Chapter 9

In situ testing

INTRODUCTION

The physical survey is that part of site investigation which aims to determine the physical properties of the ground. These are required:

1. to classify the soil into groups of materials which will exhibit broadly similar engineering behaviour; and
2. to determine parameters which are required for engineering design calculations.

Some soils, for example clays, may readily be sampled. If good-quality samples can be obtained, then laboratory testing offers the best method of determining soil and rock parameters under carefully controlled conditions. Sampling techniques have already been described in Chapters 6 and 7 and laboratory testing techniques in Chapter 8. But other types of ground are either difficult or impossible to sample and test successfully. In such cases, *in situ* tests should be used.

Information may be obtained *in situ* in at least three ways:

1. by using geophysical techniques; in particular, Chapter 4 showed how seismic techniques may be used to obtain valuable estimates of the stiffness of the ground;
2. by using *in situ* soil testing techniques, such as those described in this chapter; and
3. by making measurements using field instrumentation, such as is described in Chapter 10.

The following types of ground conditions are examples of those where *in situ* testing is either essential or desirable.

1. *Very soft or sensitive clays*. Good quality samples are hard to get. Traditionally, it was thought that piston sampling was required. More recent work has suggested that even greater care is required (for example, using the Laval sampler or the Sherbrooke sampler, Chapter 7). These samplers are relatively expensive, and are time-consuming to operate. Therefore *in situ* tests are often used to determine undrained parameters.
2. *Stoney soils*. With the possible exception of very stiff clay containing scattered gravel (for example, clayey tills) which can be sampled by careful rotary coring with either mud or polymer flush, stoney soils are almost impossible to sample, because the stones damage both the cutting shoe and the soil as a sampler is driven. Such materials may be tested *in situ* either using dynamic penetration testing, or geophysical techniques.
3. *Sands and gravels*. Sand sampling is possible (for example, using freezing techniques, or a piston sampler in a mud-filled borehole), but tends to be expensive, and to yield relatively highly disturbed samples (since even relatively minor strains imposed by sampling have the effect of destroying the soil's 'memory' of loading). Loose and uncemented gravels can also be sampled using large-diameter tubes, but suffer similarly from disturbance. Therefore *in situ* testing is commonly used in granular soils. Typically, testing is carried out using either field geophysics, or penetration testing. When accurate values of compressibility are required, then plate testing may be used.
4. *Weak, fissile or fractured rock*. The strength and compressibility of fractured rock is controlled by the discontinuities (for example, joints, fissures, faults) within it. Such materials

usually require rotary coring if they are to be sampled, but even when this can be carried out successfully it can only provide samples from which the intact (rather than the mass) properties may be determined. Therefore *in situ* testing provides the only satisfactory way to determine the engineering properties (particularly mass compressibility) of this type of material. Typical *in situ* testing techniques used in weak near-surface rocks include penetration testing, plate loading testing, field geophysical techniques, and pressuremeter testing.

In more complex projects it is common to duplicate the methods used to obtain key design parameters. For example, in recent investigations for major projects located in the London clay area it has been common to determine strength and stiffness using both field and laboratory techniques (for example, a combination of SPT, self- boring pressuremeter, field geophysics, and laboratory stress-path tests with local strain measurement).

For routine investigations, where cost and effectiveness will be major factors, the soil types on a site will have a large influence on the balance between sampling and *in situ* testing. The two activities must be carefully integrated during planning, to ensure that all the required parameters are obtained, and that they are of a quality relevant to the aims and objectives of the work.

In situ tests may be classified in a number of ways, including by cost, ease of use, method of interpretation, soil types in which they may be used, parameters which can be determined, etc. In this chapter we have considered only relatively common *in situ* tests, and divide them according to purpose, i.e. to obtain:

- penetration resistance;
- strength and/or compressibility, or
- *in situ* permeability.

On the other hand, a classification can be established on the basis of the degree to which tests can be analysed in a fundamental way to obtain real soil parameters, which is a function not only of how the test is applied to the soil, but also of the type of data collected. On this basis we can discern several groups of tests.

1. *Wholly empirical interpretation*. No fundamental analysis is possible. Stress paths, strain levels, drainage conditions and rate of loading are either uncontrolled or inappropriate. (Examples: SPT, CPT.)
2. *Semi-analytical interpretation*. Some relationships between parameters and measurements may be developed, but in reality interpretation is semi-empirical, either because both stress paths and strain levels vary widely within the mass of ground under test, or drainage is uncontrolled, or inappropriate shearing rates are used. (Examples: plate test, vane test.)
3. *Analytical interpretation*. Stress paths are controlled, and similar (although strain levels and drainage are not). (Example: self-boring pressuremeter.)

It should he noted that drainage conditions are virtually impossible to control during *in situ* testing. Tests carried out very slowly can be presumed to be drained, but may be relatively expensive because of the time taken to carry them out. Tests in clays are typically carried out rapidly, in an attempt to ensure that the soil remains undrained — this is difficult to ensure with any certainty, because the presence of thin layers of silt or sand within the test section will have a very great effect upon rates of pore pressure dissipation during testing. Tests carried out in clay soils are typically analysed to give undrained parameters (such as undrained shear strength, c_u), while in granular soils drained parameters (such as the peak effective angle of friction ‘) are determined.

When seen in terms of such practical factors as the range of soil types that may be tested, the simplicity of a test and its cost — factors which are most likely to be considered as important by practising engineers, penetration tests (and particularly dynamic penetration tests) are much more attractive than more sophisticated and analytically ‘correct’ tests, such as the self-boring pressuremeter. this then is the problem facing the engineer planning a site investigation: should cheap, rugged, simple tests be carried out, or should the risks associated with more sophisticated tests be taken, with the aim of obtaining more fundamental, and often more accurate, parameters? The answer must depend on the precision with which engineering calculations are to be made, and the types of soil to be expected, as detailed in Table 9.1.

Table 9.1 Parameters available from available *in situ* tests according to ground conditions

Test type	Parameters required							
	K_0	φ'	c_u	σ_c	E'/G	E_u	G_{max}	k
SPT		G	C	R	G	C	G	
CPT		G	C		G			
Marchetti dilatometer	G,C				G			
Borehole pressuremeter			C		G,R	C		
Plate loading test			C		G,R	C		
Field vane			C				G,C,R	
Seismic field geophysics								
Se1fboring pressuremeter	G,C	G	C		G,C			
Falling/rising head test								G
Constant head test								C
Packer test								R

G = granular, C = cohesive, R = rock.

PENETRATION TESTING

Many forms of *in situ* penetration test are in use worldwide. Designs and applications can be found in the proceedings of the European Symposia on Penetration Testing (1974, 1982), the International Symposium on Penetration Testing (ISOPT — 1988), the International Society of Soil Mechanics and Foundation Engineering (ISSMFE) *Report of the Sub-Committee on the Penetration Test for Europe* (1977), in two recent CIRIA reports (Meigh 1987; Clayton 1993) and in national standards produced, for example, by ASTM, BSI and DIN.

Penetrometers can be divided into two broad groups. The simplest are dynamic penetrometers. They consist of tubes or solid points driven by repeated blows of a drop weight. ‘Static’ penetrometers are more complex, being pushed hydraulically into the soil. The two most common penetration tests, which are used virtually worldwide, are the dynamic SPT, and the static CPT.

The Standard Penetration Test (SPT)

The ‘Standard Penetration Test’, commonly known as the ‘SPT’, is carried out in a borehole, by driving a standard ‘split spoon’ sampler (Fig. 9.1) using repeated blows of a 63.5kg (140 lb.) hammer falling through 762mm (30 in.). The hammer is operated at the top of the borehole, and is connected to the split spoon by rods. The split spoon is lowered to the bottom of the hole, and is then driven a distance of 450mm (18 in.), and the blows are counted, normally for each 76mm (3 in.) of penetration.

