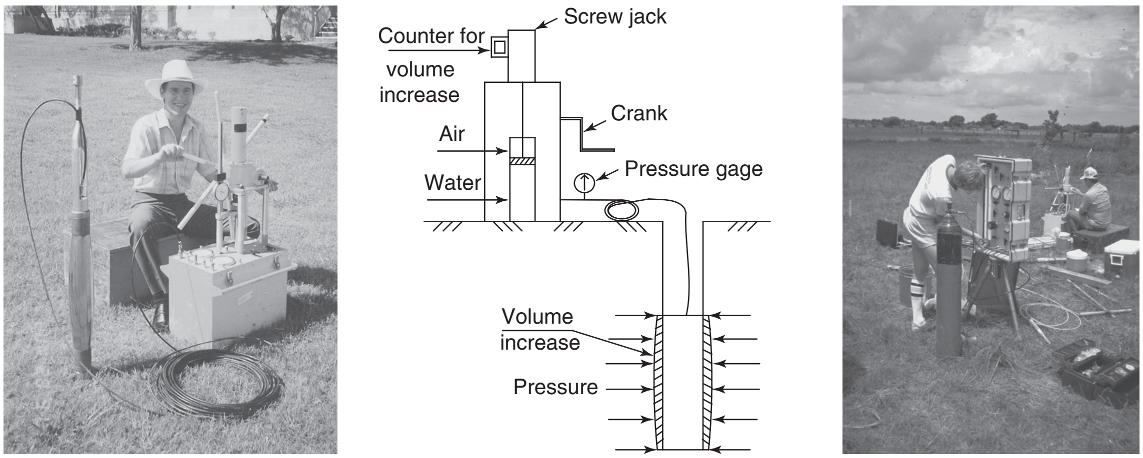


Figure 7.12 TEXAM and Menard pressuremeters.

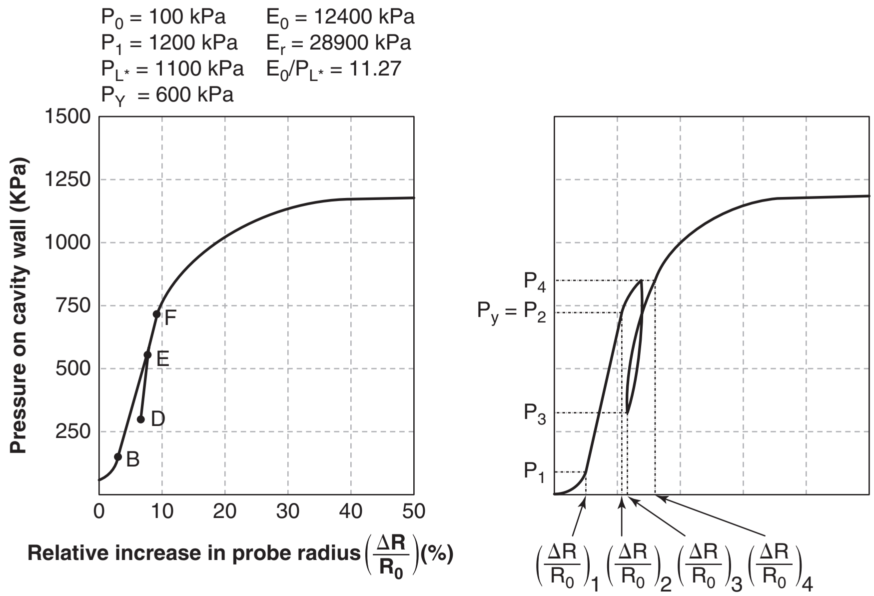


Figure 7.13 Pressuremeter test result.

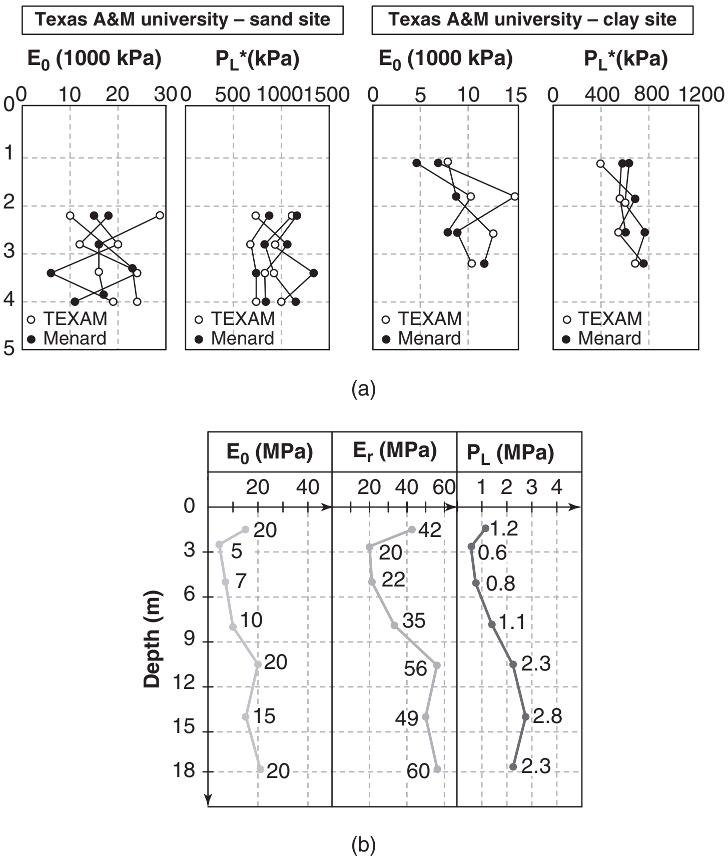


Figure 7.14 Typical PMT profile.

The most important part of a PMT is the preparation of the borehole in which to place the PMT probe. The disturbance of the walls of the borehole should be kept to a minimum and the diameter of the borehole should be only slightly larger than the PMT probe. If D_1, D_2, and D_3 refer to the diameter of the drilling tool, of the deflated probe, and of the borehole before inflation of the probe, respectively, then the following is recommended:

$$D_2 < D_1 < 1.03D_2 \tag{7.7}$$

$$1.03D_2 < D_3 < 1.20D_2 \tag{7.8}$$

The most commonly recommended method for preparing the borehole is the wet rotary method. In this case the rotation of the drill bit should be slow (about 60 rpm) and the circulation of the drilling mud should also be slow. The borehole should be advanced only as deep as necessary to perform one pressuremeter test at a time. The bottom of the borehole should be at least 1 m deeper than the PMT location, to allow any cuttings not transported up to the surface to settle at the bottom of the hole. The borehole should be prepared in one downward passage of the bit, followed by immediate retrieval of the bit; no multiple passages should be allowed, as they lead to an enlarged borehole. The borehole should be drilled to perform one PMT at a time. Other methods can also be used, as shown in Table 7.2.

The probe is calibrated to determine the amount of pressure p_c required to inflate the probe in the air. It is also calibrated to determine the amount of volume v_c necessary to inflate the probe in a tight-fitting thick steel tube. In the field and once the probe is in the borehole, the PMT is run in increments of either pressure or volume. Increases in volume have the advantage that they do not require a guess at the limit pressure. The test lasts about 10 minutes. Data reduction consists of converting the raw data into the actual pressure exerted against the wall of the borehole and the actual relative increase in borehole radius (Briaud 1992).

The modulus E_o and E_r are obtained from the portion of the curve between B and C, and D and E on Figure 7.13, respectively, by using linear elasticity. The equations to obtain E_o and E_r are:

$$E_0 = (1+\upsilon)(p_2 - p_1)\frac{\left(1+\left(\frac{\Delta R}{R_0}\right)_2\right)^2 + \left(1+\left(\frac{\Delta R}{R_0}\right)_1\right)^2}{\left(1+\left(\frac{\Delta R}{R_0}\right)_2\right)^2 - \left(1+\left(\frac{\Delta R}{R_0}\right)_1\right)^2} \tag{7.9}$$

$$E_r = (1+\upsilon)(p_4 - p_3)\frac{\left(1+\left(\frac{\Delta R}{R_0}\right)_4\right)^2 + \left(1+\left(\frac{\Delta R}{R_0}\right)_3\right)^2}{\left(1+\left(\frac{\Delta R}{R_0}\right)_4\right)^2 - \left(1+\left(\frac{\Delta R}{R_0}\right)_3\right)^2} \tag{7.10}$$

Table 7.2 Guidelines for PMT Borehole Preparation

Soil	Type	Rotary Drilling with Bottom Discharge of Prepared Mud	Pushed Thin Wall Sampler	Pilot Hole Drilling and Subsequent Sampler Pushing	Pilot Hole Drilling and Simul-taneous Shaving	Continuous Flight Auger	Hand Auger in the Dry	Hand Auger with Bottom Discharge of Prepared Mud	Driven or Vibro-Driven Sampler	Core Barrel Drilling	Rotary Percussion	Driven or Vibro-Driven or Pushed Slotted Tube
Clayey soils	Soft	2^B	2^B	2	2	NR	NR	1	NR	NR	NR	NR
	Firm to stiff	1^B	1	2	2	1^B	1	1	NR	NR	NR	NR
	Stiff to hard	1	2	1	1	1^B	NA	NA	NA	1^B	2^B	NR
Silty soils	Above GWL^C	1^B	2^B	2	2^B	1	1	2	2	NR	NR	NR
	Under GWL^C	1^B	NR	NR	2^B	NR	NR	1	NR	NR	NR	NR
Sandy soils	Loose and above GWL^C	1^B	NR	NR	2	2	2	1	2	NA	NR	NR
	Loose and below GWL^C	1^B	NR	NR	2	NR	NR	1	NR	NA	NR	NR
	Medium to dense	1^B	NR	NR	2	1	1	1	2	NR	2^B	NR
Sandy gravels or Gravelly sands below GWL	Loose	2	NA	NA	NA	NA	NA	NA	NR	NA	2	2
	Dense	NR	NA	NA	NA	NR	NA	NA	NR	NA	2	1^D
Weathered rock		1	NA	2^B	NA	1	NA	NA	1	2	2	NR

1 is first choice, 2 is second choice, NR is not recommended, and NA is nonapplicable.
B: Method applicable only under certain conditions.
C: GWL is groundwater level.
D: Pilot hole drilling required beforehand.
(After ASTM D4719.)

All parameter definitions are found in Figure 7.13. Note that the reload modulus E_r depends significantly on the amplitude of the unload-reload loop; therefore, unlike E_o, E_r is not unique. The parameter p_y is obtained by inspection as the point where the curve first departs from linearity. The limit pressure p_L is obtained by visual extrapolation of the data to a large value of $\Delta R/R_o$ equal to 0.40 or 40%. The pressure p_{oh} is found at the beginning of the curve at the point of maximum curvature during the reestablishment of the horizontal pressure that existed before placement of the PMT probe. The difference $p_L - p_{oh}$ is called the net limit pressure p_L*. Expected values of these PMT parameters are shown in Table 7.3; correlations to other soil properties are shown in Table 7.4 for sands and gravels and in Table 7.5 for silts and clays. The correlations in Tables 7.4 and 7.5 exhibit very large scatter and should be used for crude estimates only.

The applications of the PMT include the design of deep foundations under horizontal loads, the design of shallow foundations, the design of deep foundations under vertical loads, and the development of a modulus profile and the determination of other soil properties. The PMT is not very useful for slope stability and retaining structures. The advantages of the PMT are that it can be performed in most soils and rocks; that it stresses a larger soil mass than other tests; that it gives a complete stress-strain curve of the soil in situ, including cyclic loading and long-term loading; that it is relatively inexpensive; and that the quality of the test can be judged by the shape of the curve obtained. One drawback of the PMT is that the quality of the borehole influences the PMT parameters, in particular the first load modulus E_o.

7.4 DILATOMETER TEST

The dilatometer test (DMT) was developed in Italy in the mid-1970s and can be credited to Silvano Marchetti. The DMT (Marchetti 1975; Briaud and Miran 1992b; ASTM D6635) consists of pushing a flat blade located at the end of a series of rods (Figure 7.15) into a soil to a desired depth. The blade is 230 mm long, 95 mm wide, and 15 mm thick. Once the testing

Table 7.3 Expected Values of E_o and P_L in Soils

Clay					
Soil Strength	Soft	Medium	Stiff	Very Stiff	Hard
p^*_L (kPa)	0–200	200–400	400–800	800–1600	> 1600
E_0 (kPa)	0–2500	2500–5000	5000–12,000	12,000–25,000	> 2500
Sand					
Soil Strength	Loose	Compact	Dense	Very Dense	
p^*_L (kPa)	0–500	500–1500	1500–2500	> 2500	
E_0 (kPa)	0–3500	3500–12,000	12,000–22,500	>22,500	

Table 7.4 Correlations for Sand

Column A = number in table x row B

B A	E_0 (kPa)	E_R (kPa)	p^*_L (kPa)	q_c (kPa)	f_s (kPa)	N (bl/30 cm)
E_0 (kPa)	1	0.125	8	1.15	57.5	383
E_R (kPa)	8	1	64	6.25	312.5	2174
p^*_L (kPa)	0.125	0.0156	1	0.11	5.5	47.9
q_c (kPa)	0.87	0.16	9	1	50	479
f_s (kPa)	0.0174	0.0032	0.182	0.02	1	9.58
N (bl/30 cm)	0.0026	0.00046	0.021	0.0021	0.104	1

Table 7.5 Correlations for Clay

Column A = number in table x row B

B A	E_0 (kPa)	E_R (kPa)	p^*_L (kPa)	q_c (kPa)	f_s (kPa)	s_u (kPa)	N (bl/30 cm)
E_0 (kPa)	1	0.278	14	2.5	56	100	667
E_R (kPa)	3.6	1	50	13	260	300	2000
p^*_L (kPa)	0.071	0.02	1	0.2	4	7.5	50
q_c (kPa)	0.40	0.077	5	1	20	27	180
f_s (kPa)	0.079	0.0038	0.25	0.05	1	1.6	10.7
s_u (kPa)	0.010	0.0033	0.133	0.037	0.625	1	6.7
N (bl/30 cm)	0.0015	0.0005	0.02	0.0056	0.091	0.14	1

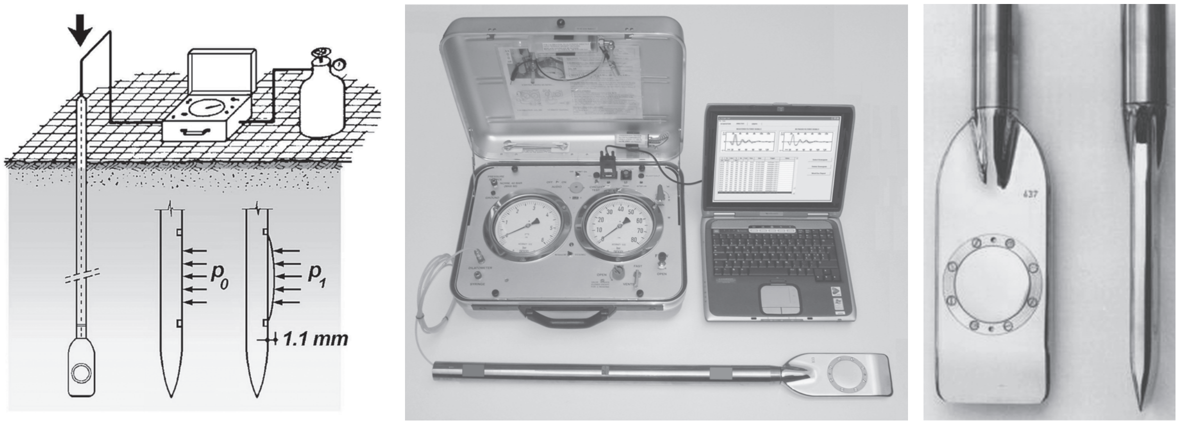


Figure 7.15 Dilatometer test and equipment. (Courtesy of Dr. Sylvano Marchetti, www.marchetti-dmt.it)

depth is reached, the operator uses gas pressure to expand horizontally into the soil a circular membrane located on one side of the blade. The membrane is 60 mm in diameter and expands 1.1 mm into the soil. Two pressures are recorded: p_o and p_1 : p_o is the pressure on the blade before expansion, and p_1 is the pressure required to produce the 1.1 mm expansion into the soil. A number of soil parameters are obtained from the DMT by using the formulas and correlations shown in Table 7.6.

The applications of the DMT include the design of foundations, the determination of soil properties, and soil classification (Figure 7.16). The advantages of the DMT include that it is fast, economical, easy to perform, and reproducible, giving a wealth of soil properties through correlations. A drawback is that it cannot be used in soils that are difficult to penetrate by pushing. Sample profiles are presented in Figure 7.17.

7.5 VANE SHEAR TEST

The vane shear test (VST) can be traced back to 1919 when it was first used in Sweden, but it is unclear if it can be credited to one person (Richards 1988). The VST (Figure 7.18) is used to determine the undrained shear strength of fine-grained soils (clays and silts). It can be performed either in the field with a field vane (ASTM D2573; Figure 7.19) or on the sample with a mini vane or a hand vane (ASTM D4648, Figure 7.20). The vane is made of two perpendicular blades, each having a 2-to-1 height-to-width ratio. The width of the field vanes varies from 38 to 92 mm; the larger vanes are used in softer soils. The width of the lab vanes varies from 10 to 20 mm. The VST consists of pushing a vane at the end of a rod into the soil until the desired depth is reached. Once the testing depth is reached, the vane is rotated at a slow rate (less than 1 degree per minute) while measuring the torque developed and the rotation angle (Figure 7.21). The peak value of the torque is recorded as T_{max}. Then the blade is rotated at least 10 times rapidly and a new maximum torque value, T_{res}, is measured.

The VST is used in saturated fine-grained soils to obtain the undrained shear strength s_u. The reason is that these soils have a low permeability and do not allow appreciable drainage during a test that typically lasts less than 10 minutes.

Table 7.6 Soil Parameters from Dilatometer Test

Symbol	Description	Basic Reduction Formulae	
p_0	Corrected first reading	$p_0 = 1.05\ (A - Zm + \Delta A) - 0.05\ (B - Zm - \Delta B)$	Zm = Gage reading when vented to atmosphere. However, if ΔA and ΔB are measured with the same gage used for current readings A & B, set Zm = 0 (Zm is compensated)
p_1	Corrected second reading	$p_1 = B - Zm - \Delta B$	
I_D	Material index	$I_D = (p_1 - p_0)/(p_0 - u_0)$	u_0 = pre-insertion pore pressure
K_D	Horizontal stress index	$K_D = (p_0 - u_0)/\sigma'_{V0}$	σ'_{V0} = pre-insertion overburden stress
E_D	Dilatometer modulus	$E_D = 34.7\ (p_1 - p_0)$	E_D is *not* a Young's modulus E. E_D should be used only *after* combining it with K_D (stress history). First obtain $M_{DMT} = R_M E_D$, then (e.g.) $E''0.8\ M_{DMT}$
K_0	Coefficient of Earth pressure in situ	$K_{0,DMT} = (K_D/1.5)^{0.47} - 0.6$	for $I_D < 1.2$
OCR	Overconsolidation ratio	$OCR_{DMT} = (0.5\ K_D)^{1.56}$	for $I_D < 1.2$
c_u	Undrained shear strength	$C_{u,DMT} = 0.22\,\sigma'_{V0}(0.5\ K_D)^{1.25}$	for $I_D < 1.2$
φ	Friction angle	$\varphi_{safe,DMT} = 28 + 14.6\ \log\ Kd - 2.1\ \log^2 Kd$	for $I_D > 1.8$
c_h	Coefficient of consolidation	$C_{h,DMTA} \approx 7cm^2/T_{flex}$	T_{flex} from A−log t DMTA−decay curve
k_h	Coefficient of permeability	$k_h = C_h \gamma_w / M_h (M_h \approx K_0 M_{DMT})$	
γ	Unit weight and description	(see chart)	
M	Vertical drained constrained modulus	$M_{DMT} = R_M E_D$ If $(I_D \leq 0.6)\ R_M = 0.14 + 2.36\ \log\ Kd$ If $(I_D \geq 3)\ R_M = 0.5 + 2\ \log\ Kd$ If $(0.6 < I_D < 3)\ R_M = R_{M,0} + (2.5 - R_{M,0})\ \log\ Kd$ where $R_{M,0} = 0.14 + 0.15(I_D - 0.6)$ If $Kd > 10\ R_M, = 0.32 + 2.18\ \log\ Kd$ If $R_M < 0.85$, set $R_M = 0.85$	
$\mathbf{U}_0$	Equilibrium pore pressure	$U_0 = p_2 \approx C\text{–}Zm + \Delta A$	In freely draining soils

(Courtesy of Dr. Sylvano Marchetti, www.marchetti-dmt.it)

Therefore, in these saturated fine-grained soils, it is reasonable to assume that the shearing process is undrained and that the undrained shear strength s_u is the parameter being measured. For a rectangular vane, the following equation gives s_u from T_{max}:

$$T_{max} = \pi s_u D^2 \left(\frac{H}{2} + \frac{D}{6}\right) \tag{7.11}$$

where D is the diameter of the vane and H is the height of the vane. Proof of this equation is shown in the solution to problem 7.4. The residual undrained shear strength s_{ur} is obtained from the same formula using T_{res}:

$$T_{res} = \pi s_{ur} D^2 \left(\frac{H}{2} + \frac{D}{6}\right) \tag{7.12}$$

The VST can be used in coarse-grained soils, but no useful result can be obtained. These soils drain fast enough that one would not be measuring the undrained shear strength, but instead the drained or partially drained shear strength.

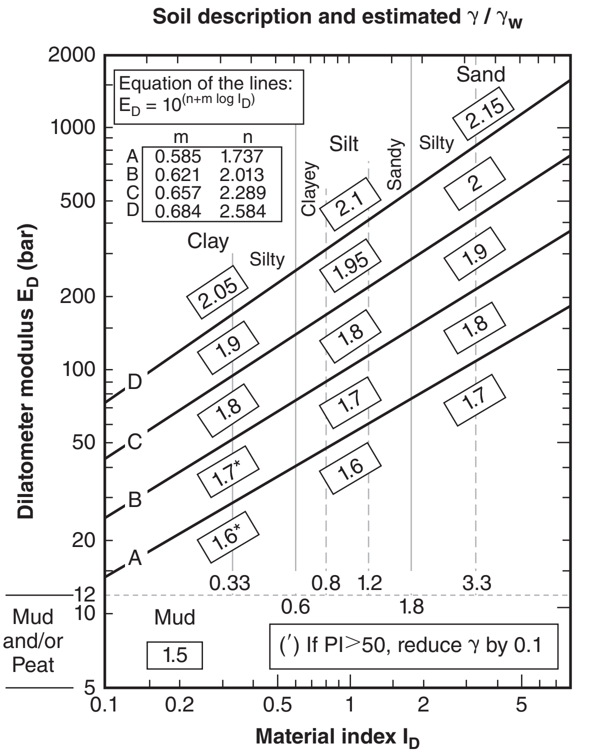


Figure 7.16 Soil classification using the DMT. (Courtesy of Dr. Sylvano Marchetti, www.marchetti-dmt.it)

Back-calculating the shear strength parameters from this test would require knowledge of the normal effective stress on the plane of failure in addition to T_{max}. This is not measured during the VST. The advantages of the VST include that it is fast, simple, economical, and useful for obtaining the undrained shear strength of fine-grained soils. A drawback is that it is limited to fine-grained soils where other methods are commonly used to obtain s_u. One exception is in offshore applications, where obtaining samples is very expensive and sample decompression can alter the true undrained strength of the soil in situ; in this case the VST is extremely useful.

7.6 BOREHOLE SHEAR TEST

The borehole shear test (BST) was developed in the USA in the 1960s and is credited to Richard Handy (Handy 1975, 1986). The BST (Figures 7.22 and 7.23) consists of drilling a borehole, removing the drilling tool, and inserting the borehole shear tester down to the testing depth. The device is made of two diametrically opposed grooved plates, which, once at the testing depth, are pushed horizontally against the wall of the borehole under a chosen total stress σ_h. After a proper time for dissipation of the pore pressures generated by the application of σ_h, the device is pulled upward to shear the soil along the side of the borehole. The force applied is measured as a function of time as it increases, and the peak force generated divided by the plates area gives the shear strength of the soil τ_f. If the shearing part of the test is performed slowly enough to ensure that no excess pore pressures arise, and if the soil has no effective stress cohesion intercept ($c = 0$), the ratio τ_f/σ_h is equal to $\tan\varphi'$ and φ' can be measured with the BST. If the shearing part of the test is performed slowly enough to ensure that no excess pore pressures arise, and if the soil has an effective stress cohesion intercept ($c' > 0$), a stage test can be performed where a second test at a higher value of σ_h follows the first one. The two tests give enough information to back-calculate c' and φ' for the soil (Figure 7.24). If, however, the test is performed rapidly, and does not allow any drainage to take place in the soil, an undrained shear strength s_u of the soil is obtained.

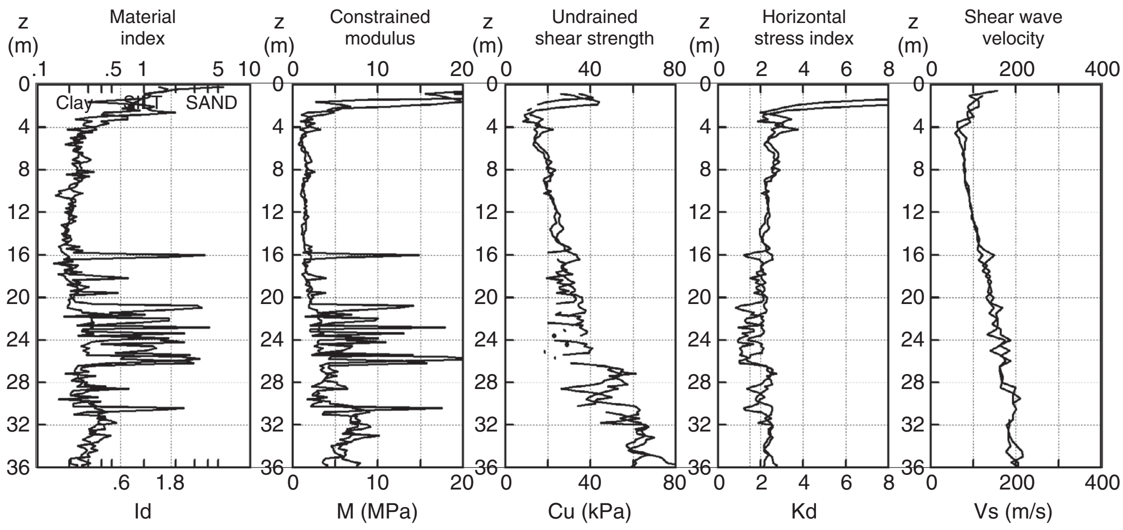


Figure 7.17 Example of dilatometer test results. (Courtesy of Dr. Sylvano Marchetti, www.marchetti-dmt.it)

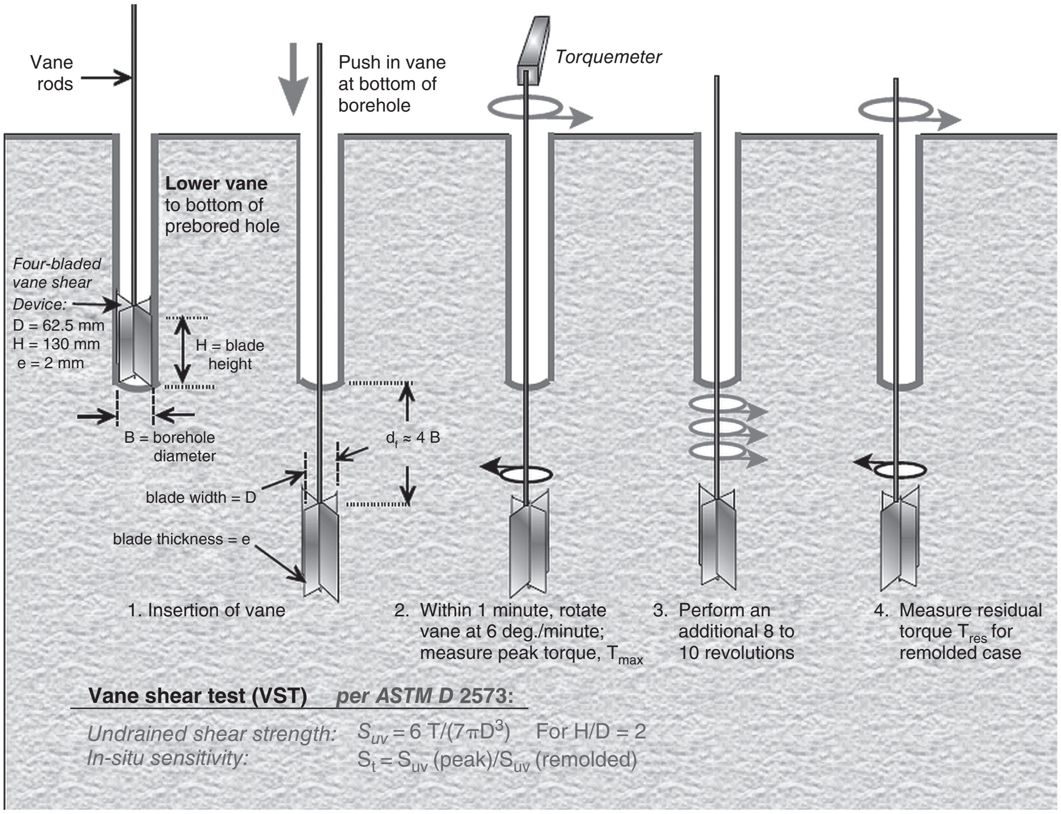


Figure 7.18 The vane shear test. (From Mayne et al. 2002. Courtesy of Professor Paul Mayne, Georgia Institute of Technology)

Figure 7.19 Field vane shear test. (Courtesy of Dr. Dimitrios P. Zekkos.)

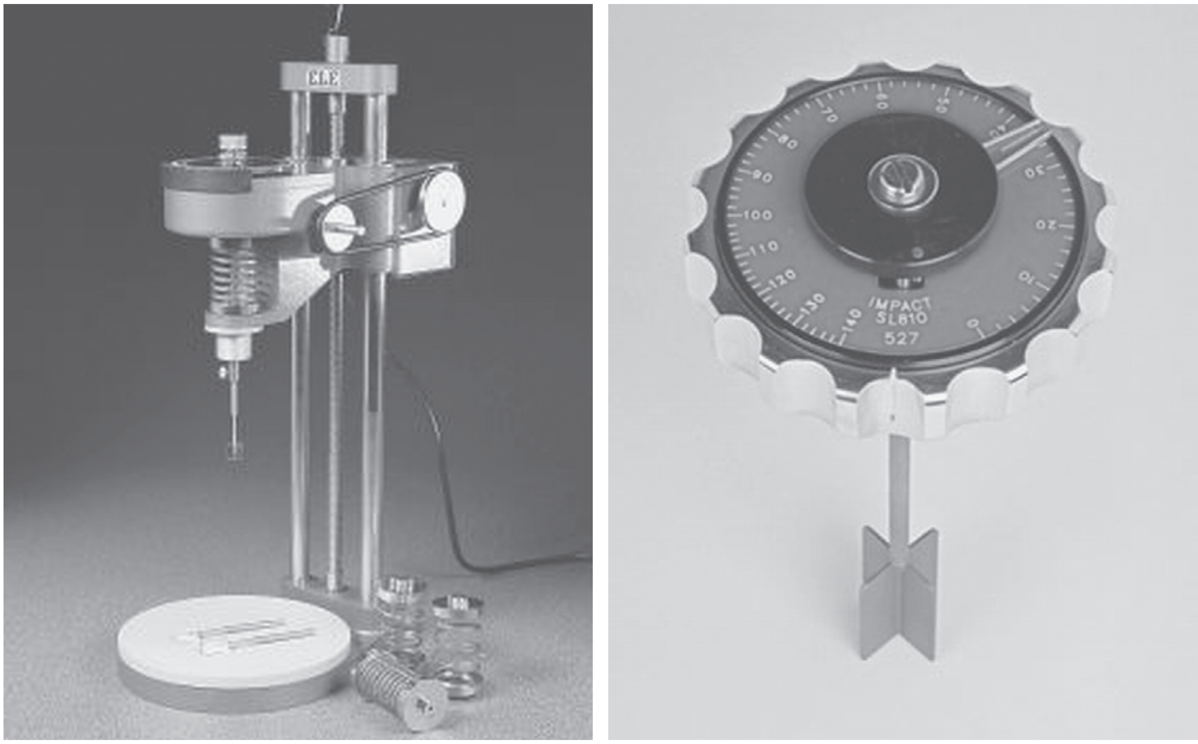


Figure 7.20 Laboratory vane shear test. (a: Courtesy of ELE International, b: Courtesy of Impact Test Equipment Ltd)

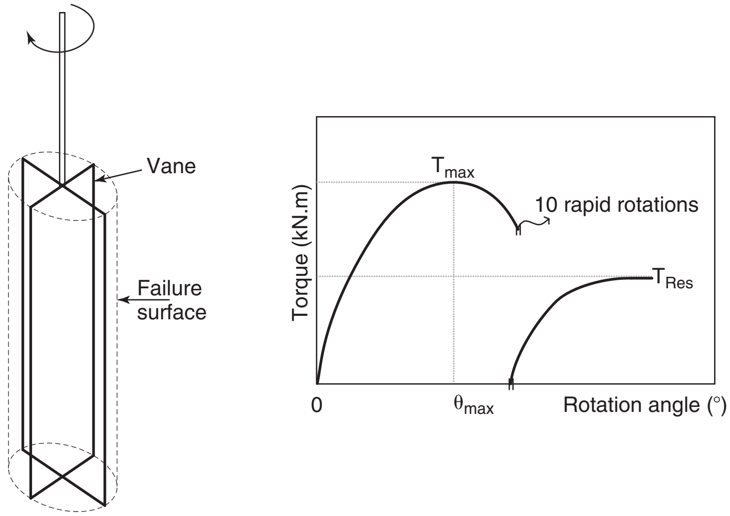


Figure 7.21 Vane shear test results.

The advantages of the BST are that it is simple, economical, and one of the best tools—if not the only tool—to obtain the friction angle of sands by direct measurements in the field. One drawback is that it is difficult to know exactly what pore pressures are generated. A pore pressure sensor on the plates helps in that respect. The phicometer developed by Philiponat (Philiponat, 1986, Philionat and Zerhouni, 1993) is a similar tool.

7.7 PLATE LOAD TEST

The plate load test or PLT (Figure 7.25; ASTM D1196 and D1195) is one of the simplest and oldest in situ tests. It consists of placing a circular plate with a diameter D on a prepared soil surface and loading the plate in steps until the desired pressure p is reached. The plate diameter is usually on the order of 0.3 m. Sometimes one or more unload-reload loops are performed during the test. All load

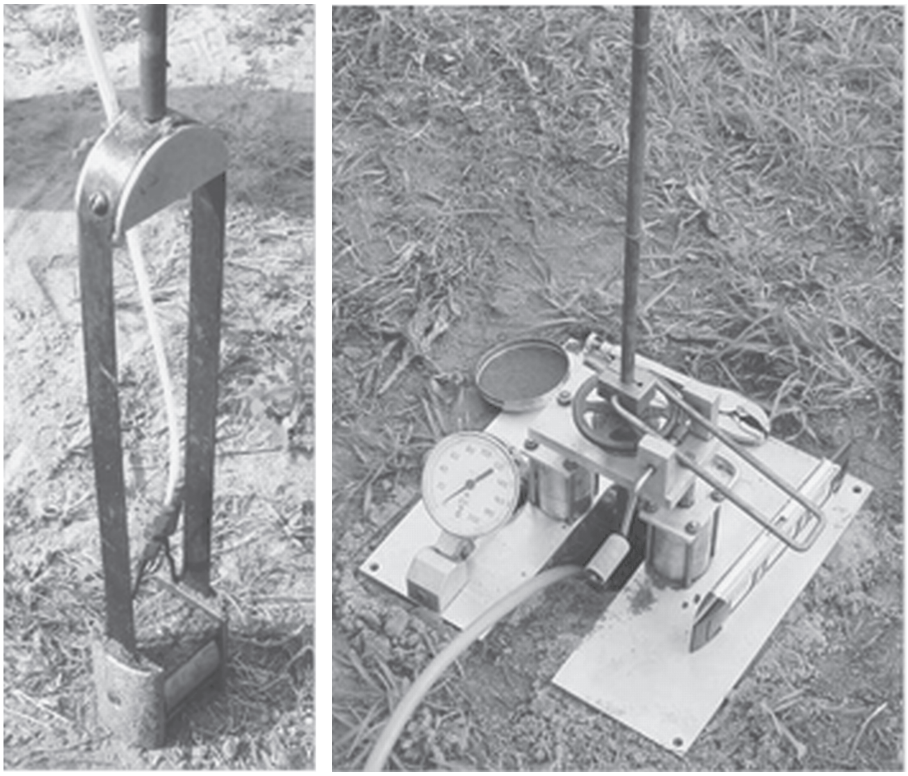


Figure 7.22 Borehole shear test device. (Courtesy of In-Situ Soil Testing, L.C.)

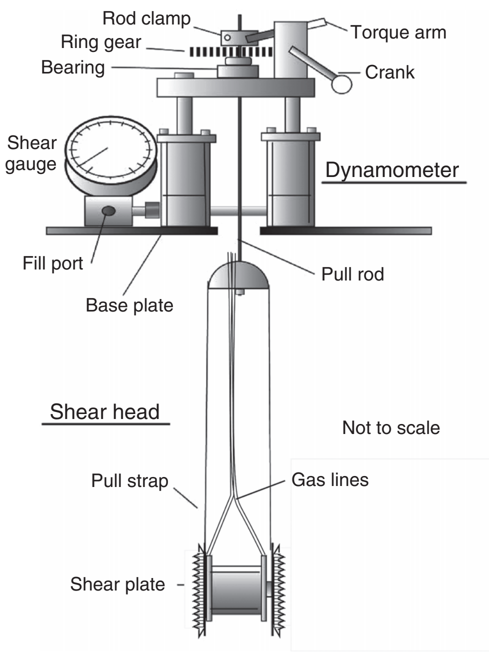


Figure 7.23 Borehole shear test device. (Courtesy of Professor Richard L. Handy, Handy Geotechnical Instruments, Inc.)