At the end of driving the split spoon is pulled from the base of the hole, and the sample is preserved in an airtight container. The penetration resistance (N) is the number of blows required to drive the split spoon for the last 300mm (1 ft) of penetration, The penetration resistance during the first 150 mm (6 in.) of penetration is ignored, because the soil is considered to have been disturbed by the action of boring the hole.

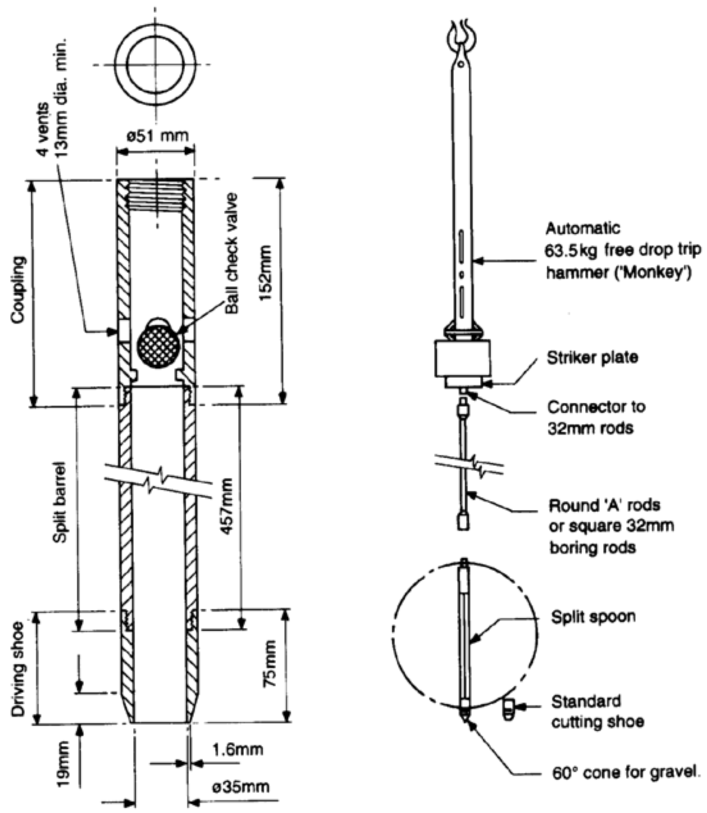

Fig. 9.1 Equipment for the standard penetration test.

The term 'Standard Penetration Test' was first coined by Terzaghi at the 1947 Texas Soil Mechanics Conference. In the USA, site investigation holes were traditionally made by wash boring and, in the 19th century, soil type was identified from the cuttings which were flushed to the top of the hole. In 1902 Colonel Charles R. Gow introduced a 1 in. diameter open-drive sampler, which was driven into the ground by repeated blows of a 110 lb. hammer (Fletcher 1965), and in subsequent years the American site investigation industry developed variations on this small-diameter tube sampler. By 1947, therefore, a number of different diameter tube samplers were in use (for example, see Hvorslev (1947)). Terzaghi recognized that by counting the blows necessary to drive a tube sampler, additional information on the consistency or density of the soil could be obtained, and at very little extra cost. What he was advocating was a procedure very similar to that currently used in the UK when taking a U100 undisturbed sample, namely the routine recording of penetration resistance.

When Terzaghi coined the term 'Standard Penetration Test' there was in reality no test, and no standard. However, he (and Peck, in their classic text *Soil Mechanics in Engineering Practice* first published a year later, in 1948) gave useful design correlations and charts which made the practical application of the data immediately obvious. The practice of determining penetration resistance therefore became widespread, both in the USA and elsewhere. In the UK, site investigation borings had traditionally been carried out using large-diameter (150—200mm dia.) well-boring equipment, coupled with 100mm dia. sampling, but even here the small-diameter open-drive sampler was rapidly adopted for the specific purpose of determining the penetration resistance of sands and gravels. Thus, while in the USA the SPT developed as an addition to tube sampling, whether in cohesive or granular soil, in the UK it was always regarded as an in situ test, and one primarily used in granular soils.

In a study of US sampling practice Mohr (1966) measured the masses of the hand- lifted hammers used drive tube samplers, and also the height to which they were lifted. At that time the practice was to drill with a crew of three, two of whom would lift the drive weight repeatedly by hand. He found that, typically, the mass of the hammer was around 140lb. (63.6kg) and that it was lifted by about 3Oin. (762mm). One of the most popular tube samplers, manufactured by Sprague and Henwood, had an outside diameter of 2 in. (52 mm) and an inside diameter of 1 in. (38mm). These, then, are the origins of the 'Standard Penetration Test'.

In the last decade, since the first edition of our book, there have been major efforts to unify SPT equipment and practice, on an international basis. In the early 1980s de Mello conceived of the idea of a series of International Reference Test Procedures (IRTP), which would be distinct from international standards in that they would provide an acceptable way in which international practices could be brought closer, rather than mandatory procedures (which some countries might be unable to adopt). The International Reference Test Procedure for the SPT was published by the ISSMFE in 1988 (see Decourt 1990). National standards are available in many countries, the most commonly followed being the British Standard (BS 1377: Part 9: 1990), the American standard (ASTM D1586 1984), and the Japanese Standard (JIS-A219 1976). CIRIA Funder Report CP/7 (Clayton 1993) gives the procedures and standards adopted around the world, as well as describing in detail the test, its strengths and weaknesses, and its uses for geotechnical design.

Correlations between SPT N value and soil or weak rock properties are wholly empirical, and depend upon an international database of information. Because the SPT is not completely standardized, these correlations cannot be considered particularly accurate in some cases, and it is therefore important that users of the SPT and the data it produces have a good appreciation of those factors controlling the test, which are:

1. variations in the test apparatus;
2. the disturbance created by boring the hole; and
3. the soil into which it is driven.

Effects of test apparatus

As can be seen in Fig. 9.1, the major components of test apparatus are the split spoon, the rods and the hammer.

Whilst split spoon design does vary to some extent, it is not thought to have a major effect on penetration resistance. The British Standard split spoon has recently been altered to bring it into line with the IRTP, by introducing a ball check valve in its head. (But note that the vents must be maintained clean and free of soil, and that BS 1377 contains a dimension error which has been corrected in Fig. 9.1). In the USA it is sometimes the practice to use a split spoon which has liners, for ease of sample storage. Seed *et al*. (1985) found that drillers sometimes omit this liner, because sample recovery is then improved, and that the omission of liners led to a 15% decrease in N. A major uncertainty at the time of writing results from the use, particularly in the UK and when gravel or stoney soils are encountered, of the solid o cone in the place of the standard open cutting shoe. There is certainly evidence (Clayton 1993) to suggest that the use of the solid cone may, in certain instances (in sands and in the chalk), have approximately doubled the penetration resistance. It is therefore recommended that, as far as possible, its use is avoided.

Rods and hammer characteristics affect penetration because, in a given soil, N is inversely proportional to the energy delivered to the split spoon (Palmer and Stuart 1957; Schmertmann and Palacios 1979). Thus if two different hammer/rod systems deliver different energies, two different penetration resistances will be recorded, where

$$\frac{N_1}{N_2} = \frac{E_2}{E_1} \tag{9.1}$$

The energy delivered to the SPT split spoon is theoretically the free-fall energy of a 63.6 kg mass falling through 762 mm, i.e. 473.43. In practice, however, it has repeatedly been shown that up to 65% of this free-fall energy may be lost (Kovacs *et al.* 1977; Seed *et al*. 1985; Riggs 1986; Skempton 1986; Clayton 1990; Decourt 1990). This may occur as a result of:

1. inertial energy absorbed by over-heavy rods, and the weight of the SPT hammer's anvil;
2. energy spent in heat and noise when the SPT weight impacts with the anvil;
3. bending energy, when rods which are bent, or rods of too small a second moment of area are used;
4. input energy reduction due to the hammer not being lifted for the full 762mm; and
5. energy losses due to friction between the various hammer components, or between lifting ropes, sheaves and catheads on the drilling rig.