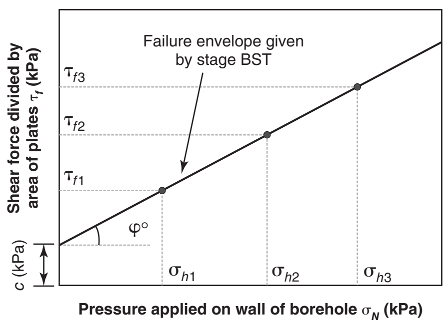


Figure 7.24 Results of a borehole shear test.

steps are held for the same period of time, during which readings of the plate settlement s are made as a function of time t. Loading the plate to soil failure is often desirable but not always possible. The load is measured with a load cell and the settlement is measured by using dial gages or electronic displacement devices (e.g., a linear variable differential transformer [LVDT]) attached to a settlement beam. It is critical that the supports of the settlement beam be far enough from the plate influence zone. Five plate diameters on each side seem appropriate.

The result of the test is a load Q versus displacement s curve (Figure 7.26), which can also be presented in normalized form as the ratio of the average pressure p under the plate over a measure of soil strength SS versus settlement of the plate s over the plate diameter D. The soil strength SS can be the ultimate bearing pressure under the plate p_u, the pressuremeter limit pressure p_L, the cone penetrometer point resistance q_c, the undrained shear strength s_u, the SPT blow

Figure 7.25 Plate load tests. (a: Photo by David Wilkins. Courtesy of Raeburn Drilling and Geotechnical (Northern) Limited; www.raeburndrillingnorthern.com. b: Courtesy of GEMTECH Limited, Fredericton, New Brunswick.)

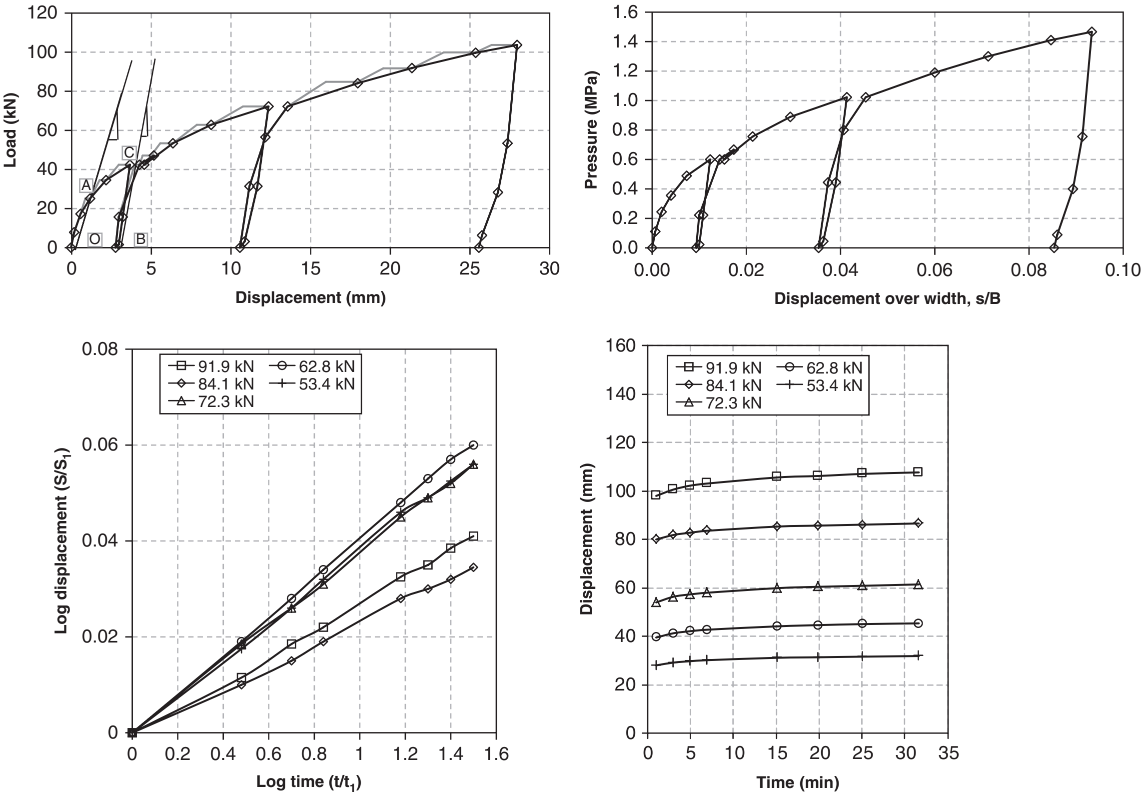


Figure 7.26 Results of load test for 0.3-m-diameter plate on medium dense silty sand.

count N, or another measure of soil strength. The ultimate bearing pressure p_u is often defined as the pressure reached when settlement of the plate is equal to 10% of the plate diameter. The advantage of plotting the results in this fashion (p/SS versus s/D) is that the results of the test become a property of the soil within the zone of influence of the plate and do not depend on the plate size (Briaud 2007). The soil modulus as measured during a plate test is obtained from the initial loading portion E_0 (O to A on Figure 7.26) or from the slope of the reloading part of the unload-reload loop E_r (B to C on Figure 7.26). The equations to be used for E_0 and E_r, if the plate can be assumed to be rigid, are:

$$E_0 = \frac{(1-\nu^2)\pi pD}{4s} \tag{7.13}$$

$$E_r = \frac{(1-\nu^2)\pi \Delta pD}{4\Delta s} \tag{7.14}$$

where E_0 is the initial modulus from a plate load test, ν is Poisson's ratio (to be taken as 0.5 if the plate test is fast enough that no drainage can take place during the test and 0.35 if the test is drained), p is the average pressure under the plate corresponding to the settlement s, D is the diameter of the plate in contact with the soil surface, E_r is the reload modulus from a plate load test, and Δp is the pressure increment during the reload loop corresponding to the settlement increment Δs.

In addition to obtaining the soil modulus, sometimes the modulus of subgrade reaction is calculated from the plate test, as follows:

$$K = \frac{p}{s} \text{ in kN/m}^3 \tag{7.15}$$

Note that K is not a soil parameter, since it depends on the size of the plate:

$$K = \frac{4E_0}{(1-\upsilon^2)\pi D} \tag{7.16}$$

Therefore, the modulus of subgrade reaction K measured with a plate of a given diameter D cannot be used for plates or footings that have diameters significantly different from D.

It is also useful to plot the settlement of the plate s versus the time t for each load step on a log-log plot (Figure 7.26). The plot of log s versus log t is remarkably linear in most cases within the working load range. The slope of that line is called the *viscous exponent n* and allows one to predict by extrapolation the displacement at much longer times than the time taken to run the plate test, based on equation 7.17:

$$\frac{s_1}{s_2} = \left(\frac{t_1}{t_2}\right)^n \tag{7.17}$$

where s_1 is the settlement after a time t_1 and s_2 is the settlement after a time t_2 and n is the slope of the log s versus log t curve for the load step corresponding to s_1. Alternatively, the soil modulus E_0 or E_r can be written as:

$$\frac{E_1}{E_2} = \left(\frac{t_1}{t_2}\right)^{-n} \tag{7.18}$$

The advantage of the plate load test is that it is very simple and economical to perform. The drawback is that it only tests a zone of soil near the ground surface (one to two plate diameters deep), although larger depths can be reached by performing the test at the bottom of open pits.

7.8 CALIFORNIA BEARING RATIO TEST

The California bearing ratio test (CBR) is a form of plate test (Figure 7.27). It can be performed in the field or in the lab. In the field (ASTM D4429), it consists of placing a 254 mm diameter plate weighing 44.5 N on the ground surface and loading it until the settlement s is 2.5 mm. The load Q corresponding to a settlement s of 2.5 mm is divided by the plate area to get the pressure p. The California bearing ratio is the ratio between p and the pressure necessary to reach a settlement s of 2.5 mm on a reference soil (crushed California limestone). The pressure necessary to create 2.5 mm of settlement of the plate on the reference soil (crushed California limestone) has been measured to be 6900 kPa. So, the reference pressure is 6900 kPa and the CBR number is a percentage given by:

Figure 7.27 CBR test in the field. (Courtesy of A F Howland Company.)

$$\text{CBR} = \frac{100 \times p(\text{kPa})}{6900} \tag{7.19}$$

This test is used primarily for pavement design, where the depth of influence of the plate is similar to the depth of influence of a truck tire. If the CBR value is less than 3%, the soil is too soft for road support without modification, values between 3% and 5% are average, and values from 5% to 15% are good. Crushed rock values are around 100%. Several correlations have been developed to link the CBR to soil properties, such as:

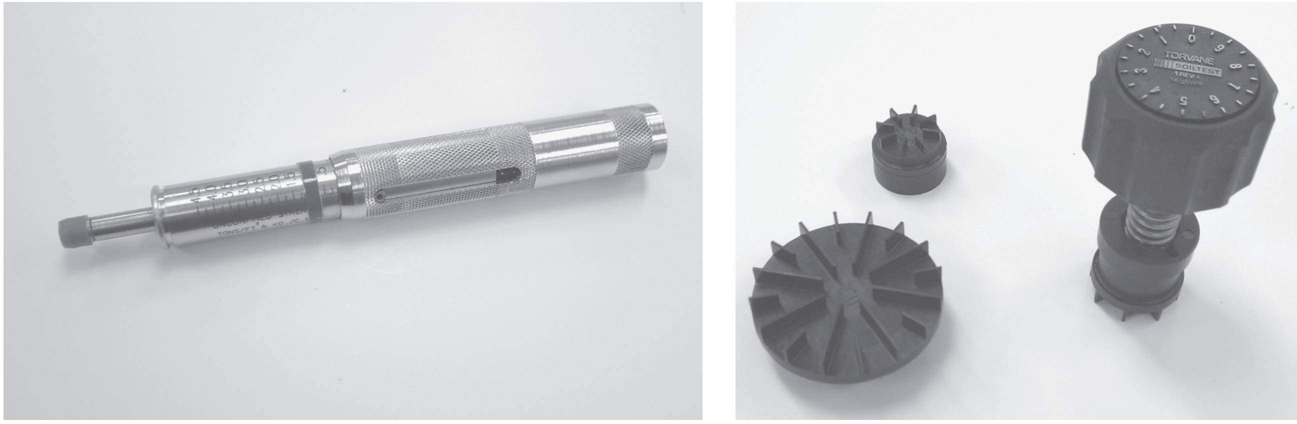
$$M_r \text{ (kPa)} = 10{,}000 \times \text{CBR} \tag{7.20}$$

$$s_u \text{ (kPa)} = 11 \times \text{CBR} \tag{7.21}$$

where M_r is the resilient modulus and s_u is the undrained shear strength.

7.9 POCKET PENETROMETER AND TORVANE TESTS

A number of simple tests can be performed on the sample in the field as soon as it is retrieved from the borehole. They are typically performed on the end of samples taken with a Shelby tube. These tests include the pocket penetrometer, the torvane, and the hand vane tests. The pocket penetrometer



(a) (b)

Figure 7.28 Pocket penetrometer and torvane: (a) Pocket penetrometer (see also this video: www.encyclopedia.com/video/PBo0UDVWhSo-hand-penetrometer-test.aspx). (b) Torvane (see also this video: www.encyclopedia.com/video/9Su3ehhLfwc-torvane-test.aspx)

test (PPT) (Figure 7.28) consists of pushing by hand the end of a spring-loaded cylinder 6.35 mm in diameter until the ultimate bearing pressure is reached. The compression of the spring increases as the force increases and a floating ring on the body of the pocket penetrometer (PP) indicates how much force is exerted. The ultimate pressure is reached when the cylinder penetrates without further increase in the PP reading. The PP number ranges from 0 to 4.5 and has been correlated with the undrained shear strength of clays (s_u (kPa) $\sim$ 30 PP), but the scatter in this correlation is very large—not to mention the fact that the mass of soil tested is extremely small. The advantage of the PPT is that it is a very simple test that gives a quick indication of the soil strength. The drawback is that it tests only a very small zone of soil and thus must not be used in design. The torvane test (TVT) (Figure 7.28) consists of pushing a set of vanes about 6.5 mm into the face of the sample and then rotating the spring-loaded cap until the spring releases because the shear strength of the soil has been reached. A maximum value indicator stays at the maximum reading reached during the rotation and indicates the shear strength of the soil. The hand vane shear test (VST) (section 7.5, Figure 7.20) is also a simple and quick test that can be performed on the end of a Shelby tube sample. These three simple tests are mostly used on silts and clays. Of the three, the hand vane is the most reliable.

7.10 POCKET ERODOMETER TEST

The pocket erodometer test (PET) (Figure 7.29, Briaud, Bernhardt, and Leclair 2011) is to erosion resistance what the pocket penetrometer test is to shear resistance. The pocket erodometer (PE) is a regulated mini-jet-impulse-generating device. The water jet comes out of the nozzle at 8 m/s and is aimed horizontally at the vertical face of the sample. Verification that the velocity is 8 m/s when leaving the nozzle is achieved by aiming the jet from a height H (Figure 7.29), measuring the distance x where the water reaches the floor, and using the following equation:

$$v_{0x} = \frac{x}{\sqrt{\frac{2H}{g}}} \tag{7.22}$$

where v_{0x} is the velocity at the nozzle and g is the acceleration due to gravity. The depth of the hole in the surface of the sample created by 20 impulses of water is recorded. The depth of the hole is entered in the erosion chart (shown in Figure 7.30) to determine the erodibility category of the soil.

Figure 7.29 Pocket erodometer test.

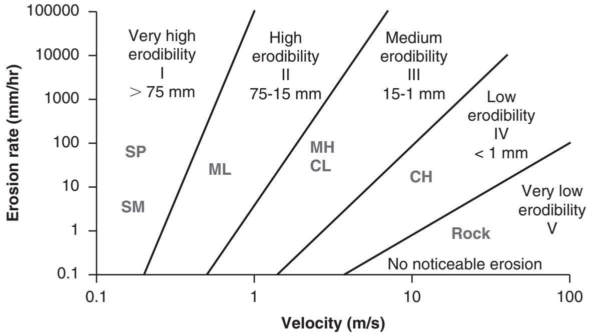


Figure 7.30 Erosion chart for various erosion depths from the PET.

This erosion category allows the engineer to make preliminary decisions in erosion-related work. The advantage of the PET is its simplicity; its drawback is that it tests a very small portion of the soil.

7.11 COMPACTION CONTROL TESTS

Soil compaction is one of many techniques of soil improvement and is discussed in Chapter 20. In short, the soil to be used at the site is tested in the laboratory where compaction tests are performed. The results of these tests are used to establish the target values (dry unit weight, modulus, water content) to be achieved during the compaction process in the field. In the field it becomes necessary to verify that the target value has been reached. These in situ tests include tests to measure the dry unit weight (e.g., sand cone method, rubber balloon method, nuclear density probe), water content (e.g., nuclear density probe, field oven test), and soil modulus (e.g., BCD, falling weight deflectometer).

7.11.1 Sand Cone Test

The sand cone test (SCT; Figure 7.31) consists of digging a hole in the ground, obtaining the weight and the volume of the soil excavated, drying the soil and obtaining the dry weight, and calculating the water content and the dry unit weight. More specifically, a standard steel plate with a 172 mm diameter hole through it is placed on the ground surface. A hole is dug into the ground through the hole in the steel plate to a depth of about 150 mm. The excavated soil is weighed, then dried, then weighed again. This gives the water content of the soil that was in the hole. As soon as the hole is excavated, an inverted funnel in the form of a cone is placed on top of the opening in the base plate and a bottle full of sand of known unit weight is connected to the top of the funnel. (The weight of the bottle full of sand is measured beforehand.) The valve between the bottle and the funnel is then opened and the sand of known unit weight flows out of the bottle until the hole in the ground and the funnel above it are full. The valve is closed, the bottle is disconnected, and the bottle is weighed again. The difference in weight of the bottle before and after filling the hole, divided by the known unit weight of the sand, gives the volume of the hole plus the funnel. Because the volume of the funnel is known, the volume of the hole can be deduced and the dry unit weight is obtained from the dry weight and the volume of the soil in the hole.

7.11.2 Rubber Balloon Test

The rubber balloon test (RBT; Figure 7.32) follows exactly the same procedure as the sand cone method except that the volume of the soil excavated is measured in a different way. The rubber balloon device is a cylinder filled with water up to a level indicated on a graduated scale. At the bottom of

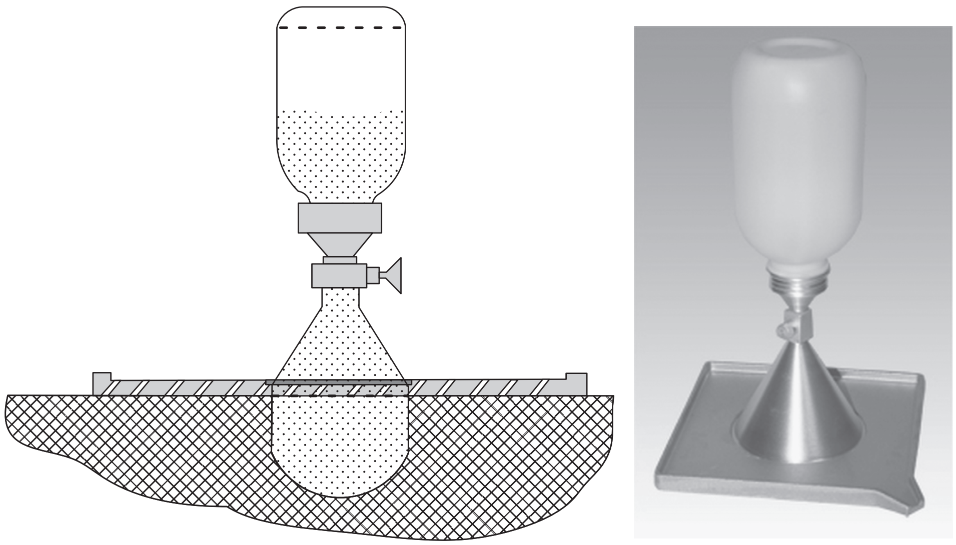


Figure 7.31 Field unit weight and water content by sand cone test. (b: Courtesy of Durham Geo Slope Indicator.)

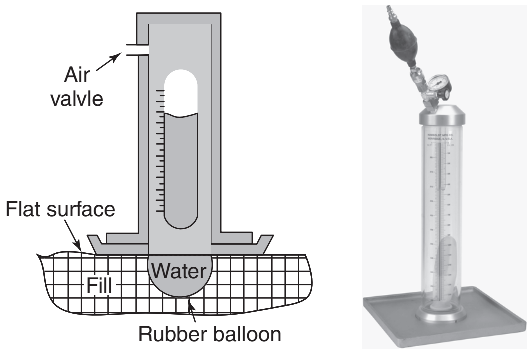


Figure 7.32 Field unit for testing weight and water content by rubber balloon. (b: Courtesy of Humboldt Mfg. Co.)

the cylinder is a rubber balloon that can be expanded into the hole below by pumping water into it. When the balloon fills the hole, the reading on the graduated scale on the cylinder gives the volume of the hole. The data reduction is the same as for the sand cone test.