Because energy losses may be significant, it is important both to comply with standards relating to the rods, and to have a reasonable idea of the energy delivered by the SPT hammer in use. For this reason BS 1377 specifies that 'the rods used for driving the sampler shall be made of steel of a quality and have a stiffness equal to or greater than type AW drill rods complying with BS 4019', and that for holes deeper than 20 m 'rods with a stiffness equal to or greater than BW drill rods .. . shall be used'. Traditionally, in the UK, solid 1in. square section rods have been used for SPT testing at shallow depths, with 1in. square rods being used at depth. BS 1377 also states that rods heavier than 10.0kg/rn shall not be used, and that rods and rod coupling shall not be bent — 'when measured over the whole length of each rod the relative deflection shall not be greater than 1 in 1000' (i.e. 3mm for a 3 m-long rod). The stiffnesses of these, and the rods recommended in the IRTP, are given in Table 9.2.

Table 9.2 Stiffnesses and weights of various SPT rods

Reference	Rod type	Rod diameter (mm)	Section modulus Ze ($m^3 \times 10^6$)	Rod weight (kg/m)
IRTP	—	40.5	4.28	4.33
	—	50.0	8.59	7.23
	—	60.0	12.95	10.03
BS 1377	AW	43.6	5.10	4.57
	BW	54.0	8.34	7.86
—	Solid square	31.8 ($1^{1/4}$ in.)	5.33	7.89
		38.1 ($1^{1/2}$ in.)	9.22	11.37

Table 9.2 suggests that traditional square boring rods may be used at depths of less than 20m, but not deeper where self-weight may be a problem. The experience of using both square rods and AW rods suggests that AW rods are much to be preferred. Square rods are relatively heavy, and have a less satisfactory coupling system than round rods. They appeared more easily bent during handling, and were observed to bend under self-weight when used in very deep holes. But in general, provided that rods are in good condition, are straight, and have straight couplings, the use of either round (AW or BW) rods or square rods should not lead to significant differences in penetration resistance.

The many different types of SPT hammer in use around the world may conveniently be divided into the following categories.

1. *Automatic trip hammers*. Automatic trip hammers (Fig. 9.1), which are standard in the UK, are also used in Israel, Australia and Japan. This is the best type of hammer, because the energy delivered per blow is consistent. Clayton (1990) reports tests on a Dando automatic trip hammer which gave an average energy of 73% of the free-fall energy, with a standard deviation of only 2.8%.

2. *Hand-controlled trip hammers*. Hand-controlled trip hammers are not widely used. An example is given in Ireland, Moretto and Vargas (1970). The weight is lifted by hoist to what is judged to be the correct height, and then tripped to give a free fall. Inconsistencies can occur if the driller is careless in assessing how high to lift the weight.
3. *Slip-rope hammers.* Slip-rope hammers are widely used over much of the world, including the USA, Japan, and South America. Common types of slip-rope hammer are shown in Fig. 9.2. The weight is lifted by a rope which passes over a sheave on the top of the mast of the drilling rig, and is pulled via a cathead. To deliver consistent energy the operator not only has to lift the weight repeatedly to the correct height, but also has to release it from the cathead in a consistent manner. This is extremely difficult. Energy is lost in friction as the rope slips over the rotating cathead, and also as the sheave is turned. The amount of energy lost on the cathead depends upon its condition, and how many turns of rope the operator uses.

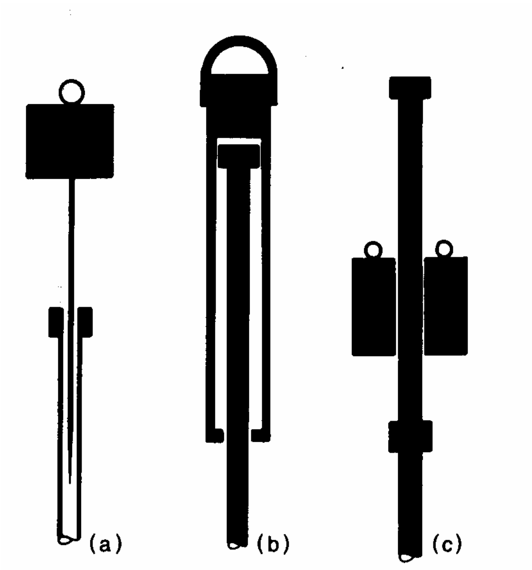

Fig. 9.2 Sections through American SPT slip-rope hammers: (a) pinweight hammer; (b) safety hammer; (c) donut hammer (after Riggs 1986).

4. *Hand-lifted hammers.* Hand-lifted hammers are almost identical today to those used in the USA in the 1920s and 1930s, which were described above. They are not widely used, except in relatively undeveloped countries. Here the weight is lifted and dropped by hand, and energy is lost in the sheave bearing.

A full discussion of SPT energy measurement is given by Clayton (1990). ASTM D4633—86 is the American Standard for SPT hammer energy measurement. Basically, the procedure consists of placing a load cell in the rod string at a distance of more than 10 rod diameters below the underside of the hammer, with the rods and split spoon in a hole which is as deep as possible, and preferably more than 12 m deep. A fast data acquisition unit is used to capture the force — time relationship as the energy from each blow of the hammer passes down the rods. The energy transferred is obtained from the following integration:

$$E = \frac{c}{AE}\int_0^{t''} F(t)^2\,dt \qquad (9.2)$$

where c = propagation velocity of the stress wave in the rods (normally approximately 5.1 m/ms), A = cross-sectional area of the rod, E= Young's modulus of the rod, F(t) = force measured in the rod at time t, and t' is the time taken for the stress wave to travel to the base of the rods and be reflected back to the load cell.

If the length of rods between the load cell and the base of the rods is L, then

$$t' = \frac{2L}{c} \tag{9.3}$$

and if L is equal to or greater than 15 m, the error in determining the rod energy will be less than 2%.

The Oyo Geologger 3030 SPT measuring module provides a convenient and easy-to-use method of determining the energy delivered by a given hammer system. The energy delivered by each blow of a single hammer should be measured over a large number of blows (more than 30), and for each blow the rod energy ratio (ER_r) — the ratio between delivered energy and the free-fall energy, should be reported. For each hammer the range, mean and standard deviation of rod energy ratio should be calculated and reported. On critical jobs, energy measurements should be carried out for all hammer/rig/operator combinations.

Measurements of energy so far carried out have shown that hammer systems vary both in the consistency with which blows are delivered, and also the average energy delivered. With some hammer types there will be considerable variations of energy not only depending upon the hammer, but also on the operator and the rig. The slip-rope hammer is a particularly poor tool. For an American slip-rope hammer Kovacs *et al.* (1977) measured blow-by-blow rod energy ratios which varied from as little as 35% to as much as 69% which, if reflected in the average rod energy ratio, would imply a 100% variation in penetration resistance. International measurements are shown in Table 9.3.

Table 9.3 Measured SPT rod-energy ratios

Country	Hammer type	Release mechanism	Average rod energy ratio (%)	Source references*
Argentina	Donut	Cathead	45	1
Brazil	Pin weight	Hand dropped	72	3
China	Automatic donut	Hand trip	60	1
	Donut	Dropped	55	2
	Donut	Cathead	50	1
Colombia	Donut	Cathead	50	3
Japan	Donut	Tombi	78-85	1,4
	Donut	Cathead, 2 turns + special release	65-67	1,2
UK	Automatic	Trip	73	5
USA	Safety	Cathead, 2 turns	55-60	1,2
	Donut	Cathead, 2 turns	45	1
Venezuela	Donut	Cathead	43	3

* (1) Seed *et al.* (1985); (2) Skempton (1986); (3) Decourt (1986), (4) Riggs (1986), (5) Clayton (1990).