7.11.3 Nuclear Density/Water Content Test

The nuclear density/water content test is a device to measure indirectly the density and water content of a soil at the soil surface. It consists of sending radiation from a source into the soil and counting the amount of radiation coming back to a detector. In the case of the nuclear density test, a source generating medium-energy gamma rays is used. These gamma rays send photons into the soil (photons are particles of light; see section 8.4.1). These photons go straight to the detector, or bump into the soil particles (Compton scattering) and deflect to arrive at the detector, or do not arrive at the detector. The gamma rays arriving at the detector are counted, and the gamma count is inversely proportional to density. In the case of the water content test, a source generating high-energy neutrons is used. The principle is that when a high-energy neutron hits a much heavier atomic nucleus, it is not slowed down significantly. However, if it hits an atomic nucleus that is about the same weight as the neutron, then the neutron is slowed down significantly. The hydrogen atom has a nucleus that is very comparable in weight to the neutron, and therefore is very good at slowing neutrons down. Because water has a lot of hydrogen, counting the number of slow neutrons coming back to a detector will indicate how much water is in the soil.

The test can be done in direct transmission or in back-scatter mode. In the direct transmission mode, the source rod penetrates into the soil anywhere from 75 mm to 220 mm (Figure 7.33); the detector is on the bottom side of the nuclear gage. This mode is preferred for density measurements. In the back-scatter mode, the nuclear gage sits on the soil surface and the source and detectors are on the bottom side of the gage (Figure 7.33). This is the mode used for water content determination. The nuclear gage is calibrated by the manufacturer initially and after any repair. The calibration consists of placing the gage on a sufficiently large block of material of known density and known water content.

7.11.4 Field Oven Test

The field oven (Figure 7.34) is a very simple instrument which is used to determine the water content of a soil in the field. A small piece of soil is carved from the soil surface; the sample is placed between the two plates of the field oven which looks like a waffle maker. A load cell located below the heating pad gives the weight of the sample. Then the two plates are closed and the oven dries the soil sample. After a

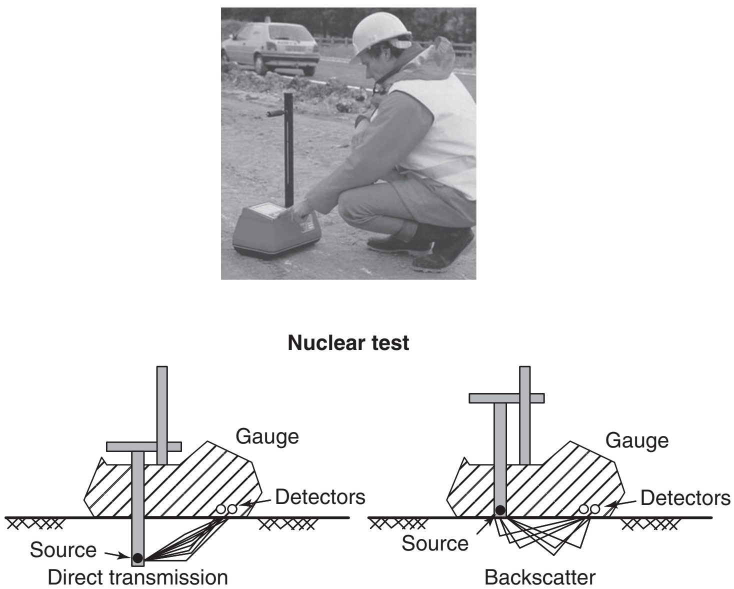


Figure 7.33 Nuclear density probe test for unit weight and water content.

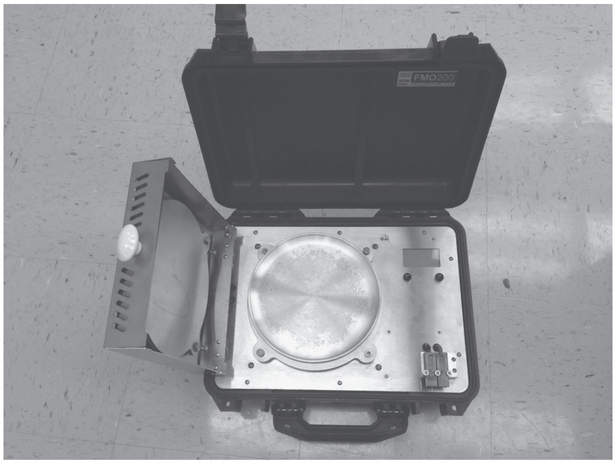


Figure 7.34 Field oven test (FOT) for water content.

few minutes, the soil is dry and the heating plates are opened. The load cell records the dry weight of the sample and the water content is displayed.

7.11.5 Lightweight Deflectometer Test

The lightweight deflectometer (LWD) test (Figure 7.35) (ASTM E2583) consists of dropping a weight guided along a rod from a chosen height onto a plate resting on the ground surface. The typical values for the LWD are a weight of 100 N, a drop height of 0.5 m, and a plate diameter of 0.2 m. A load cell located above the plate measures the force versus time signal and a geophone attached to the plate measures the deflection of the plate during the impact. The soil modulus is back-calculated from the knowledge of the peak force F and the peak deflection Δ. The soil modulus E is calculated using the theory of elasticity:

$$E = f(1 - \nu^2)\frac{4F}{\pi D \Delta} \tag{7.23}$$

where E is the soil modulus measured by the LWD, f is a plate rigidity factor (1 for flexible plates and 0.79 for rigid plates), ν is Poisson's ratio (range from 0.3–0.45, depending on soil type), F is the maximum force on the force versus time plot, D is the plate diameter, and Δ is the maximum displacement on the displacement versus time plot.

For example, referring to the flexible plate LWD test in Figure 7.36, the modulus would be calculated as:

$$E = 1(1 - 0.35^2)\frac{4 \times 7.5}{\pi \times 0.2 \times 0.55 \times 10^{-3}} = 76.3\,\text{MPa} \tag{7.24}$$

7.11.6 BCD Test

A modulus E can also be obtained with a device called the BCD (Figure 7.37). It consists of a 150 mm diameter, 2 mm thick flexible steel plate at the bottom of a rod with handles—a kind of scientific cane. Strain gages are mounted on the back of the plate to record the bending that takes place during the loading test. When the operator leans on the handle, the load on the plate increases and the plate bends. If the soil is soft (low modulus), the plate bends a lot. If the soil is hard (high modulus), the plate does not bend much. The amount of bending is recorded by the strain gages and is correlated to the modulus of the soil below.

The test is called the BCD test or BCDT (Briaud, Li, and Rhee 2006) and is performed as follows. First, the BCD plate is placed on top of the ground surface (Figure 7.37). Then the operator leans on the handles of the BCD and the vertical load increases. When the load goes through 223 N, a load sensor triggers the reading of the strain gages. The device averages the strain gage values, uses the internal calibration equation linking the strains to the modulus, and displays the modulus E. This evaluates the level of compaction achieved at that location.

The modulus obtained with the BCD corresponds to a reload modulus, to a mean stress level averaging about

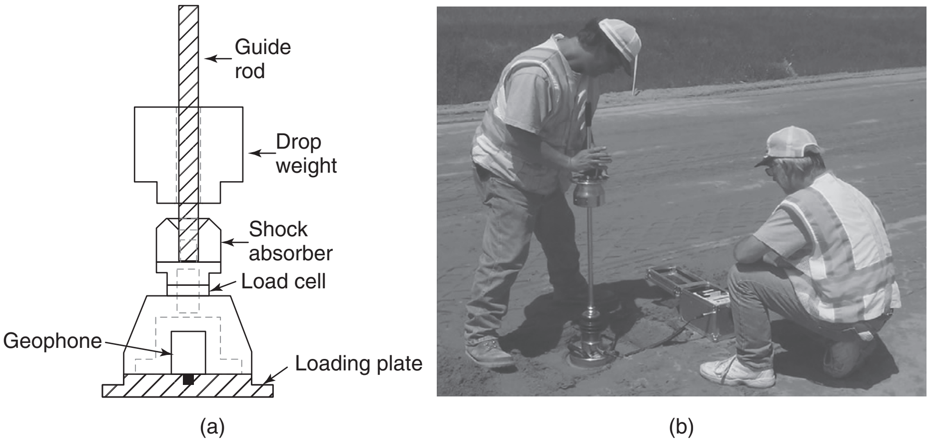


Figure 7.35 Falling weight deflectometer for soil modulus: (a) Principle. (b) Equipment. (b: Courtesy of Minnesota Department of Transportation.)

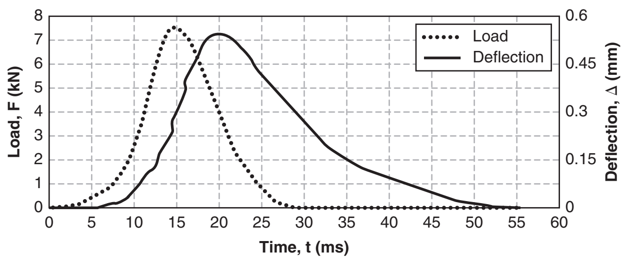


Figure 7.36 Falling weight deflectometer data.

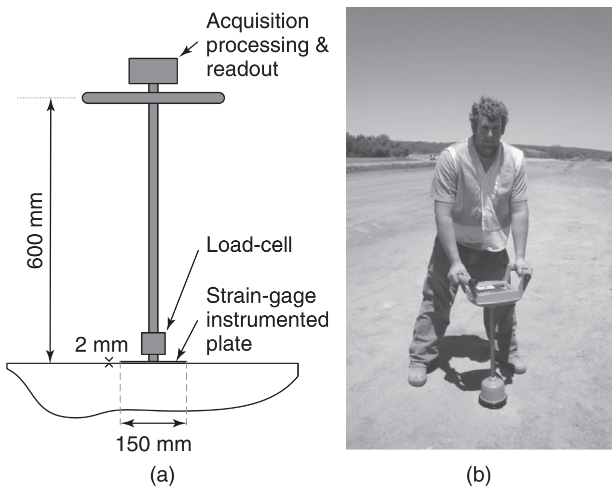


Figure 7.37 BCD test for soil modulus: (a) Principle. (b) Equipment.

50 kPa within the zone of influence, to a strain level averaging 10^{-3} within the zone of influence, and to a time of loading averaging about 2 s. The BCD test can also be performed in the laboratory on top of the compaction mold to obtain the modulus versus water content curve in parallel with the dry density versus water content curve (see chapter 20 section 20.2).

7.12 HYDRAULIC CONDUCTIVITY FIELD TESTS

The purpose of these hydraulic conductivity in situ tests is to measure the hydraulic conductivity k (m/s) of the soil. The soil can be either below the groundwater level (saturated), or above the groundwater level (saturated by capillary action or unsaturated). For saturated soils below the GWL, several tests exist, including the borehole tests (falling head test, rising head test, constant head tests), the pumping test, and the cone penetrometer dissipation test. For soils above the GWL, the tests include the sealed double-ring infiltrometer (SDRI) test and the two-stage borehole permeameter.

7.12.1 Borehole Tests

Borehole tests consist of drilling a borehole, changing the water level in the borehole, and recording the movement of the

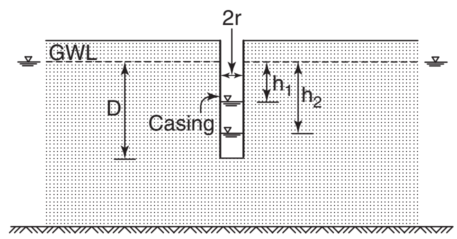


Figure 7.38 Inflow well test in deep uniform soil. (After Hunt 1984.)

water level in the borehole as a function of time. Sometimes the borehole is cased to help in keeping the borehole stable. The data collected are used to back-calculate the hydraulic conductivity k. The equations to calculate k are based on developing the governing differential equation for the problem and then solving it while satisfying the boundary conditions. This is where the problem becomes quite complicated and requires charts or software. The following examples are cases in which the geometry is simple.

When the soil layer is deep and uniform, when the casing goes down to the bottom of the borehole, and when the water is bailed out so that the water level starts far below the groundwater level outside of the casing (Figure 7.38), the hydraulic conductivity k is obtained from the equation:

$$k = \frac{2\pi r \ Ln\dfrac{h_1}{h_2}}{11(t_2 - t_1)} \tag{7.25}$$

where r is the radius of the casing, h_1 and h_2 are the distances from the groundwater level in the soil deposit outside of the casing to the level of the water in the casing, and t_1 and t_2 are the times at which h_1 and h_2 are measured. This equation applies when the depth D as shown in Figure 7.38 is between 0.15 m and 1.5 m.

In the case where the pervious soil layer to be tested is underlain by an impervious layer, where the uncased boring (or screened boring) penetrates through the entire pervious layer all the way to the impervious layer, and where the water level is maintained constant by pumping at a flow rate Q

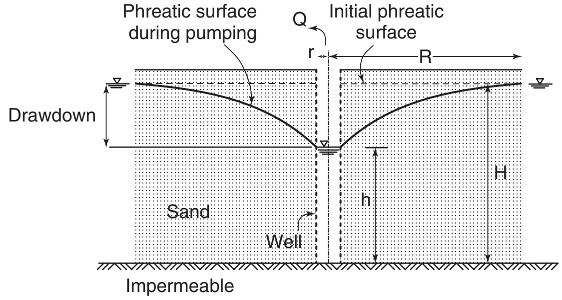


Figure 7.39 Pumping test in sand layer using one boring. (After Hunt 1984.)

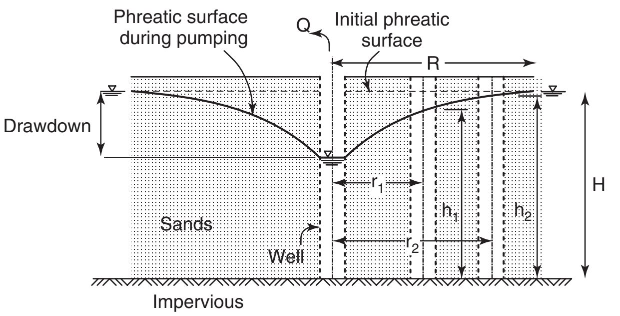


Figure 7.40 Pumping test in sand layer using three borings. (After Hunt 1984.)

(as shown in Figure 7.39), the hydraulic conductivity k is obtained from the equation:

$$k = \frac{Q \ Ln\dfrac{R}{r}}{\pi(H^2 - h^2)} \tag{7.26}$$

where Q is the flow rate pumped out of the well to maintain the water level constant in the well, r is the radius of the borehole, R is the radius of the zone of influence where the water table is depressed, H is the vertical distance between the bottom of the boring (impervious layer) and the groundwater level at or further than R, and h is the vertical distance between the bottom of the boring and the water level in the borehole. Note that for this equation to apply, a steady-state flow must be reached; this may take a time related to the hydraulic conductivity itself. Finding the value of R requires some borings down to the groundwater level away from the test boring.

To improve the precision of this test, observation borings can be drilled at radii r_1 and r_2 from the test boring and the vertical distances h_1 and h_2 between the bottom of the boring (impervious layer) and the water level in the observation borings recorded (Figure 7.40). Then equation 7.26 becomes:

$$k = \frac{Q \ Ln\dfrac{r_2}{r_1}}{\pi(h_2^2 - h_1^2)} \tag{7.27}$$

In the case where the pervious layer to be tested is sandwiched between two impervious layers, where the uncased boring (or screened boring) penetrates through the first two layers and stops at the top of the second impervious layer, and where the water level is maintained constant by pumping at a flow rate Q (as shown in Figure 7.41), the hydraulic conductivity k is obtained from the equation:

$$k = \frac{Q \ Ln\dfrac{R}{r}}{2\pi D(H - h)} \tag{7.28}$$

where Q is the flow rate pumped out of the well to maintain the water level constant in the well, r is the radius of the borehole, R is the radius of the zone of influence where the water table is depressed, H is the vertical distance between the bottom of the boring (top of the second impervious layer) and the groundwater level at or further than R, and h is the vertical distance between the bottom of the boring (top of the second impervious layer) and the water level in the borehole. Note that for this equation to apply, a steady-state flow must be reached; this may take a time related to the hydraulic conductivity itself. Finding the value of R requires some borings down to the ground-water level away from the test boring.

To improve the precision of this test, observation borings can be drilled at radii r_1 and r_2 from the test boring and the vertical distances h_1 and h_2 between the top of the second impervious layer and the water level in the borehole recorded (Figure 7.42). Then equation 7.28 becomes:

$$k = \frac{Q \ Ln\dfrac{r_2}{r_1}}{2\pi D(h_2 - h_1)} \tag{7.29}$$

Solutions for more complicated geometries are found in Mansur and Kaufman (1962) and in Cedergren (1967). The advantages of these tests are that they give a large-scale value of k in the field which includes the mass features of the soil deposit. Some of the drawbacks are the lack of control over problems such as filter cake development around the

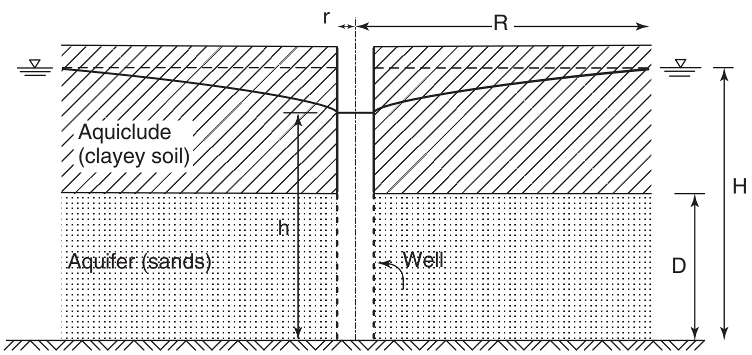


Figure 7.41 Pumping test in confined aquifer. (After Hunt 1984.)

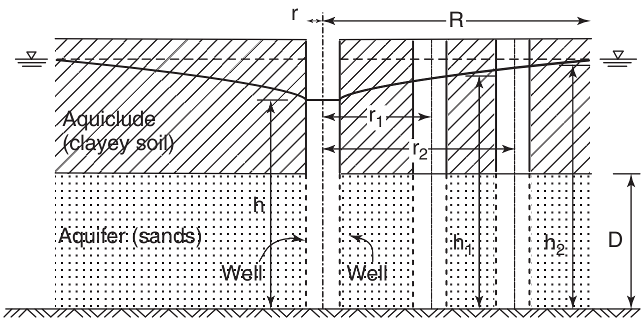


Figure 7.42 Pumping test in confined aquifer using three borings.

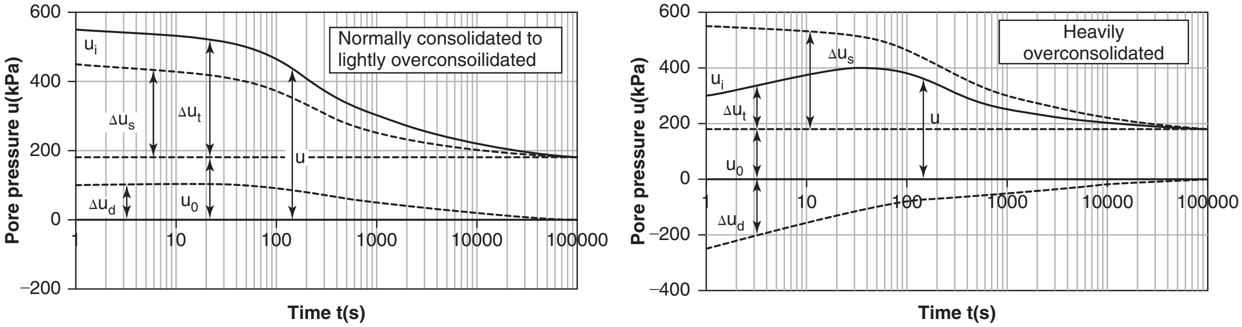


Figure 7.43 Decay of excess pore pressure in piezocone dissipation test.

wall of the borehole, and quick conditions development in high-gradient situations.

7.12.2 Cone Penetrometer Dissipation Test

The cone penetrometer dissipation test (CPDT) is performed during a CPT sounding and makes use of the cone point equipped with a pore pressure measuring sensor: a *piezocone*. The piezocone is pushed to a depth below the groundwater level where the measurement of k has to be made, the penetration stops, the initial excess pore pressure is read, and then the decay of excess pore pressure versus time is recorded. Two situations can arise: heavily overconsolidated soil or normally to lightly overconsolidated soil.

In the case of normally consolidated to lightly overconsolidated soil, the decay of excess pore pressure will be monotonic (Figure 7.43*a*). In the case of heavily overconsolidated soils, the response shows first an increase in excess pore pressure followed by a decrease (Figure 7.43*b*). The reason for this dual behavior is that the total excess pore pressure Δu_t has two components: one is due to the water stress response Δu_s to the mean all-around compression of the soil element (spherical stress tensor); the other is due to the water stress response Δu_d to the shearing of the soil element (deviatoric stress tensor). When the soil element is subjected to an all-around mean pressure, Δu_s is always positive, but when the soil element is subjected to a shear stress, Δu_d can be positive or negative depending on the change in volume of the element during shearing. If the soil element decreases in volume during shearing, it is called *contractive*, Δu_d is positive, and both Δu_s and Δu_d decrease as a function of time (Figure 7.43*a*). If, however, the soil element increases in volume during shearing, it is called *dilatant*, and Δu_d is negative. The combination of Δu_s decreasing with time and Δu_d increasing with time (becoming less negative) leads to a bump on the decay curve (Figure 7.43*b*).

The initial pore pressure when recording starts is u_i. Note that two u_i values exist depending on the location of the pore-pressure measuring device. In the case of a monotonic decay and for the pore-pressure measurement right behind the cone point (shoulder), Parez and Fauriel (1988) proposed a correlation between t_{50} and the hydraulic conductivity k (Figure 7.44), which is well represented by the equation:

$$k(\mathrm{cm/s}) = \left(\frac{1}{251\ t_{50}\,(s)}\right)^{1.25} \tag{7.30}$$

Where k is the hydraulic conductivity in cm/s and t_{50} is the time in seconds to reach a decrease in water stress equal to 50% of the total decrease in water stress.

A typical example is shown in Figure 7.43a for a lightly overconsolidated clay. The time to 50% dissipation is found halfway between the initial value u_i ($t = 1$s in Figure 7.43*a*) and the equilibrium value corresponding to the hydrostatic pressure u_0. In the case of a decay curve exhibiting a rise followed by a decay (highly overconsolidated soil), obtaining the hydraulic conductivity k from the dissipation curve is more complicated (Burns and Mayne 1998).

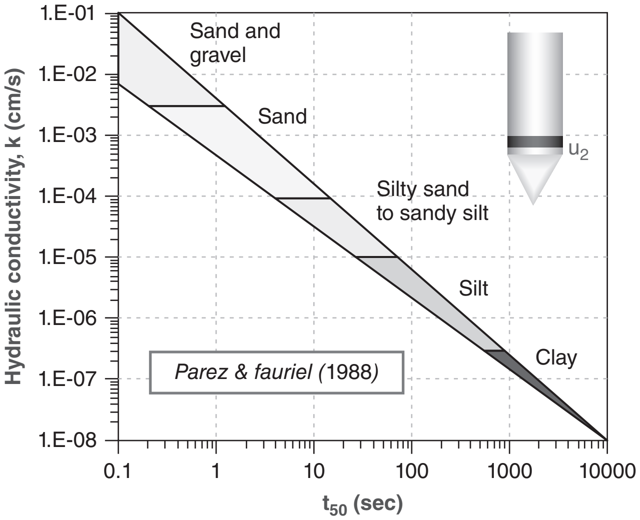


Figure 7.44 Relationship between t_{50} and hydraulic conductivity for piezocone dissipation test. (From Mayne, Christopher, Berg, and DeJong 2002. Courtesy of Professor Paul Mayne, Georgia Institute of Technology, USA.)

7.12.3 Sealed Double-Ring Infiltrometer Test

The sealed double-ring infiltrometer test (SDRIT) was developed in the late 1970s in the USA and is credited to Steve Trautwein and David Daniel (1994). The SDRIT aims at measuring the hydraulic conductivity at shallow depth in soils above the groundwater level. A typical situation is testing to obtain the hydraulic conductivity k of a 1 m thick clay liner above a free-draining layer of sand and gravel. The test setup starts by placing a square outer ring about 4 m in size in the soil surface and embedding and grouting the walls of the ring about 0.45 m below the surface (Figure 7.45). Then an inner ring is placed in the center of the outer ring and the walls are embedded and grouted about 0.15 m into the ground. The outer ring is open to the atmosphere while the inner ring is sealed. A tube goes from the inner ring to a deformable plastic bag, where it can be easily connected and disconnected. The bag is filled with water and weighed, and the entire system is saturated with water. The SDRIT is often used to test soils that are not saturated, in which case tensiometers are placed at different depths to measure the tension in the water within the layer being tested (see Chapter 10 on water stress for an explanation of how tension occurs in the soil water and Chapter 9 on laboratory tests for an explanation of how tensiometers work). As the water seeps through the unsaturated soil layer below the SDRI, the water fills the voids in the soil, thereby saturating the soil; a wetting front advances and the plastic bag loses water. The volume of water Q leaving the plastic bag and entering the soil is measured by weighing the bag as a function of time.

Reducing the data of an SDRIT requires knowledge of water flow through saturated and unsaturated soils (see chapter 13). Obtaining the hydraulic conductivity k from the SDRIT requires some assumptions: (1) steady-state seepage; (2) vertical, one-dimensional flow; and (3) saturated conditions. If the soil is unsaturated to start with, it will take time for the water to permeate through the soil layer thickness and saturate the soil. This time can be several weeks. To obtain the hydraulic conductivity k from the SDRIT data, the following equations are used:

$$v = k\, i \tag{7.31}$$

This is called *Darcy's law* and is explained in Chapter 13 on flow through soils; v is the discharge velocity; and i is the hydraulic gradient, defined as the loss of total head Δh_t of the flowing water per distance travelled Δz.

$$i = \frac{\Delta h_t}{\Delta z} \tag{7.32}$$

Conservation of mass leads to:

$$V_f = v_d A t C \tag{7.33}$$

where V_f is the volume of water that has infiltrated the soil in a time t, A is the plan view area of the inner ring, and v_d is the discharge velocity. This leads to an expression for k:

$$k = \frac{\dfrac{V_f}{At}}{\dfrac{\Delta h_t}{\Delta z}} \tag{7.34}$$

If the test is run long enough that the whole layer becomes saturated, then Δh_t is the vertical distance from the bottom of the layer to the level of the water in the outer ring and Δz is the thickness of the layer. The tensiometer readings help in deciding when this stage has been reached. If this assumption is made but the wetting front has not penetrated the whole layer, then i will be underestimated and the k value obtained will be lower than the true k value. If the test does not reach this stage and the water front has penetrated to a depth D_w below the top of the soil surface, the value of Δz is D_w and the value of Δh_t is:

$$\Delta h_t = H + D_w + h_p \tag{7.35}$$

where h_p is the tension in the water on the wetting front expressed in height of water. This value can be obtained from the tensiometer readings. Here two assumptions can be made: (1) h_p is given by the tensiometers, or (2) $h_p = 0$. In practice, the second assumption seems to give more acceptable results, especially as the test is often run to prove that the hydraulic conductivity of the soil layer is lower than 10^{-9} m/s (clay liner for waste disposals). Indeed, with assumption 2 ($h_p = 0$), Δh_t is underestimated and k is overestimated.

When the layer being tested swells, it is necessary to take the swelling into account. In this case some of the water leaving the plastic bag is stored in the swelling process while some of the water is seeping through the soil. Ignoring the

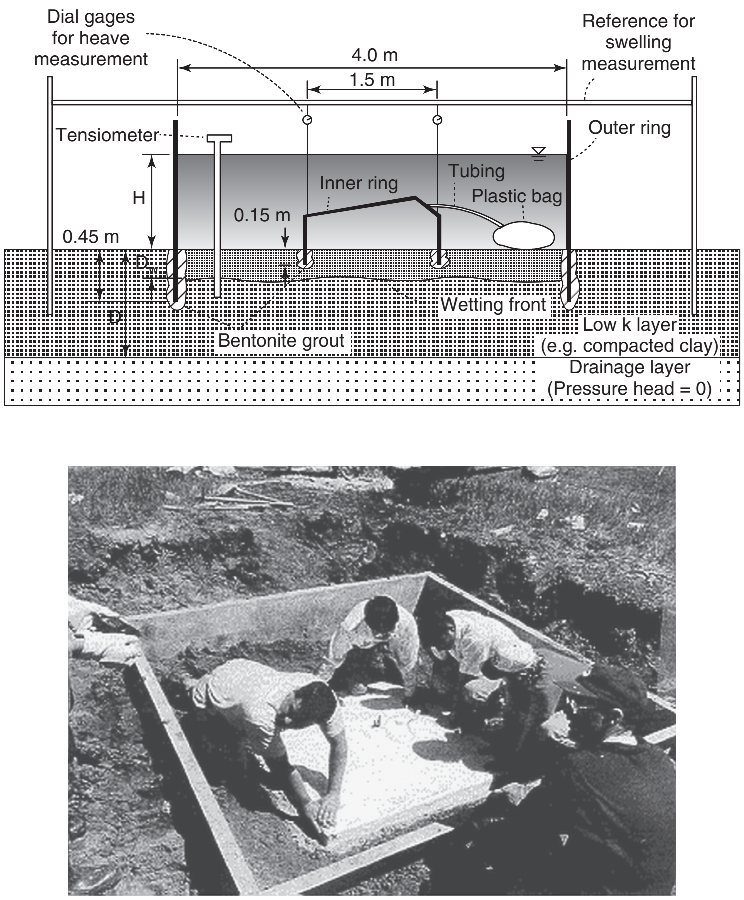


Figure 7.45 Sealed double-ring infiltrometer. (Courtesy of Professor Xiaodong Wang, University of Wisconsin, USA.)

swelling component would give an overestimated value of V_f and therefore an overestimated value of k. The volume of water V_s used to increase the volume of the soil through swelling is measured as follows: A reference beam is set up above the SDRI (Figure 7.44) and the vertical movement of the inner ring is recorded with respect to that beam (using dial gages, for example). The volume V_s corresponding to the vertical movement of the inner ring is subtracted from the volume of water V_t leaving the plastic bag to obtain the true volume V_f. flowing through the soil.

7.12.4 Two-Stage Borehole Permeameter Test

The two-stage borehole permeameter test (TSBPT) was developed in the USA in the 1980s and is credited to Gordon Boutwell (Boutwell and Derick 1986). The TSBPT aims at measuring the vertical and horizontal hydraulic conductivity at shallow depth in soils above the groundwater level. A typical situation is testing to obtain the hydraulic conductivity k of a 1 m thick clay liner above a free-draining layer of sand and gravel. The test takes place in two stages.

Stage 1 consists of drilling a hole (for example, 0.5 m deep and 0.1 m in diameter), inserting a permeameter (e.g., open PVC 75 mm diameter pipe with graduated cylinder above, Figure 7.46a) in the open hole, sealing the permeameter to the walls of the borehole by grouting, and keeping the bottom of the boring open and intact. Once the borehole is sealed, the test consists of filling the permeameter with water and letting the water seep into the soil through the bottom of the casing. The drop in water level in the graduated tube is recorded as a function of time. The hydraulic conductivity k_1 from stage 1 is calculated from the following equation (Hvorslev 1949):

$$k_1 = \frac{\pi d^2}{11D(t_2 - t_1)} Ln \frac{h_1}{h_2} \tag{7.36}$$

where d is the diameter of the graduated tube above the permeameter, D is the diameter of the permeameter, and h_1 and h_2 are the heights of water above the bottom of the casing recorded at times t_1 and t_2 respectively. The k_1 values are plotted as a function of time until steady state is reached. Note that this equation assumes that the material below the casing is uniform to a large depth. It is prudent to use it only if the depth to the next layer is at least 5 borehole diameters below the bottom of the boring.

Stage 2 consists of deepening the borehole (for example, 0.2 m deeper and 75 mm in diameter), and repeating the permeability test (falling head test). The hydraulic conductivity

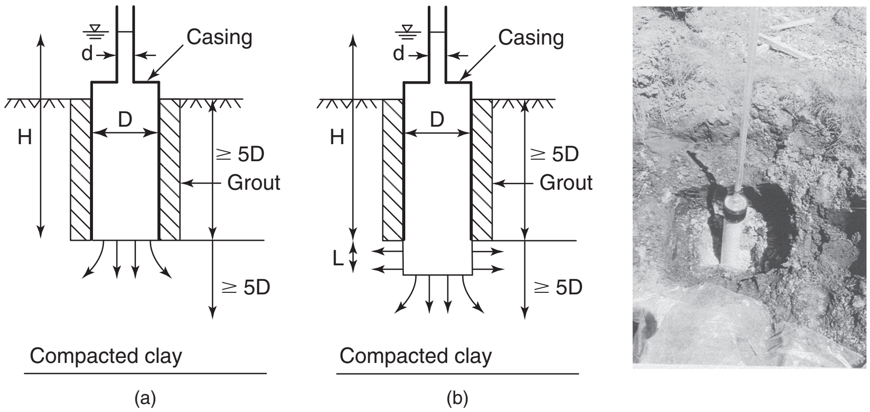


Figure 7.46 Two-stage borehole permeameter: (a) Stage 1; (b) Stage 2. (Third picture: Courtesy of Craig Benson, University of Wisconsin.)

k_2 from stage 2 is calculated from the following equations (Hvorslev 1949):

$$k_2 = \frac{A}{B} Ln \frac{h_1}{h_2} \tag{7.37}$$

with

$$A = d^2 \ Ln\left(\frac{L}{D} + \sqrt{1 + \left(\frac{L}{D}\right)^2}\right) \tag{7.38}$$

$$B = 8L(t_2 - t_1)\left(1 - 0.562e^{-1.57\frac{L}{D}}\right) \tag{7.39}$$

Note that A is in m^2 while B is in m.s. The k_2 values are plotted as a function of time until steady state is reached. Then the anisotropy can be taken into account by using the ratio k_2/k_1 and relating it to the ratio k_h/k_v. This is done by first defining m as:

$$m = \sqrt{\frac{k_h}{k_v}} \tag{7.40}$$

where k_h and k_v are the hydraulic conductivity in the horizontal and vertical directions respectively. Then k_2/k_1 is related to m through:

$$\frac{k_2}{k_1} = m \frac{Ln\left(\frac{L}{D} + \sqrt{1 + \left(\frac{L}{D}\right)^2}\right)}{Ln\left(\frac{mL}{D} + \sqrt{1 + \left(\frac{mL}{D}\right)^2}\right)} \tag{7.41}$$

In equation 7.41, all quantities are known except m, which can therefore be obtained. Alternatively, m can be found by using Figure 7.47, which presents k_2/k_1 versus k_h/k_v for L/D ratios of 1, 1.5, and 2. Once m is known, k_h and k_v can be found as follows (Daniel, 1989):

$$k_h = mk_1 \tag{7.42}$$

$$k_v = k_1/m \tag{7.43}$$

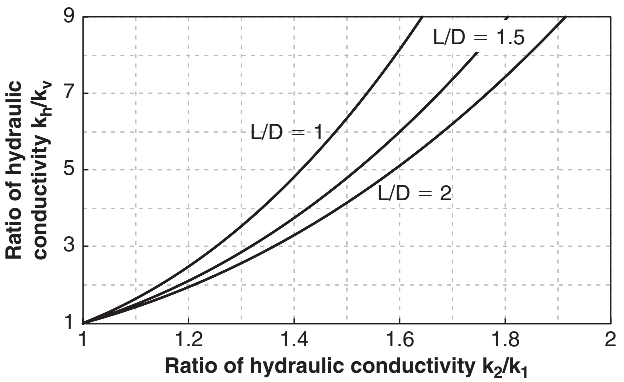


Figure 7.47 Relationship between k_1/k_2 and m for two-stage borehole permeameter. (After Daniel 1989.)

The analysis of both stage 1 and stage 2 presented here makes a number of limiting assumptions that may or may not be verified in the field (Daniel 1989).

7.13 OFFSHORE IN SITU TESTS

The in situ tests most commonly used offshore are the cone penetrometer test and the vane shear test. Other in situ tests used offshore include the pressuremeter test, the dilatometer test, and a number of geophysical tests (see Chapter 8).

The offshore CPT is used for stratigraphy, classification, undrained shear strength in fine-grained soils, and friction angle and relative density in coarse-grained soils. It is performed from the seabed or down a borehole. The seabed systems (Figure 7.48) are lowered to the seabed and provide the vertical reaction against which to push the CPT. A total push of 100 kN can be expected from these units. The rods are prestrung on the seabed unit. The downhole systems (Figure 7.49) consist of lowering the CPT system through the drill string that drilled the borehole, latching the CPT system to the bottom of the drill string, and pushing the CPT into the soil below by using the mud pressure in the drill string.

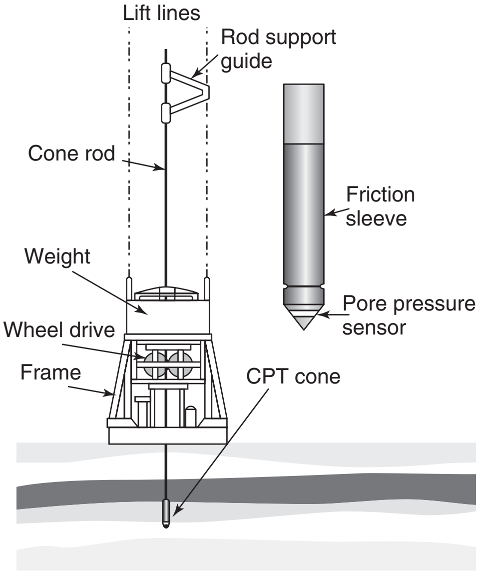


Figure 7.48 Seabed units to deploy the CPT offshore. (a and b: Image courtesy Swan Consultants Ltd., Copyright EFS Danson 2005.)