On the basis of this type of measurement, it has become clear that SPT N values should be converted, where possible, to an equivalent standard penetration resistance (N) equivalent to a delivered energy of 60%, using the equation

$$N_{60} = N_{measured} \times \frac{E_{measured}}{E_{60}} \tag{9.4}$$

where E_{60} = 60% of the free-fall hammer energy (0.6 x 473.4J = 284.0 J), N_{60} = penetration resistance corrected to 60% rod energy ratio, $N_{measured}$ = measured penetration resistance, and $E_{measured}$ is the measured rod energy.

From the figures in Table 9.2 it can be estimated that corrections of the order of +40 to —30% may need to be applied to N values. This will certainly be necessary when the results are to be used in a precise way, for example in assessing the liquefaction potential of sands.

Effects of borehole disturbance

The effects of borehole disturbance on the SPT can be severe, leading to reductions in penetration resistance as high as 70—80%. But the actual amount and effects of disturbance vary considerable with soil type, and as a result of the method of drilling and casing the hole, and its diameter.

The maximum depth to which disturbance affects the soil below the base of a borehole is, in broad terms, a function of its diameter. Most evidence suggests that disturbance can be significant down to three borehole diameters below the base of the hole. The diameter of hole used for SPT testing may vary considerably, from wash- bored 60mm dia. holes, through typical 200mm dia. British light-percussion boreholes, to pile holes of more than 1 m dia. In the original, wash-bored, boreholes the maximum depth of disturbance was not likely to exceed 180 mm, and the borehole was maintained full of fluid by virtue of the drilling method. In British conditions the depth of disturbance can certainly be greater than the entire depth of the test.

Granular soils are the most severely affected. Conventional wisdom suggests that only fine-grained or silty sands are prone to disturbance, and that this disturbance results from boiling into the base of the borehole, because the hole has not been kept full of fluid, and soil has 'boiled' into its base. It is certainly true that fine-grained and silty sands are at risk, if uncemented, but this is also true for all uncemented granular soils, including coarse alluvial sands and gravels (Connor, 1980). It is now also clear that it is virtually impossible to prevent boiling occurring in this type of soil if the drilling process uses casing which extends to the bottom of the borehole, and if tools are withdrawn from the hole without the water level being constantly recharged. Thus the relatively large light-percussion or 'shell and auger' boring used in the UK cannot give good results, even when the most exacting specifications and the highest levels of supervision are applied. The best results are obtained by using small-diameter rotary or wash-bored holes, with mud flush, where drilling tools are withdrawn slowly from the borehole, and with the casing kept a minimum of 1 m above the base of the hole where possible. Unfortunately such drilling techniques are only suitable in sands. The data of Connor (1980) and Mallard (1983) suggest that N may be reduced to 1/5th of its correct value by aggressive drilling or unsuitable technique in sands and gravels.

N values in chalk are also significantly affected by drilling technique. Although evidence is limited, and the mechanisms are unclear, it would appear that penetration resistance may be halved by drilling disturbance. In other weak rocks it is also likely that borehole disturbance may be significant, since the test is often terminated at 100 blows, and penetration falls short of the full 450mm.

In clays, and particularly in overconsolidated clays, there is little evidence to suggest that disturbance is a problem. The SPT has been widely and successfully used in the UK in these types of ground. And there is evidence that it can be used more economically and more reliably than the conventional combination of sampling and triaxial testing normally used in the UK (for example, Stroud (1974)).

Interpretation and use

We have already noted that the measured penetration resistance, N, should be corrected for hammer energy, to give the standard value of N. In addition since the SPT brings the soil to failure, and because the strength of granular soil will be strongly dependent on effective stress level, it will be necessary to correct 'N' values from sands and from gravels to a standard overburden pressure level when the test is used to determine relative density. Where penetration resistance is corrected, the reference vertical stress level is 1 kg/cm2, or 100 kPa. The penetration resistance corrected both for rod energy and for overburden pressure is termed (N1)60. Skempton (1986) suggests that for relatively recently deposited normally consolidated sand it may be reasonable to assume

$$\frac{(N_1)_{60}}{D_r^2} = 60 \tag{9.5}$$

where D_r = relative density of the sand. A suitable overburden correction chart is given by Liao and Whitman (1985).

For many other applications the use of an overburden correction may not be necessary, because the increase in strength and stiffness caused by effective stress increase is arguably reflected in an increased penetration resistance. In using the SPT in design it is important to look in detail at the origins of methods of calculation, to see how they were derived, before deciding which corrections are appropriate.

Many direct methods of calculating foundation settlements have been based upon SPT penetration resistance, but systematic research (Bratchell *et al.* 1975; Simons and Menzies 1977; Talbot 1981; Milititsky *et al.* 1982; Clayton *et al.* 1988) suggests that most are inaccurate. We recommend the methods by Schultze and Sherif (1973) and Burland and Burbidge (1982) because comparative calculations have shown them to be of higher accuracy than others. Piling design methods have been considered by Poulos (1989).

Stroud (1989) gives an extremely useful guide to the way in which soil and weak rock parameters can be obtained using the results of the SPT, and the reader is referred to this for further details. The references shown in Table 9.4 are also of value. Figure 9.3 shows the extremely good correlation obtained for the overconsolidated clays in the UK between N and c_u.

Table 9.4 Recommended correlations between SPT penetration resistance and soil and weak rock parameters

Parameter and ground conditions	Reference
Effective angle of friction of sand	Peck *et al.* (1974) Mitchell *et al.* (1978)
Stiffness of sand	Stroud (1989)
Gmax of sand	Crespellani and Vannucchi (1991)
Undrained shear strength of clay	Stroud (1974)
Coefficient of compressibility (my)	Stroud and Butler (1975)
Drained Young's modulus of clay	Stroud (1989)
Unconfined compressive strength of weak rock	Stroud (1989)
Mass compressibility of fractured chalk	Stroud (1989)

The cone penetration test (CPT)

The 'Cone Penetration Test', normally referred to as the 'CPT', is carried out in its simplest form by hydraulically pushing a 60° cone, with a face area of 10cm^2 (35.7mm dia.), into the ground at a constant speed (2 ± 0.5 cm/s) whilst measuring the force necessary to do so. Most commonly, however, a friction cone is used. The shear force on a 150 cm^2 'friction sleeve', with the same outer diameter as the cone and located immediately above the cone, is then also measured. Both electrical and mechanical means of measuring cone resistance and side friction are currently used, with the shape of the cone differing considerably according to the method in use. The cone is driven from ground surface, without making a borehole, using a special mobile hydraulic penetrometer rig.

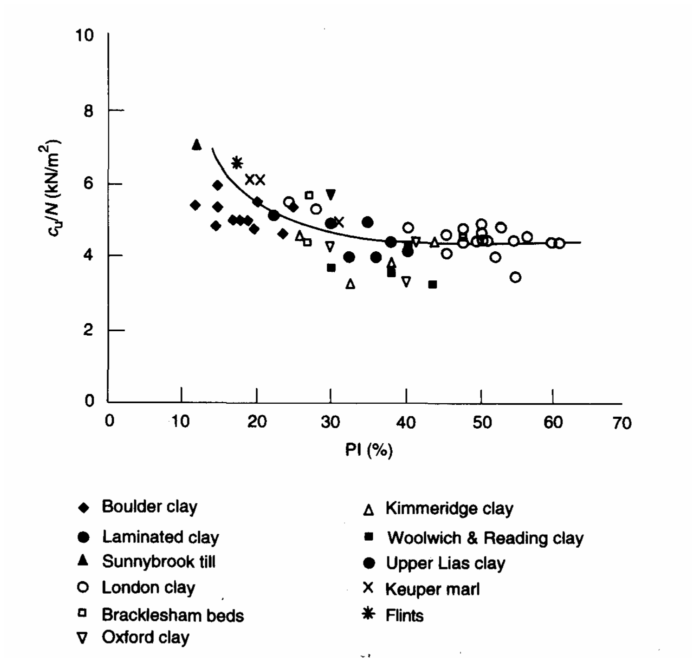

Fig. 9.3 Ratio of undrained shear strength (c_u) determined on 100mm diameter specimens to SPT N, as a function of plasticity (Stroud 1974).