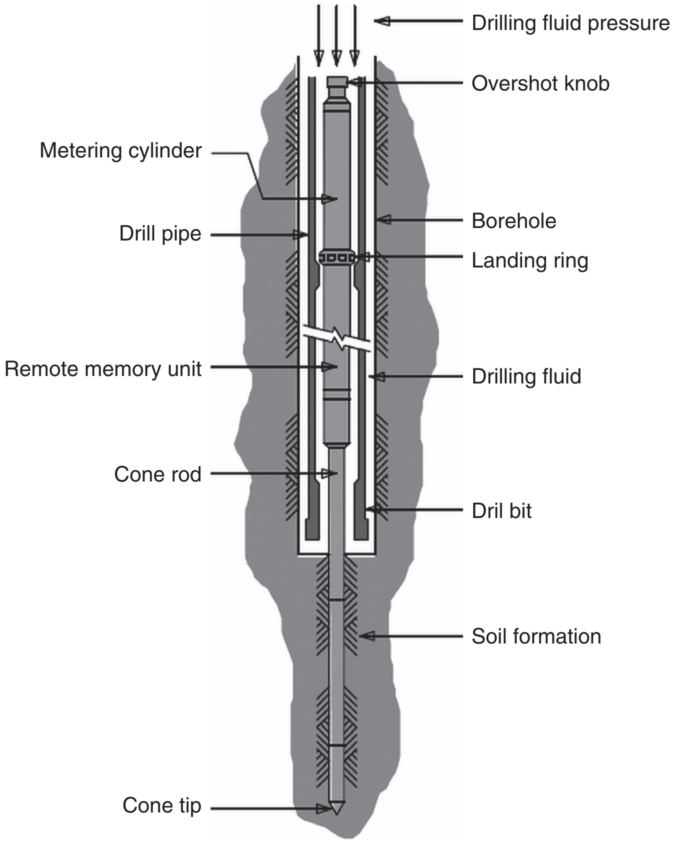


Figure 7.49 “Dolphin” downhole system to deploy the CPT offshore. (Courtesy of FUGRO Inc.)

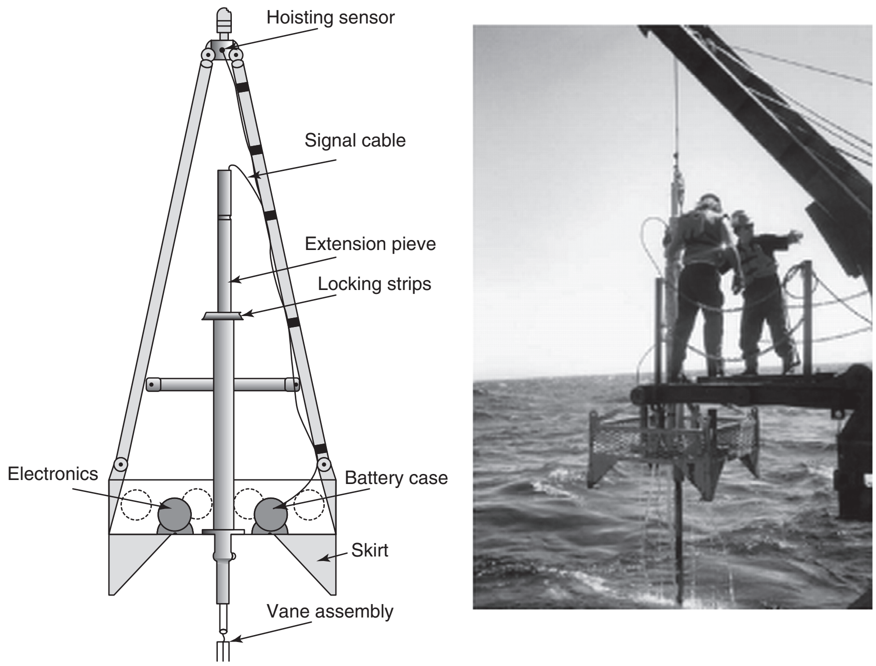


Figure 7.50 Seabed system for vane shear test. (Image courtesy Swan Consultants Ltd., Copyright EFS Danson 2005.)

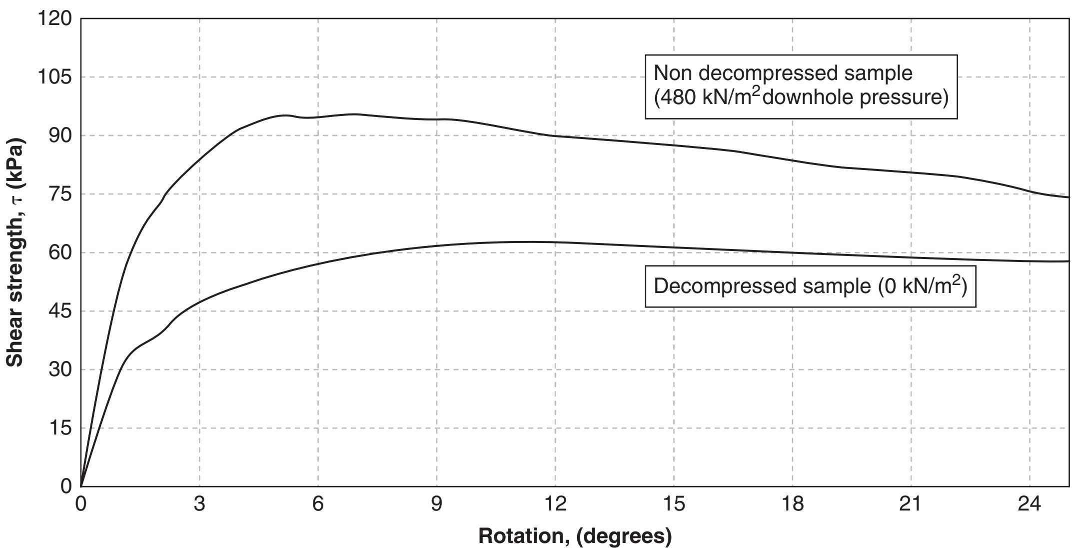


Figure 7.51 Influence of sample disturbance on vane shear results. (After Denk et al. 1981.)

The drill string is typically steadied by clamping the drill string to an external mass resting on the seabed.

The offshore vane shear test is used to measure the undrained shear strength of fine-grained soils. Like the CPT, the VST can be performed from a downhole tool (Figure 7.49) or from a seabed platform (Figure 7.50). Although samples can be taken, obtaining the undrained shear strength from such samples in the laboratory suffers from the decompression of the sample when it is brought back to the surface. In gassy soils, this decompression can be very significant and reduce the undrained shear strength by up to 40% (Figure 7.51; Denk et al. 1981). The VST measures the undrained shear strength in situ and therefore does not allow decompression. As a result, the value obtained is much more reliable.

PROBLEMS

7.1 Assume that the blow count profile shown in Figure 7.4 is an uncorrected blow count profile obtained for a silty sand. Assume further that the energy recorded during these SPT tests was 332 J, that the groundwater level was at the surface, and that the soil has a significant amount of silt. Create the corrected profile for energy level N_{60}, the corrected profile for stress level N_1, and the corrected profile for silt content N'. Then create the combined corrected profile for energy, stress level, and silt content, $N'_{1(60)}$.

7.2 A pressuremeter test gives the test curve shown in Figure 7.2s. Calculate the first load modulus E_0, the reload modulus of the first loop E_{r1}, the yield pressure p_y, the horizontal pressure p_{oh} corresponding to the reestablishment of the horizontal in situ stress, and the limit pressure p_L. What do you think each parameter can be used for?

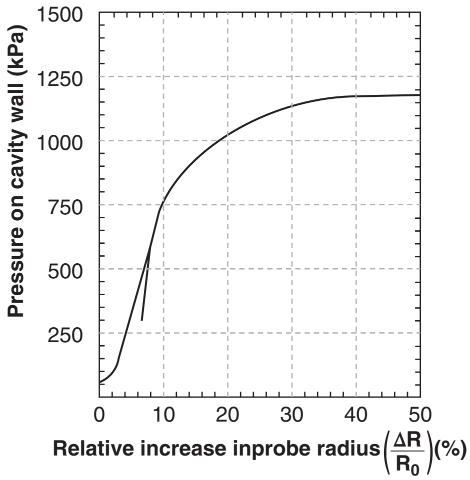


Figure 7.2s Pressuremeter test results.

7.3 Use the CPT profiles of Figure 7.8 to identify the main soil layers. Then classify the soil in each layer according to the CPT classification systems of Figure 7.10 and Figure 7.11.

7.4 Develop the equation for a rectangular vane that links the maximum torque $T_{\max}$ to the undrained shear strength s_u of a fine-grained soil.

7.5 Why is the vane test not used in coarse-grained soils? Develop a way, including placing instrumentation on the vane, that would allow the vane test to give the effective stress friction angle of a sand with no effective stress cohesion intercept.

7.6 A borehole shear test is performed in a saturated clay below the water level. The test is performed fast enough to ensure no drainage. When the horizontal pressure is applied, the plates penetrate 4 mm into the soil of the borehole wall. How long should the plates be for the end effect created by the resistance of the wedge at the leading edge of the plates to represent less than 10% of the shear force measured?

7.7 A plate test gives the load settlement curve shown in Figure 7.26. The plate is 0.3 m in diameter and the test is performed at the ground surface. Calculate the soil modulus from the early part of the plate test curve. Would you use this modulus to calculate the settlement of a 3 m by 3 m square footing? Explain.

7.8 Use the elastic settlement equation for a plate test to explain why the modulus of subgrade reaction K is not a soil property while the soil modulus E is. Which one would you rather use and why?

7.9 Calculate the settlement of a footing on sand after 50 years under a pressure of 100 kPa if the settlement after 1 hour under a pressure of 100 kPa during a load test is 10 mm. The soil has a viscous exponent $n = 0.04$.

7.10 Pocket erodometer tests (PETs) are performed on the end of Shelby tube samples retrieved from a levee. The average depth of the PET holes is 6 mm and the standard deviation is 2 mm. Estimate the rate of erosion if the mean velocity overflowing the levee will be 5 m/s. If the levee is subjected to overtopping for 2 hours (hurricane), how much erosion is likely to take place?

7.11 A sand cone apparatus is used to check the dry density of a compacted soil. The weight of dry sand used to fill the test hole and the funnel of the sand cone device is 8.7 N. The weight of dry sand used to fill the cone funnel is 3.2 N. The unit weight of the dry sand is calibrated to be 15.4 kN/m^3. The weight of the wet soil taken out of the test hole is 7.5 N and the water content of the soil from the test hole is 13.2%. Calculate the dry density of the compacted soil.

7.12 A lightweight deflectometer is used to obtain the modulus of the compacted soil. The plate is 200 mm in diameter and the results of the tests are shown in Figure 7.36. Calculate the modulus of deformation of the soil. What approximate stress level and strain level does it correspond to?

7.13 A borehole is drilled into a deep and uniform clay layer to a depth of 1.5 m. A 75 mm inside diameter casing is lowered to the bottom of the 100 mm diameter borehole and sealed to the borehole walls. The water is bailed out so that the water level starts 1 m below the groundwater level outside of the casing at time equal 0. Three days later the water level has risen 0.3 m in the casing. Calculate the hydraulic conductivity *k* of the clay layer.

7.14 A 10 m thick layer of silty sand is underlain by a deep layer of high-plasticity clay. The groundwater level is 2 m below the ground surface. A 100 mm diameter boring is drilled to a depth of 10 m and cased with a screen that allows the water to enter the borehole freely along the borehole walls. A pump is set up to pump the water out of the hole and reaches a steady state condition after 2 days; at that time it is able to maintain the water level in the hole at a depth of 6 m when the flow rate is 0.2 cubic meters per minute. Additional boreholes indicate that the radius of influence of the depressed water level is 9 m. Calculate the hydraulic conductivity of the silty sand layer.

7.15 A cone penetrometer dissipation test is performed at a depth of 15.2 m below the groundwater level in a silt deposit. The results of the tests are given in Figure 7.43a. Calculate the hydraulic conductivity of the silt layer.

7.16 A sealed double-ring infiltrometer is used to evaluate the field-scale hydraulic conductivity of a 1 m thick clay liner underlain by a free-draining layer of sandy gravel. The SDRI has a square outside ring that is 4 m by 4 m and an inside ring that is 1 m by 1 m. The wall of the outer ring is embedded and sealed 0.45 m below the ground surface and the wall of the inner ring is embedded and sealed 0.15 m below the ground surface. Water is poured into the infiltrometer to a height of 0.5 m above the ground surface and the inner ring is capped. After a period of one week, during which the liner below the infiltrometer becomes saturated and a steady-state flow develops, the daily volume of water flowing into the liner is 0.01 m^3 as measured by a plastic bag connected to the sealed inside ring. The soil swells, and vertical movement measurements of the inside ring indicate that this swelling amounts to 0.004 m^3 per day. Calculate the hydraulic conductivity of the liner.

7.17 A two-stage permeameter test is conducted to evaluate the vertical and horizontal hydraulic conductivity of a clay liner. In stage 1, a 0.1 m diameter borehole is drilled to a depth of 0.35 m. A 0.075 m inside diameter pipe is lowered to the bottom of the open borehole and sealed to the walls of the borehole. A 10 mm inside diameter graduated tube is placed on top of the 75 mm diameter pipe; then the pipe and the falling head permeameter fitted on top of it are saturated and the water seeps through the liner. After reaching a steady state, the following measurements are recorded. At time equal 0, the water is 0.6 m above the ground surface. After 30 minutes of infiltration, the water has dropped to a height of 0.5 m above the ground surface. In stage 2, a 75 mm borehole is advanced 0.2 m below the bottom of the stage 1 borehole (0.55 m below surface). The falling head permeameter test is repeated and the water level falls from 0.6 m above the ground surface at time equal 0 to 0.5 m above the ground surface in 5 minutes. Calculate the vertical and horizontal hydraulic conductivity of the clay liner.

7.18 Discuss the advantages and drawbacks of in situ tests versus laboratory tests.

Problems and Solutions

Problem 7.1

Assume that the blow count profile shown in Figure 7.4 is an uncorrected blow count profile obtained for a silty sand. Assume further that the energy recorded during these SPT tests was 332 J, that the groundwater level was at the surface, and that the soil has a significant amount of silt. Create the corrected profile for energy level N_{60}, the corrected profile for stress level N_1, and the corrected profile for silt content N'. Then create the combined corrected profile for energy, stress level, and silt content, $N'_{1(60)}$.

Solution 7.1

The corrections of the SPT values are shown in Table 7.1s and are based on the following formulas:

$$\text{Correction for energy level}: N_{60} = N_{\text{measured}} \times \left(\frac{E_{\text{measured}}\ (\text{J})}{285\ (\text{J})} \right)$$

$$\text{Correction for stress level}: N_1 = N_{\text{measured}} \times \left(\frac{100}{\sigma'_{v0}\ (\text{kPa})} \right)^{0.5}$$

$$\text{Correction for silt content}: N' = 15 + \left(\frac{N_{\text{measured}} - 15}{2}\right)$$

$$\text{Combined corrections}: N'_{1(60)} = 15 + \left(\frac{N_{60} \times \left(\frac{100}{\sigma'_{0v}}\right)^{0.5} - 15}{2}\right)$$

Table 7.1s Corrected SPT Values

Depth	Measured	Energy level		Stress level			Silt	Combination
	N_{measured}	E_{measured}	N_{60}	γ_{sat}	σ'_{ov}	N_1	N'	$N'_{1(60)}$
m	bpf	J	bpf	kN/m^3	kPa	bpf	bpf	bpf
1.5	15	332	17	19	14	40	15	31
3	20	332	23	19	28	38	18	30
4.5	17	332	20	19	41	26	16	23
6	12	332	14	19	55	16	14	17
7.5	18	332	21	19	69	22	17	20
9	21	332	24	19	83	23	18	21
10.5	24	332	28	19	96	24	20	22
12	28	332	33	19	110	27	22	23
13.5	31	332	36	19	124	28	23	24
15	30	332	35	19	138	26	23	22
16.5	32	332	37	19	152	26	24	23
18	29	332	34	19	165	23	22	21
19.5	31	332	36	19	179	23	23	21

The corrections of the SPT values are plotted on the graph shown in Figure 7.1s.

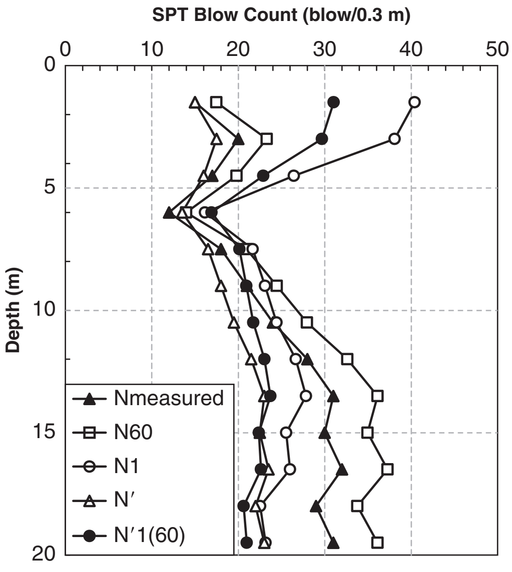


Figure 7.1s Corrected SPT values.

Problem 7.2

A pressuremeter test gives the test curve shown in Figure 7.2s. Calculate the first load modulus E_0, the reload modulus of the first loop E_{r1}, the yield pressure p_y, the horizontal pressure p_{oh} corresponding to the reestablishment of the horizontal in situ stress, and the limit pressure p_L. What do you think each parameter can be used for?

Solution 7.2

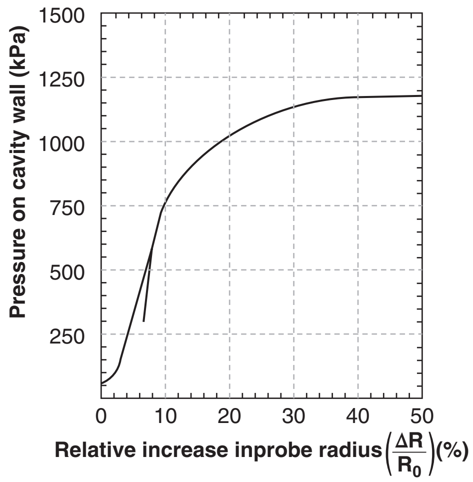


Figure 7.2s Pressuremeter test results.

According to the test results shown in Figure 7.2s, the following parameters are obtained:

- First load modulus $E_0 = (1 + 0.35)\dfrac{1500}{(0.18 - 0.017)} = 12423$ kPa
- The reload modulus of the first loop $E_{r1} = (1 + 0.35)\dfrac{1500}{(0.12 - 0.05)} = 28928$ kPa
- The yield pressure $p_y = 700$ kPa
- The horizontal pressure $p_{0h} = 120$ kPa
- The limit pressure $p_L = 1200$ kPa

The applications of the PMT include the design of deep foundations under horizontal loads, the design of shallow foundations, the design of deep foundations under vertical loads, and the determination of a modulus profile and other soil properties. The PMT is not very useful for slope stability and retaining structures.

The first load and reload modulus can be used in settlement analysis. The yield pressure can be used as an upper limit for the allowable foundation pressures. The limit pressure can be used to calculate the ultimate capacity of the foundation.

Problem 7.3

Use the CPT profiles of Figure 7.8 to identify the main soil layers. Then classify the soil in each layer according to the CPT classification systems of Figure 7.10 and Figure 7.11.

Solution 7.3

A total of 10 layers are identifiable from the CPT profiles of Figure 7.8 and are shown in Figure 7.3s and Table 7.2s. Furthermore, the porewater pressure profile can be extended back to zero pressure and indicates that the water level is at a depth of 2.5 m below the ground surface. The classifications of the soil layers based on Figures 7.10 and 7.11 are presented in Table 7.2s, Figure 7.4s, and Figure 7.5s. At a coarser level, the stratigraphy can be simplified as shown in Figure 7.6s.

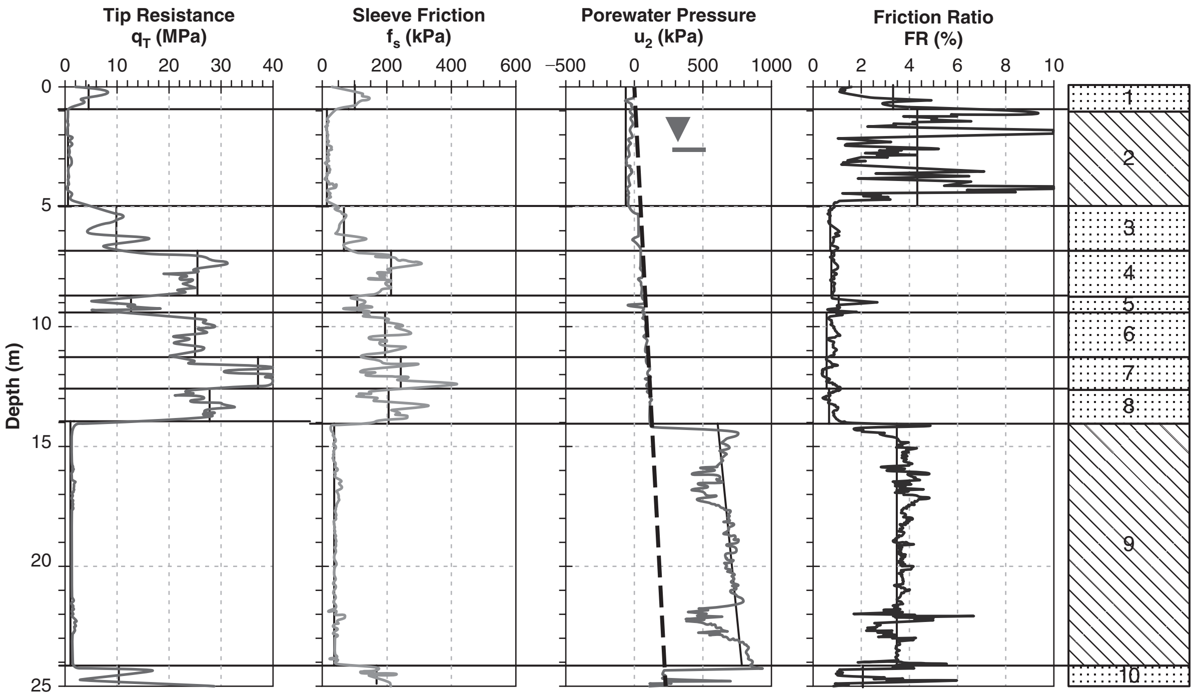


Figure 7.3s Soil layers. (Courtesy of Professor Paul Mayne, Georgia Institute of Technology)

Table 7.2s Classification of Soil Layers

Layer	Depth (m)	q_t (Mpa)	q_t (Bar)	f_s (kPa)	FR (%)	Figure 7.10	Figure 7.11
1	0.0–1.0	4.0	40	100	3.40	Sandy silts & silt	Silty sand
2	1.0–5.0	0.8	8	10	4.30	Clays	Clay
3	5.0–7.0	10.0	100	60	0.70	Sands	Sand to silty sand
4	7.0–8.8	26.0	260	210	0.70	Sands	Sand
5	8.8–9.5	13.0	130	100	1.10	Sands	Sand to silty sand
6	9.5–11.3	26.0	260	200	0.60	Sands	Gravelly sand
7	11.3–12.7	37.0	370	250	0.60	Sands	Gravelly sand
8	12.7–14	28.0	280	200	0.60	Sands	Gravelly sand
9	14–24.2	0.9	9	40	2.70	Clayey silts	Silty clay
10	24.2–25	10.0	100	160	2.00	Silty sands	Silty sand

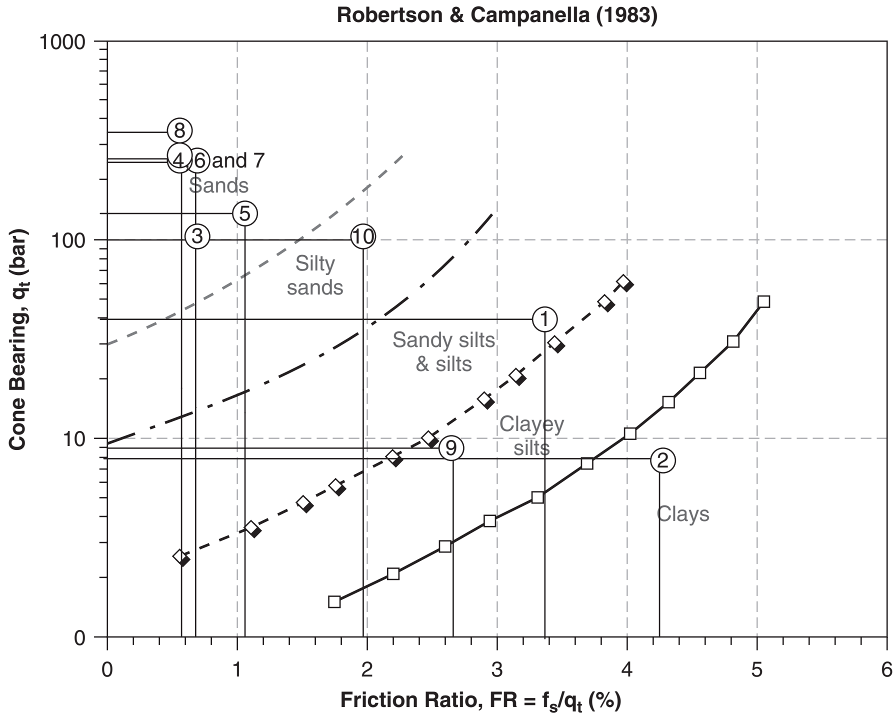


Figure 7.4s Soil classification based on CPT results. (Courtesy of Professor Paul Mayne, Georgia Institute of Technology)

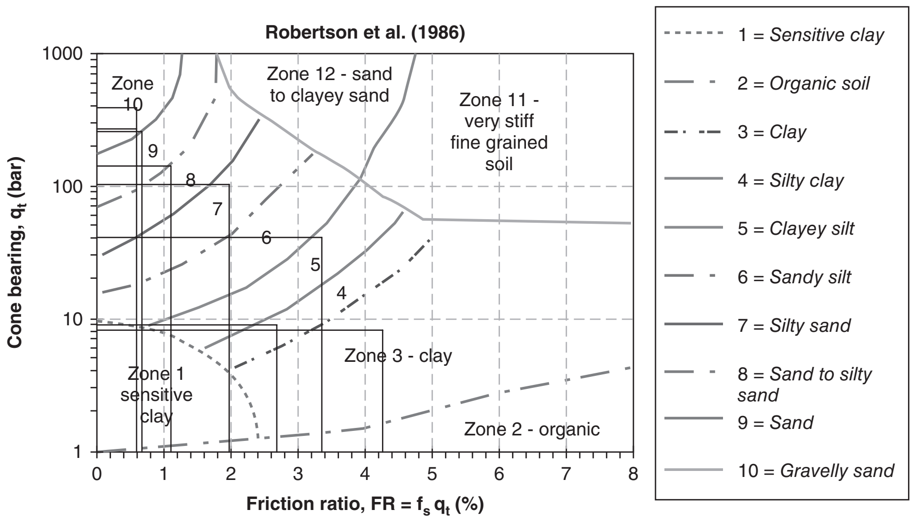


Figure 7.5s Soil classification based on CPT results. (Courtesy of Professor Paul Mayne, Georgia Institute of Technology)

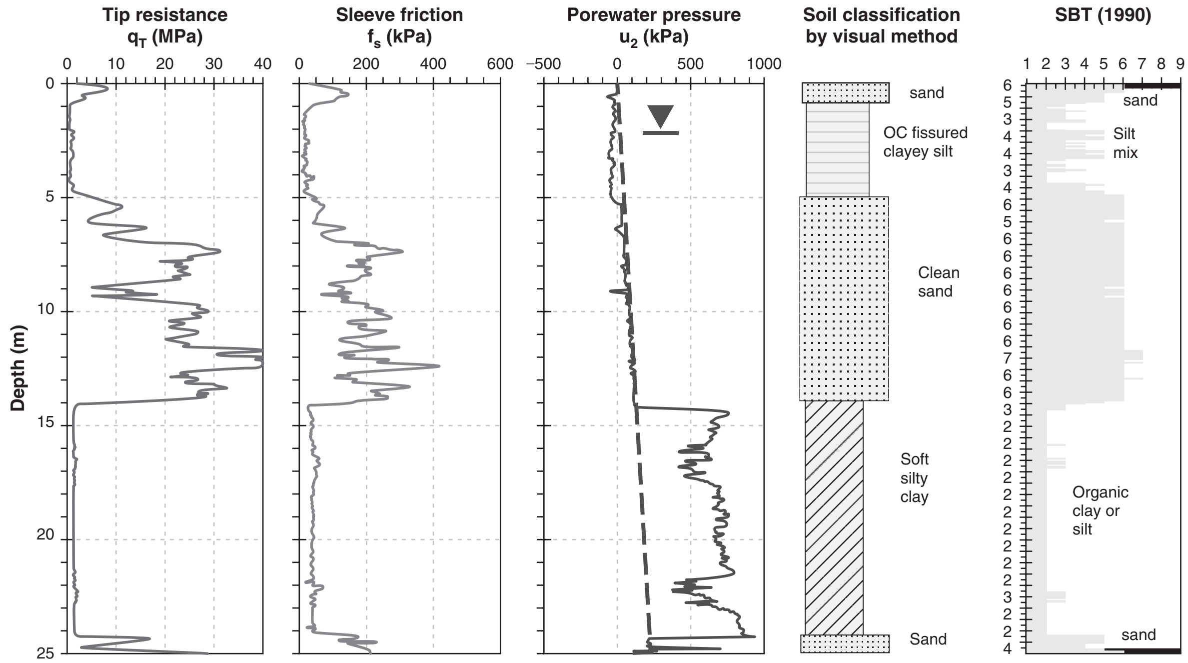


Figure 7.6s Simplified stratigraphy. (From Mayne 2007a, b, Courtesy of Professor Paul Mayne, Georgia Institute of Technology, USA.)

Problem 7.4

Develop the equation for a rectangular vane that links the maximum torque T_{max} to the undrained shear strength s_u of a fine-grained soil.

Solution 7.4

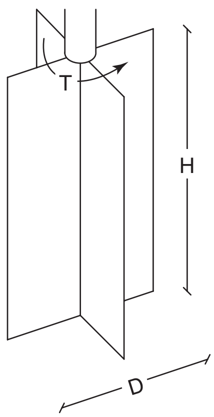


Figure 7.7s Vane subjected to torque.

The failure surface around the vane is a cylinder with a diameter D and a height H. The torque generated from the sides of the cylinder is:

$$T_1 = \pi D H s_u \frac{D}{2}$$

The torque generated by the top and bottom of the cylinder (ignoring the area occupied by the rod) is:

$$T_2 = \int_0^{\frac{D}{2}} 2.\pi.r.s_u.r.dr = 2.\pi.s_u\left(\frac{r^3}{3}\right)_0^{\frac{D}{2}} = \pi.s_u\frac{D^3}{12}$$

$$T = T_1 + 2T_2 = \pi DHs_u\frac{D}{2} + 2\pi.s_u.\frac{D^3}{12}$$

$$T = \pi s_u.D^2\left(\frac{H}{2} + \frac{D}{6}\right)$$

For vanes with H = 2D, the equation becomes:

$$T_1 = \frac{7}{6}\pi s_u D^3$$

Problem 7.5

Why is the vane test not used in coarse-grained soils? Develop a way, including placing instrumentation on the vane, that would allow the vane test to give the effective stress friction angle of a sand with no effective stress cohesion intercept.

Solution 7.5

The vane test gives one measurement: the torque at failure. It can easily be used to obtain the undrained shear strength of a fine-grained soil because in this case the strength is represented by one parameter, s_u. The vane test cannot be used easily to obtain the drained or effective stress parameters (c and ϕ) because we need three equations to solve for the three parameters involved: σ', c, and ϕ. The shear strength equation is:

$$\tau_f = c + \sigma' \tan\varphi$$

If c = 0, the shear strength equation becomes:

$$\tau_f = \sigma' \tan\varphi$$

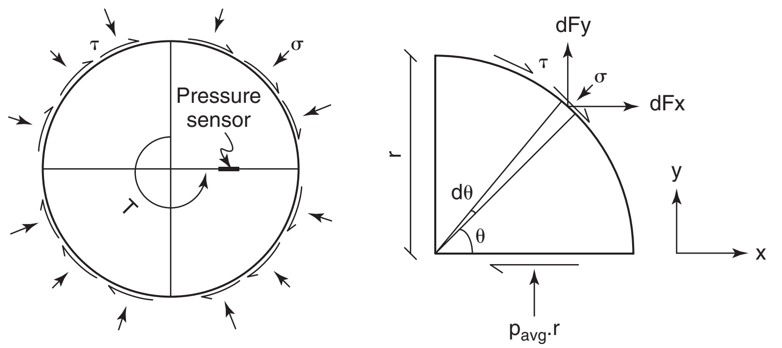


Figure 7.8s Applied stresses on vane.

To get ϕ from the vane test in this case, it is necessary to make two separate measurements. This can be accomplished by placing a pressure sensor on one of the blades, as shown in Figure 7.8s. A free-body diagram of a quadrant of the failing soil mass gives the following equations:

$$\begin{cases} dF_y = \sigma.r.d\theta.\sin\theta + \tau.r.d\theta.\cos\theta \\ dF_x = \sigma.r.d\theta.\cos\theta + \tau.r.d\theta.\sin\theta \end{cases}$$

Based on these equilibrium equations:

$$p.r = F_y = \int_0^{\frac{\pi}{2}} (\sigma.r.\sin\theta + \tau.r.\cos\theta)d\theta = -\sigma.r.\cos\theta + \tau.r.\sin\theta|_0^{\frac{\pi}{2}} = (\tau + \sigma)r$$

$$p = \tau + \sigma$$

At failure:

$$\tau_f = \sigma' \tan\varphi$$

$$\sigma' = p - \tau_f \to \tau_f = (p - \tau_f)\tan\varphi \to \tau_f = \frac{\tan\varphi}{1+\tan\varphi} p$$

From problem 7.4, we have:

$$T = \pi D H \tau_{side}\frac{D}{2} + \pi\tau_{top}\frac{D^3}{12} + \pi\tau_{bottom}\frac{D^3}{12}$$

$$T = \pi D H \frac{\tan\varphi}{1+\tan\varphi} p \frac{D}{2} + \pi\gamma' z \tan\varphi \frac{D^3}{12} + \pi\gamma'(z+H)\tan\varphi\frac{D^3}{12}$$

$$T = \pi\frac{D^2}{2}H\frac{\tan\varphi}{1+\tan\varphi}p + (2z+H)\gamma'\pi\tan\varphi\frac{D^3}{12}$$

$$(2z+H)\pi\gamma'\frac{D^3}{12}\tan^2\varphi + \left((2z+H)\pi\gamma'\frac{D^3}{12} + \frac{1}{2}\pi p D^2 H - T\right)\tan\varphi - T = 0$$

$$\tan\phi = \frac{-B+\sqrt{B^2+4AT}}{2A}$$

$$A = (2z+H)\pi\gamma'\frac{D^3}{12}$$

$$B = (2z+H)\pi\gamma'\frac{D^3}{12} + \frac{1}{2}\pi p D^2 H - T$$

T: torque applied to the vane
D: diameter of the vane
H: height of the vane
ϕ: internal friction angle of sand
p: pressure on the blade of the vane (which is measured by a sensor)
Y: unit weight of soil
z: depth of top of the vane

Problem 7.6

A borehole shear test is performed in a saturated clay below the water level. The test is performed fast enough to ensure no drainage. When the horizontal pressure is applied, the plates penetrate 4 mm into the soil of the borehole wall. How long should the plates be for the end effect created by the resistance of the wedge at the leading edge of the plates to represent less than 10% of the shear force measured?

Solution 7.6

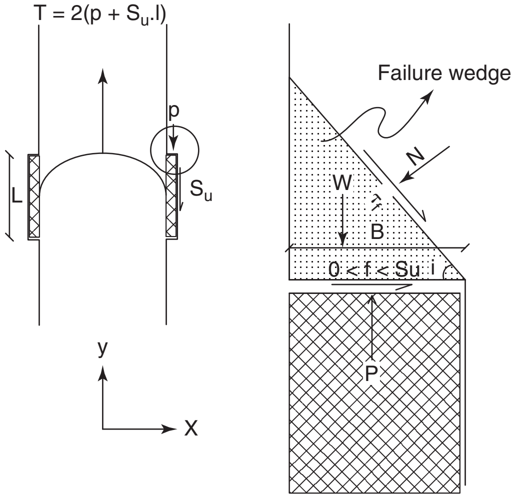


Figure 7.9s Borehole shear test.

$$\begin{cases} \sum F_x = 0 \rightarrow N \sin i = fB + \dfrac{\tau_f.B}{\cos i}\cos i \\ \sum F_y = 0 \rightarrow p = N\cos i + W + \dfrac{\tau_f.B}{\cos i}\sin i \end{cases}$$

$$N = \frac{f.B}{\sin i} + \frac{\tau_f B}{\sin i}$$

$$W = \frac{1}{2}\gamma B^2 \tan i$$

$$p = \left(\frac{f.B}{\sin i} + \frac{\tau_f B}{\sin i}\right)\cos i + \frac{1}{2}\gamma B^2 \tan i + \frac{\tau_f B}{\cos i}\sin i$$

$$p = \frac{f.B}{\tan i} + \frac{\tau_f B}{\tan i} + \frac{1}{2}\gamma B^2 \tan i + \tau_f B \tan i$$

Because B, the penetration of the blades into the soil, is typically very small (say, less than 10 mm), and because the weight of wedge W is a function of B^2, it is reasonable to neglect the influence of the weight of the wedge in calculating P:

$$p \sim \frac{f.B}{\tan i} + \frac{\tau_f B}{\tan i} + \tau_f B \tan i$$

By assuming i = $45^\circ + ^{\varphi}/2$ and using Mohr-Coulomb theory, we have:

$$p \sim \frac{f.B}{\tan\left(45 + \frac{\phi}{2}\right)} + \frac{s_u B\cos\phi}{\tan\left(45 + \frac{\phi}{2}\right)} + s_u B \cos\phi \tan\left(45 + \frac{\phi}{2}\right)$$

$$p \sim \frac{f.B}{\tan\left(45 + \frac{\phi}{2}\right)} + 2s_u B$$

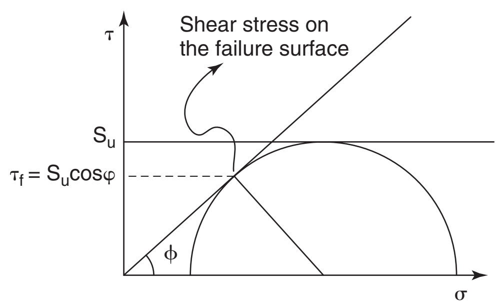


Figure 7.10s Stress envelope.