The CPT was developed in Holland in 1934, and was originally used as a means of locating and evaluating the density of sand layers within the soft deltaic clays of that country, for driven pile design. The original cone, and the mechanical Delft cone described by Vermeiden (1948) are shown in Fig. 9.4. The latter, which was developed by the Delft Laboratory for Soil Mechanics, is in widespread use in Holland and in many other parts of the world. Its development overcame the major problem of the original cone, where soil particles could become lodged between the cone and the bottom face of the rods. The value of the Delft cone was increased very significantly by Begemann, who introduced the mechanical friction cone (Fig. 9.5) above the Delft mantle (see Begemann (1965)). The electric cone (Fig. 9.6), where measurements are made using strain gauges or transducers located immediately above the cone, was first developed in 1948, but only came into widespread use in the late 1960s. Measurement of the pore pressures developed at the cone end during penetration first took place in the late 1960s and early 1970s. Other developments and enhancements of the cone have also taken place, and continue to this day.

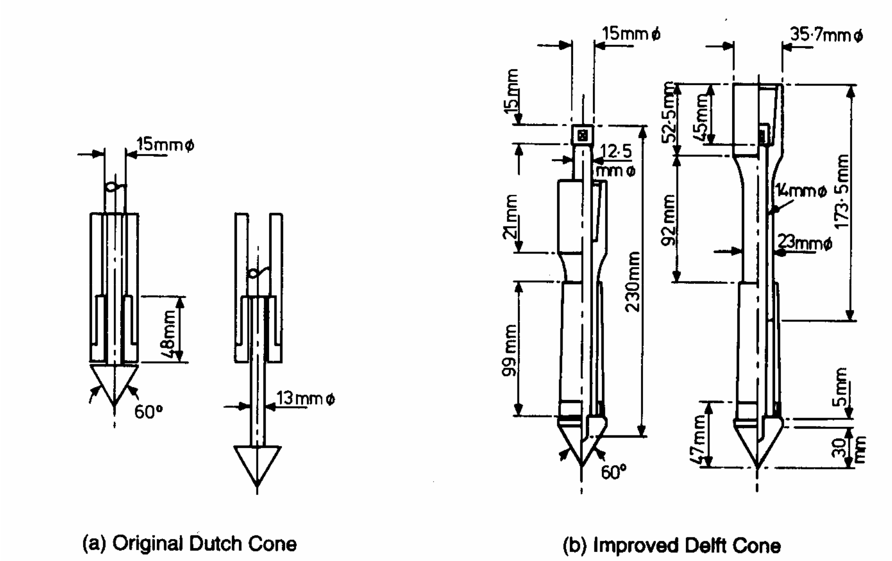

Fig. 9.4 Original dutch cone and improved mechanical Delft cone (Lousberg *et al.* 1974).

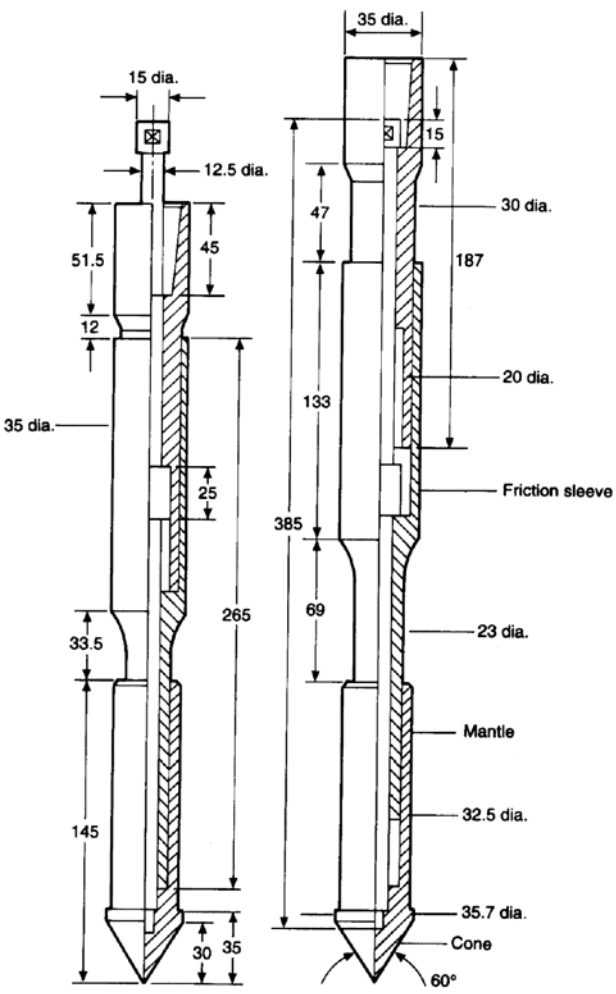

Fig. 9.5 Begemann mechanical friction cone (left, fully closed; right, fully extended) (Meigh 1987)

Mechanical cone testing

The method of advancing a mechanical cone is considerably more complex than for an electric cone, because force measurement must be made whilst the components are moving, in order to minimize friction. For the simpler Delft cone the procedure is as follows.

1. Advance cone end by 8cm, by pushing down (at the ground surface) on a string of solid 15 mm diameter rods which extend inside the outer hollow rods from the cone to the ground surface.
2. Whilst the cone is moving at the standard rate, measure cone resistance at the ground surface, either using a hydraulic load cell connected to a pressure gauge, or with an electrical transducer, positioned at the top of the rod string, at ground surface.
3. After recording the cone resistance by advancing the cone, push the outer rods downwards by 20 cm. During the last 12cm of this part of the drive the cone and rods should move together.
4. Repeat the entire process, to give intermittent force measurements at 20cm depth intervals.
5. Every metre add new inner and outer rods.

When the mechanical friction cone is used the procedure becomes more complex and the procedure is as follows.

1. Advance the inner rods, and the cone end, by 4cm.
2. Measure cone resistance whilst the cone rods are moving.
3. Continue to advance the inner rods, engaging the friction sleeve.

4. Measure the total force resulting from the sum of the cone resistance and side friction.
5. Obtain the force on the friction sleeve by subtracting the first measured force from the second.
6. In the final stage the outer rods are pushed down by 20 cm, taking the friction sleeve with them for the last 16 cm, and the cone for the last 12 cm.
7. Repeat the procedure, to give a measurement of cone resistance and of side friction every 20 cm.
8. Every metre add new inner and outer rods.

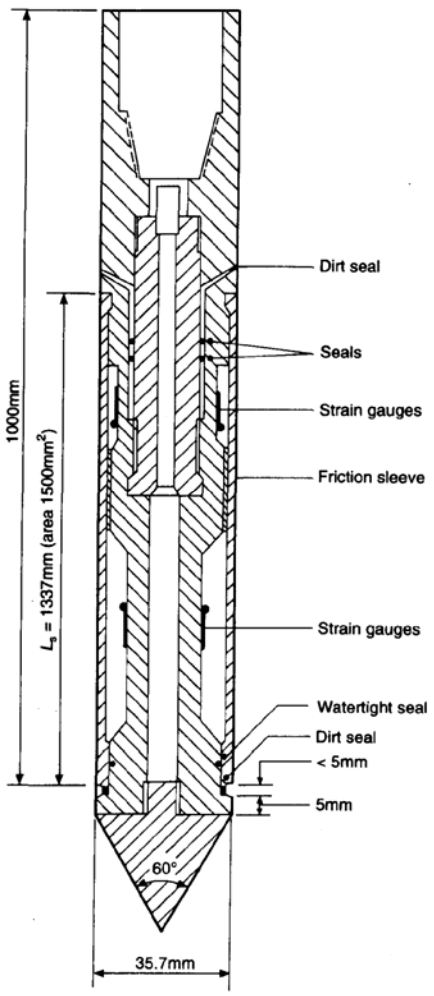

Fig. 9.6 Electric friction cone (largely after Meigh 1987).

Because the Delft and Begemann friction cones are mechanical, they are rugged, simple to use and to maintain. They can give reliable results provided the equipment is properly maintained, and the testing carried out with care. Against this, however, they have a system of measurement which can lead to serious errors, some of which have been described by Begemann (1969) and de Ruiter (1971). Because friction develops between the inner rods and the inside wall of the outer rods the cone resistance should always be measured whilst the inner rods are moving relative to the outer rods, in order to keep this friction to a minimum. Pushing the inner and outer rods at the same time as measuring cone resistance will result in large irregularvariations in rod friction, and noticeable decreases in the measured cone resistance after the penetration is stopped to allow the addition of rods.

At high cone resistances, loads as high as 10 tonne may be need to be applied to the cone. At 30m depth the compression of the inner rods may be of the same order as the 8cm stroke used in a Delft cone and, although the top of the inner rods is pushed downwards by the correct amount, the cone will not then advance ahead of the outer rods. This effect will obviously be more serious when a Begemann cone is in use, because the available stroke is only 4cm. In addition, in deep soft soils, corrections should be made to mechanical cone data to compensate for the mass of the rods.

Electric cone testing

Electric cones are more expensive, both in terms of cone manufacture and data logging and recording. They have the advantages, however, of being simpler to use, of measuring forces close to their point of application (and therefore without the frictional and rod-shortening effects described above), and of providing almost continuous data with respect to soil depth. Figure 9.6 shows a schematic diagram of an electric cone. Cone resistance is measured as standard, and side friction measurement is also extremely common. In addition, the following measurements may be available, depending upon the cone manufacturer:

1. cone inclination, to check that the cone is not drifting out of vertical;
2. pore pressure (in the 'piezocone');
3. soil resistivity (used, for example, in pollution studies);
4. ground vibration, using three-component geophones (in the 'seismic cone');
5. gamma-ray backscatter (for density determination);
6. pressuremeter values (see later); and
7. sound (the 'acoustic' penetrometer).

Meigh (1987) lists the advantages of the electric penetrometer as including:

1. improved accuracy and repeatability of results, particularly in weak soils;
2. better delineation of thin strata (because readings can be taken more frequently);
3. faster over-all speed of operation;
4. the possibility of extending the range of sensors in or above the tip (see above); and
5. more manageable data handling.

The speed and convenience with which the electric cone may be used has led to its widespread adoption in many countries, although mechanical cones are still common. It will be seen by comparing Figs 9.5 and 9.6 that mechanical and electric friction cones have significantly different geometries, and this has important implications for the interpretation of cone data, as will be seen below. Electric cone data can be processed more-or-less as penetration is carried out, to produce not only plots of cone resistance and sleeve friction, but also to provide estimates of soil type and soil parameters. This gives the engineer the opportunity to make decisions regarding both the design of a ground investigation and the design of the civil engineering works even while testing is proceeding.

The piezocone

The measurement of pore water pressure during cone testing is not as common as the measurement of cone resistance and side friction, but the last five or so years have seen a major increase in awareness of the tremendous potential of this tool, especially when testing in soft, primarily cohesive, deposits. A porous element is included in the apparatus, with an electronic pore pressure transducer mounted in a cavity behind it.

As shown in Fig. 9.7 there are three popular positions for this porous element. The major applications of the piezocone are as follows.

1. *Profiling*. The inclusion of a thin pore-pressure-measuring element allows the presence of thin granular layers to be detected within soft cohesive deposits. Such layers are of great importance to the rate of consolidation of a soft clay deposit.
2. *Identification of soil type*. The ratio between excess pore pressure and net cone resistance (see below) provides a useful (although soil-type specific) guide to soil type.

3. *Determining static pore pressure*. Measurements of the static pore pressure can be made in granular soils (where dissipation is rapid), and estimates can be made in clay, either when the cone is stopped to add rods, or by deliberately waiting for full dissipation of the excess pore pressures set up by penetration.
4. *Determination of in situ consolidation characteristics*. In clays, the horizontal coefficient of consolidation, c, can be determined by stopping the cone, and measuring pore pressure dissipation as a function of time (Torstensson 1977, 1982; Acar *et al.* 1982; Tavenas *et al.* 1982).

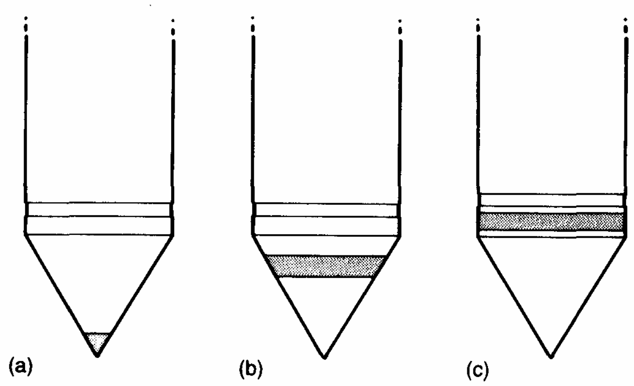

Fig. 9.7 Positions of porous tips on piezocones.

The seismic cone

Seismic cones contain either one or two three-component geophone arrays, mounted internally, some distance behind the friction sleeve. Where two array are used, the vertical distance between the arrays will be of the order of 1 m, or more. The use of the seismic cone has been discussed in Chapter 4. In recent years it has proved a valuable tool for determining the benchmark value of very small strain stiffness (G_0), by means of either parallel cross-hole testing or, more normally (because it is considerably more economical) down-hole testing.

Standards and reference test procedure

In the last decade, since the writing the first edition of this book, there have been important developments in the development and standardization of the CPT. Current standards include BS 1377: Part 9:1990 and ASTM D3441 (1986). An International Reference Test Procedure (IRTP) can be found in the First International Symposium on Penetration Testing (ISOPT1 — ISSMFE 1988), and an excellent review of the cone test is given by Meigh (1987). During use the cone end will be worn down, and regular checks should therefore be made to ensure that it continues to comply with the standard dimensions and tolerances given in the codes. The surface roughness of the cone and the friction sleeve significantly affects cone resistance and should be maintained at a prescribed value (Meigh (1987) recommends a roughness of 0.5 m ± 50%). Calibration of measurement systems should be carried out regularly, and zero load measurements taken before and after each test. De Ruiter (1982) suggests that zero drift should not exceed 1—2% of the rated maximum load, and Meigh notes that the aim should be for errors in cone resistance and side friction not to exceed 3% of range.

Interpretation and use

The basic measurements made by a cone are:

1. the axial force necessary to drive the 10 cm^2 cone into the ground at constant velocity; and
2. the axial force generated by adhesion or friction acting over the 150 cm^2 area of the friction jacket.

For piezocones, the basic measurement is the pore pressure developed as penetration proceeds.

Routine calculations convert these measurements into cone resistance, local side friction and friction ratio.

Cone resistance, q_c (normally in MPa) can be calculated from:

$$q_c = \frac{F_c}{A_c} \tag{9.6}$$

where F_c = force required to push the cone into the ground, and A_c plan area of the cone, i.e. $10cm^2$.

Local side friction, f_s (normally in MPa), can be calculated from:

$$f_s = \frac{F_s}{A_s} \tag{9.7}$$

where F_s shear force on the friction sleeve, and A_s = area of the friction sleeve, i.e. 150 cm^2.