If $\Phi = 30^\circ$ for upper and lower limits of f, we will have:

$$\begin{cases} f = 0 \rightarrow p = 2s_u B \\ f = s_u \rightarrow p = \left(2 + \dfrac{\sqrt{3}}{3}\right) s_u.B \end{cases}$$

$$2s_u B < p < \left(2 + \frac{\sqrt{3}}{3}\right) s_u.B$$

P is the force needed to fail the wedge of soil above the borehole shear device. If this force must be less than 10% of the force measured by the borehole shear device, then:

$$T_{\text{measured}} = 2(p + s_u.l) \rightarrow \frac{2p}{T} < 10\% \rightarrow p < 0.1(p + s_u.l) \rightarrow l > \frac{0.9p}{0.1s_u}$$

This assumes that the borehole shear device is associated with a plane strain failure, which is a simplifying assumption. In this case, the requirements on the length of the BSD to ensure that the end effect is less than 10% of the measured value are:

$$f = 0 \rightarrow l > 18B$$
$$f = s_u \rightarrow l > 23.2B$$

In the worst condition, which is ($f = s_u$), the length of plates must be longer than 23.2B. If $B = 4$ mm, for example, then $l > 92.8$ mm.

Problem 7.7

A plate test gives the load settlement curve shown in Figure 7.26. The plate is 0.3 m in diameter and the test is performed at the ground surface. Calculate the soil modulus from the early part of the plate test curve. Would you use this modulus to calculate the settlement of a 3 m by 3 m square footing? Explain.

Solution 7.7

The pressure versus displacement/width curve is shown in Figure 7.11s.

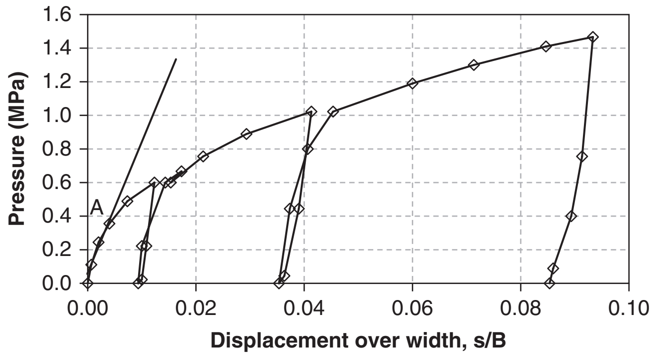


Figure 7.11s Pressure versus displacement/width curve.

The soil modulus is calculated based on point A in Figure 7.11s using the following equation:

$$E = \frac{\pi(1-\nu^2)pB}{4s} = \frac{\pi(1-\nu^2)p}{4 \times \frac{s}{B}} = \frac{\pi(1-0.35^2) \times 0.36}{4 \times 0.004} = 62\,\text{MPa}$$

The soil modulus obtained in this fashion from the plate test is 62 MPa.

I would not use this soil modulus to calculate the settlement of a 3 m by 3 m footing without checking the soil stratigraphy first. The plate bearing test can only give the response of the soil down to a depth of about twice the plate diameter, which is 0.6 m in this case. It cannot reflect the soil property beneath the 3 m by 3 m square footing unless they are the same.

Problem 7.8

Use the elastic settlement equation for a plate test to explain why the modulus of subgrade reaction K is not a soil property while the soil modulus E is. Which one would you rather use and why?

Solution 7.8

The elastic settlement equation for a plate load test is:

$$s = \frac{I(1-\nu^2)pB}{E}$$

Here, I is the shape factor, E is the soil modulus, p is the average pressure under the footing, B is the plate diameter, and ν is the Poisson's ratio. The modulus of subgrade reaction K is calculated as the ratio between the pressure and the settlement:

$$K = \frac{p}{s} = \frac{p}{\frac{I(1-\nu^2)pB}{E}} = \frac{E}{I(1-\nu^2)B}$$

Therefore, the modulus of subgrade reaction K is a function of the soil modulus E and the foundation size B. The larger the foundation is, the smaller the modulus of subgrade reaction is.

I would prefer to use the soil modulus E because it is a true soil property, whereas K is not. Indeed, as shown here, K depends on E and B. Any K value determined from a given size foundation test cannot be used directly for a different size without paying attention to the scale effect.

Problem 7.9

Calculate the settlement of a footing on sand after 50 years under a pressure of 100 kPa if the settlement after 1 hour under a pressure of 100 kPa during a load test is 10 mm. The soil has a viscous exponent $n = 0.04$.

Solution 7.9

Based on equation 7.14, the settlement s_2 of a footing after $t_2 = 50$ years under a pressure of 100 kPa based on the settlement s_1 of the same footing after $t_1 = 1$ hour is:

$$\frac{s_1}{s_2} = \left(\frac{t_1}{t_2}\right)^n$$

With $s_1 = 10\,\text{mm}$, $t_1 = 1\,\text{hr}$, $t_2 = 50$ years $= 50 \times 365 \times 24 = 438{,}000$ hr, and $n = 0.04$:

$$s_2 = \frac{s_1}{\left(\frac{t_1}{t_2}\right)^n} = \frac{10}{\left(\frac{1}{438000}\right)^{0.04}} = 16.8\ \text{mm}$$

So, the calculated settlement of the footing after 50 years under a pressure of 100 kPa is 16.8 mm.

Problem 7.10

Pocket erodometer tests (PETs) are performed on the end of Shelby tube samples retrieved from a levee. The average depth of the PET holes is 6 mm and the standard deviation is 2 mm. Estimate the rate of erosion if the mean velocity overflowing the levee will be 5 m/s. If the levee is subjected to overtopping for 2 hours (hurricane), how much erosion is likely to take place?

Solution 7.10

Using Figure 7.30 and a PET hole depth of 6 mm, the soil category is category III or medium erodibility. For this category, the PET hole varies between 1 mm and 15 mm, corresponding to erosion rates of 3 mm/hr and 2000 mm/hr respectively. For 6 mm, the erosion rate is estimated to be near the middle of the range on the logarithmic scale and an erosion rate of 80 mm/hr is selected (Figure 7.12s). With 2 hours of overtopping at this rate, 160 mm of erosion is estimated.

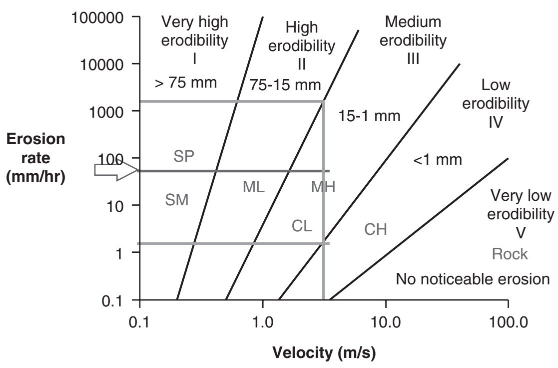


Figure 7.12s Erosion chart for various erosion depths from the PET.

Problem 7.11

A sand cone apparatus is used to check the dry density of a compacted soil. The weight of dry sand used to fill the test hole and the funnel of the sand cone device is 8.7 N. The weight of dry sand used to fill the cone funnel is 3.2 N. The unit weight of the dry sand is calibrated to be 15.4 kN/m^3. The weight of the wet soil taken out of the test hole is 7.5 N and the water content of the soil from the test hole is 13.2%. Calculate the dry density of the compacted soil.

Solution 7.11

The weight of dry sand used to fill the test hole is 8.7 N − 3.2 N = 5.5 N. The volume of the test hole is therefore 5.5×10^{-3} kN/15.4 kN/m^3 = 3.57×10^{-4} m^3. Therefore, the wet unit weight of the compacted soil is 7.5×10^{-3} kN/3.57×10^{-4} m^3 = 21 kN/m^3. Finally, the dry unit weight of the compacted soil is $21/(1 + 0.132) = 18.56$ kN/m^3.

Problem 7.12

A lightweight deflectometer is used to obtain the modulus of the compacted soil. The plate is 200 mm in diameter and the results of the tests are shown in Figure 7.36. Calculate the modulus of deformation of the soil. What approximate stress level and strain level does it correspond to?

Solution 7.12

The modulus of deformation of the soil is:

$$E = 1(1 - 0.35^2)\frac{4 \times 7.5}{\pi \times 0.2 \times 0.55 \times 10^{-3}} = 76.3\,\text{MPa}$$

This modulus of deformation corresponds to the stress level P:

$$P = \frac{4 \times 7.5}{\pi \times 0.2^2} = 238\ \text{kPa}$$

Problem 7.13

A borehole is drilled into a deep and uniform clay layer to a depth of 1.5 m. A 75 mm inside diameter casing is lowered to the bottom of the 100 mm diameter borehole and sealed to the borehole walls. The water is bailed out so that the water level starts 1 m below the groundwater level outside of the casing at time equal 0. Three days later the water level has risen 0.3 m in the casing. Calculate the hydraulic conductivity k of the clay layer.

Solution 7.13

In this case, equation 7.25 applies because the soil layer is deep and uniform, because the casing goes down to the bottom of the borehole, and because the water is bailed out to a depth far below the groundwater level outside of the casing (Figure 7.38). Therefore, the hydraulic conductivity k is obtained from:

$$k_{hyd} = \frac{2\pi r}{11(t_2 - t_1)} \ln \frac{h_1}{h_2}$$

where r is the radius of the casing (0.075 m), t_1 is 0, t_2 is 3 days, h_1 is the depth below the groundwater level at time t_1 (1 m), and h_2 is the depth below the groundwater level at time t_2 (0.7 m). Therefore, the solution is:

$$k_{hyd} = \frac{2\pi \times 0.075}{11(3 - 0)} \ln \frac{1}{0.7} = 5.08 \times 10^{-3}\ \text{m/day} = 5.87 \times 10^{-5}\ \text{mm/sec}$$

Problem 7.14

A 10 m thick layer of silty sand is underlain by a deep layer of high-plasticity clay. The groundwater level is 2 m below the ground surface. A 100 mm diameter boring is drilled to a depth of 10 m and cased with a screen that allows the water to enter the borehole freely along the borehole walls. A pump is set up to pump the water out of the hole and reaches a steady state condition after 2 days; at that time it is able to maintain the water level in the hole at a depth of 6 m when the flow rate is 0.2 cubic meters per minute. Additional boreholes indicate that the radius of influence of the depressed water level is 9 m. Calculate the hydraulic conductivity of the silty sand layer.

Solution 7.14

In this case, equation 7.26 applies because the pervious soil layer to be tested is underlain by an impervious layer, because the uncased boring (or screened boring) is penetrating through the entire pervious layer all the way to the top of the impervious layer, and because the water level is maintained constant by pumping at a flow rate Q, as shown in Figure 7.39. Therefore, the hydraulic conductivity k is obtained from:

$$k = \frac{Q \ Ln\frac{R}{r}}{\pi(H^2 - h^2)}$$

where Q is the flow rate pumped out of the well to maintain the water level constant in the well ($0.2\,m^3/\min = 288\,m^3/day$), r is the radius of the borehole (0.1 m), R is the radius of the zone of influence where the water table is depressed (9 m), H is the vertical distance between the bottom of the boring (impervious layer) and the groundwater level at or further than R (8 m), and h is the vertical distance between the bottom of the boring and the water level in the borehole (4 m). Therefore, the solution is:

$$k = \frac{288 \times \ln\frac{9}{0.1}}{\pi(8^2 - 4^2)} = 8.59\ {}^{m}\!/_{day} = 9.94 \times 10^{-2}\ mm/sec$$

Problem 7.15

A cone penetrometer dissipation test is performed at a depth of 15.2 m below the groundwater level in a silt deposit. The results of the tests are given in Figure 7.43a. Calculate the hydraulic conductivity of the silt layer.

Solution 7.15

We can calculate the hydraulic conductivity of the silt layer using equation 7.30:

$$k(cm/s) = \left(\frac{1}{251 \times t_{50}\,(s)}\right)^{1.25}$$

with $t_{50} = 450$ sec, so $k = \left(\frac{1}{251\times450}\right)^{1.25} = 4.83 \times 10^{-7}\ cm/sec = 4.17 \times 10^{-4}\ {}^{m}\!/_{day} = 4.83 \times 10^{-6}\ mm/sec$

Problem 7.16

A sealed double-ring infiltrometer is used to evaluate the field-scale hydraulic conductivity of a 1 m thick clay liner underlain by a free-draining layer of sandy gravel. The SDRI has a square outside ring that is 4 m by 4 m and an inside ring that is 1 m by 1 m. The wall of the outer ring is embedded and sealed 0.45 m below the ground surface and the wall of the inner ring is embedded and sealed 0.15 m below the ground surface. Water is poured into the infiltrometer to a height of 0.5 m above the ground surface and the inner ring is capped. After a period of one week, during which the liner below the infiltrometer becomes saturated and a steady-state flow develops, the daily volume of water flowing into the liner is 0.01 m^3 as measured by a plastic bag connected to the sealed inside ring. The soil swells, and vertical movement measurements of the inside ring indicate that this swelling amounts to 0.004 m^3 per day. Calculate the hydraulic conductivity of the liner.

Solution 7.16

The hydraulic conductivity of the clay layer for this test can be obtained by using equation 7.34:

$$k = \frac{v}{i} = \frac{\frac{V_f}{A \times t}}{\frac{\Delta h_t}{\Delta z}}$$

$$V_f = V_t - V_s = 0.01 - 0.004 = 6 \times 10^{-3}\ m^3$$

$$\Delta h_t = 1.5\ m$$

$$t = 1\ day$$

$$\Delta z = 1 \text{ m}$$
$$A = 1 \text{ m}^2$$
$$k = \frac{\frac{6 \times 10^{-3}}{1 \times 1}}{\frac{1.5}{1}} = 4 \times 10^{-3} \text{ m/day} = 4.62 \times 10^{-5} \text{ mm/sec}$$

Problem 7.17

A two-stage permeameter test is conducted to evaluate the vertical and horizontal hydraulic conductivity of a clay liner. In stage 1, a 0.1 m diameter borehole is drilled to a depth of 0.35 m. A 0.075 m inside diameter pipe is lowered to the bottom of the open borehole and sealed to the walls of the borehole. A 10 mm inside diameter graduated tube is placed on top of the 75 mm diameter pipe; then the pipe and the falling head permeameter fitted on top of it are saturated and the water seeps through the liner.

After reaching a steady state, the following measurements are recorded. At time equal 0, the water is 0.6 m above the ground surface. After 30 minutes of infiltration, the water has dropped to a height of 0.5 m above the ground surface. In stage 2, a 75 mm borehole is advanced 0.2 m below the bottom of the stage 1 borehole (0.55 m below surface). The falling head permeameter test is repeated and the water level falls from 0.6 m above the ground surface at time equal 0 to 0.5 m above the ground surface in 5 minutes. Calculate the vertical and horizontal hydraulic conductivity of the clay liner.

Solution 7.17

In the first stage, k_1 can be calculated using the following equation:

$$k_1 = \frac{\pi d^2}{11D(t_2 - t_1)} Ln\frac{h_1}{h_2} = \frac{\pi \times 0.01^2}{11 \times 0.075(30 - 0)} Ln\frac{0.6}{0.5} = 2.31 \times 10^{-6} \text{ m/min} = 3.33 \times 10^{-3} \text{ m/day}$$
$$k_1 = 3.85 \times 10^{-5} \text{ mm/sec}$$

In the second stage, k_2 can be calculated using the following equation:

$$k_2 = \frac{A}{B} Ln\frac{h_1}{h_2}$$

with

$$A = d^2 \ Ln\left(\frac{L}{D} + \sqrt{1 + \left(\frac{L}{D}\right)^2}\right) = 0.01^2 \ Ln\left(\frac{0.2}{0.075} + \sqrt{1 + \left(\frac{0.2}{0.075}\right)^2}\right) = 1.70 \times 10^{-4} \text{ m}^2$$

and

$$B = 8L(t_2 - t_1)\left(1 - 0.562e^{-1.57\frac{L}{D}}\right) = 8 \times 0.2 \times (5 - 0) \times \left(1 - 0.562e^{-1.57\frac{0.2}{0.075}}\right) = 7.93 \text{ m. min}$$

So

$$k_2 = \frac{A}{B} Ln\frac{h_1}{h_2} = \frac{1.70 \times 10^{-4}}{7.93} Ln\frac{0.6}{0.5} = 3.90 \times 10^{-6} \text{ m/min} = 5.61 \times 10^{-3} \text{ m/day} = 6.49 \times 10^{-5} \text{ mm/sec}$$

$$\frac{k_2}{k_1} = 1.70$$

Based on Figure 7.47, $m = \sqrt{\frac{k_h}{k_v}} = \sqrt{4.84} = 2.2$

$$k_h = m \times k_1 = 2.2 \times 3.33 \times 10^{-3} = 7.32 \times 10^{-3} \text{ m/day} = 8.47 \times 10^{-5} \text{ mm/sec}$$
$$k_v = \frac{k_1}{m} = \frac{3.33 \times 10^{-3}}{2.2} = 1.51 \times 10^{-3} \text{ m/day} = 1.74 \times 10^{-5} \text{ mm/sec}$$

Problem 7.18

Discuss the advantages and drawbacks of in situ tests versus laboratory tests.

Solution 7.18

The advantages and drawbacks of in situ tests versus laboratory tests are summarized in Table 7.3s.

Table 7.3s Advantages and Drawbacks of In Situ and Laboratory Tests

Laboratory Testing Advantages	Drawbacks	In Situ Testing Advantages	Drawbacks
Easier to analyze theoretically			Difficult to analyze theoretically
	Small-scale testing	Larger-scale testing	
Drainage can be controlled			Drainage difficult to control
	Time consuming	Relatively fast to perform	
Elementary parameters easier to obtain			Elementary parameters harder to obtain
	In situ stresses must be simulated	Testing under in situ stresses	
Soil identification possible			Soil identification rarely possible
	Some disturbance	Less disturbance for some tests	