Friction ratio, R_f (in %), can be calculated from:

$$R_f = \frac{f_s}{q_c} \tag{9.8}$$

Because of the geometry of the electric cone, where pore water pressure acts downwards on the back of the cone end (Fig. 9.8), the cone resistance will be under- recorded. When used in deep water, for example, for offshore investigations, the force exerted by groundwater will be significant, and if pore pressures are measured (with the piezocone), cone resistance can be corrected for this effect. The corrected, 'total', cone resistance, q_t is:

$$q_t = q_c + (1-\alpha)u \tag{9.9}$$

where α = ratio of the area of the shaft above the cone end to the area of the cone (10 cm^2), typically 0.15 to 0.3, and u=pore pressure at the top of the cone.

Because the pore pressure is not always measured at the top of the cone, but is sometimes measured either on the face, or on the shoulder (Fig. 9.7), a factor must be applied to the measured pore pressure. This factor (β) is based upon pore pressure distributions calculated using the strain path method. Thus:

$$q_t = q_c + (1-\alpha)(u_0 + \beta\Delta u) \tag{9.10}$$

where β = ratio between the calculated excess pore pressure at the top of the cone and at the point of measurement, u_0 = hydrostatic pore pressure, and Δu = excess pore pressure caused by cone penetration. Pore pressure distributions measured and calculated around piezocones are shown in Fig. 9.9.

In soft cohesive soils, at depth, much of the cone resistance may be derived from the effect of overburden, rather than the strength of the soil. In these circumstances the 'net cone resistance' may be calculated:

$$q_n = q_c - \sigma_v \tag{9.11}$$

where q_n = net cone resistance, and σ_v = vertical total stress at the level at which q_n is measured. Net cone resistance can only be calculated once the distribution of bulk unit weight with depth is known, or can be estimated.

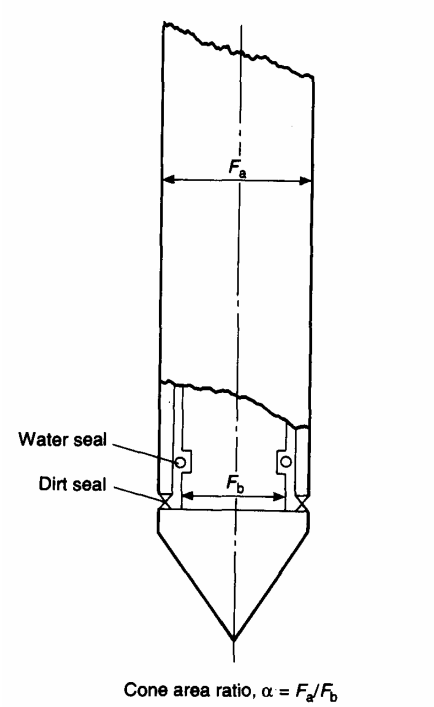

Fig. 9.8 Definition of cone area ratio, α.

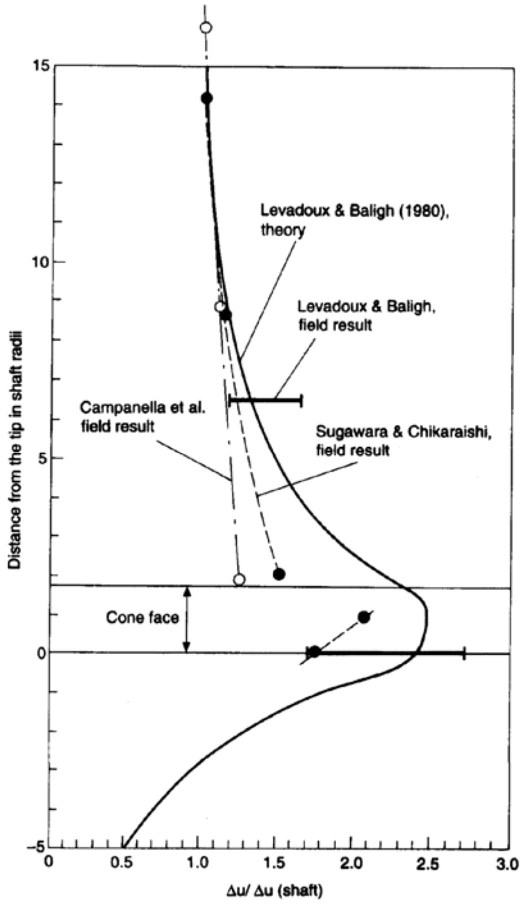

Fig. 9.9 Distribution of excess pore pressure over the cone (Coutts 1986).

Typical results of a friction cone test are given in Fig. 9.10. The original development of side friction measurement was made by Begemann using a mechanical cone, as shown in Fig. 9.5, who found the useful correlation between friction ratio and soil type shown in Fig. 9. 11a. He defined soil type by its percentage of particles finer than 16 tm, and found that on a plot of side friction versus cone resistance each type of soil plotted as a straight line passing through the origin. This has led to more sophisticated charts such as that shown in Fig. 9.11b, and for the piezocone to correlations based upon the relationship between excess pore pressure and net cone resistance ($q_n = q_c - \sigma_v$).

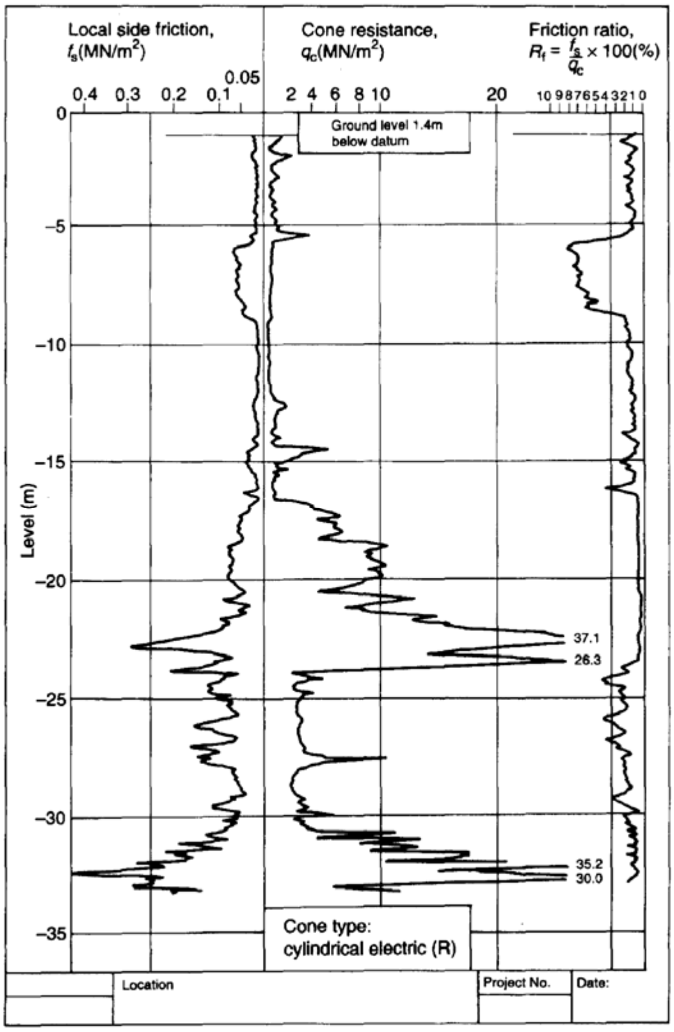

Fig. 9.10 Typical record of a friction cone penetration test (te Kamp, 1977, from Meigh, 1987).

The classification of soils is normally carried out on the basis of the value of cone resistance in combination with the friction ratio. Generally, the diagnostic features of the common soil types are as given in Table 9.5.

As with the SPT, the CPT provides important data in cohesionless soils, because of our inability to obtain good-quality, undisturbed samples for laboratory testing. Empirical correlations are widely used to obtain estimates of relative density, effective angle of shearing resistance (φ'), and stiffness. It should be borne in mind that empirical correlations are soil-type dependent, and therefore are of limited accuracy. Useful relationships between angle of shearing resistance and cone resistance, q_c, can be found in Schmertmann (1978), and Durgunoglu and Mitchell (1975). A correlation between q_c and SPT N, based on particle size, is shown in Fig. 9.12.

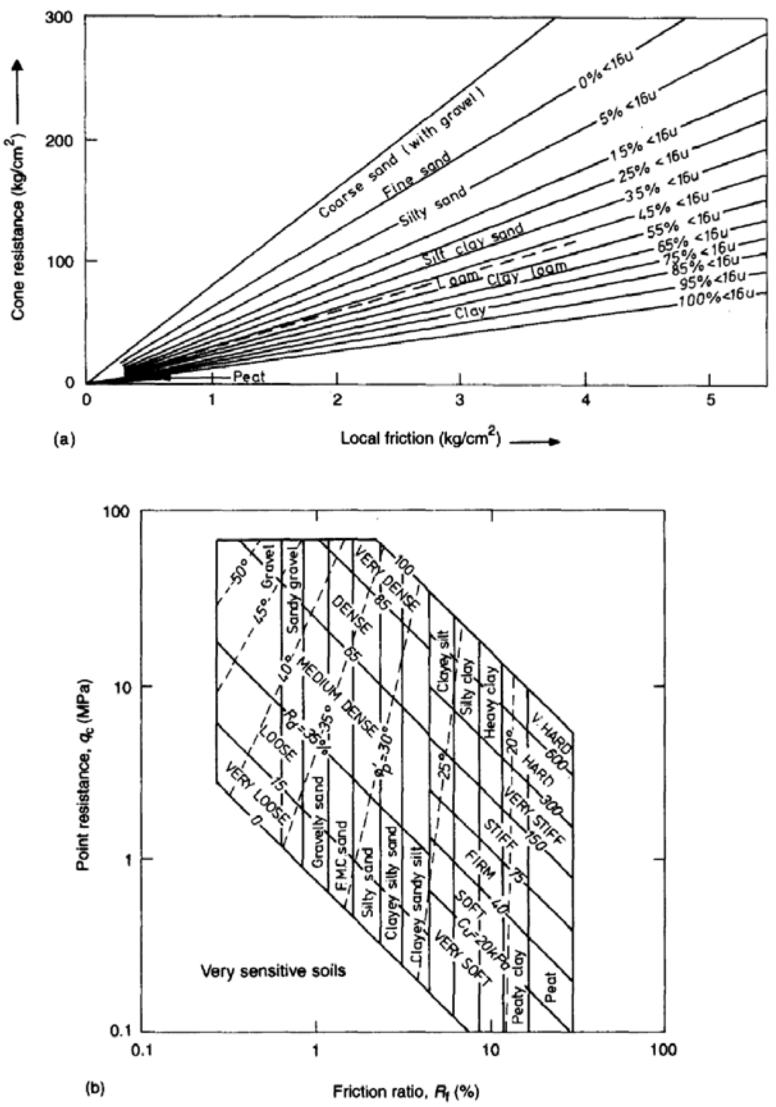

Fig. 9.11 (a) relationship between soil type, cone resistance and local friction (Begemann 1956); (b) soil identification chart for a mechanical friction cone (Searle 1979).

Table 9.5 Diagnostic features of soil type

Soil type	Cone resistance	Friction ratio	Excess pore pressure
Organic soil	Low	Very high	Low
Normally consolidated clay	Low	High	High
Sand	High	Low	Zero
Gravel	Very high	Low	Zero

Stiffness is often expressed in terms of the constrained modulus, M (where M is the stiffness in the vertical direction when lateral strain is prevented), in the form.

$$M = \alpha_M q_c \qquad (9.12)$$

For normally consolidated sands, M typically lies in the range 3—11, whilst for overconsolidated sands, values are somewhat higher. For a discussion of available data see Meigh (1987). Well-known methods of predicting the settlement of shallow footings (de Beer and Martens 1957; Schmertmann 1970; Schmertmann *et al.* 1978) use cone resistance directly. For example, Schmertmann *et al.* (1978)

use E = 2.5 q_c. Such relationships, although of great practical value, are known to be of limited accuracy. This is to be expected, because the CPT test involves the continual failure of soil around the cone, and cone resistance is a measure of the strength of the soil, rather than its compressibility. It has been shown (Lambrechts and Leonards 1978) that while the compressibility of granular soil is very significantly affected by over- consolidation, strength is not. This shortcoming is shared by the SPT. However, as we showed in the first edition of this book, settlements of spread footings predicted using the CPT tend to be considerably more accurate than those using the SPT, because there is no borehole disturbance. In a comparative study based upon case records, Dikran (personal communication) found that the ratio of calculated/observed settlements fell in the range 0.21—2.72, for four traditional methods of calculation using the CPT. For the SPT the variation was 0.15—10.8.

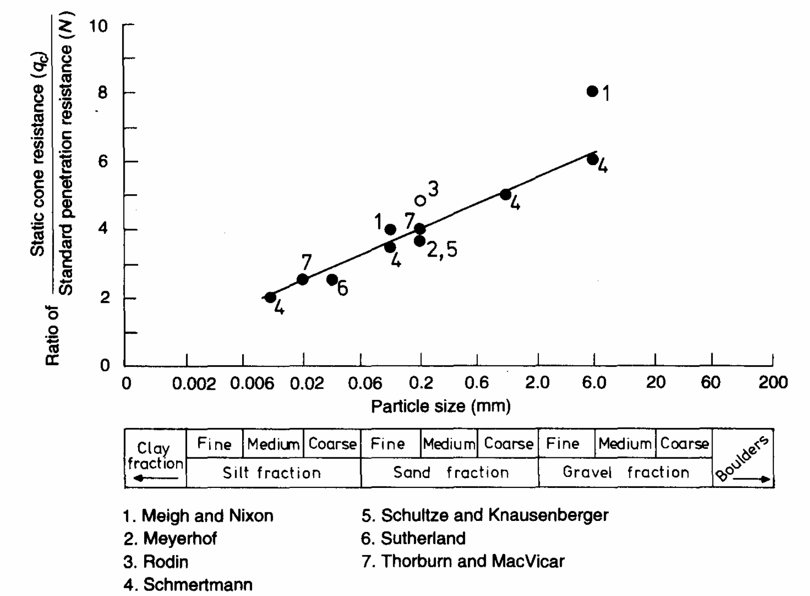

Fig. 9.12 Ratio of (CPT q_c) (SPT N) as a function of D50 particle size of the soil (Thorburn, 1971).

When calculating the point resistance of piles in sand based upon cone resistance, it is normal to consider the static cone penetrometer as a model of the pile, and simply apply a reduction factor of between two and six to give allowable bearing pressure (Van der Veen and Boersma 1957; Sanglerat 1972). Sand deposits are rarely uniform, and so an averaging procedure is used with the q_c values immediately above and below the proposed pile tip position (Schmertmann 1978). The side friction of piles may be calculated directly from the side friction of the cone, or by correlation with cone resistance.

In *cohesive soils*, the CPT is routinely used to determine both undrained shear strength and compressibility. In a similar way to the bearing capacity of a foundation, cone resistance is a function of both overburden pressure (σ_v) and undrained shear strength (c_u):

$$q_c = N_k C_u + \sigma_v \tag{9.13}$$

so that the undrained shear strength may be calculated from:

$$c_u = \frac{(q_c - \sigma_v)}{N_k} \tag{9.14}$$

provided that N_k is known, or can be estimated. The theoretical bearing capacity factor for deep foundation failure cannot be applied in this equation because the cone shears the soil more rapidly than other tests, and the soil is failed very much more quickly than in a field situation such as an embankment failure.

At shallow depths, or in heavily overconsolidated soils, the vertical total stress in the soil is small, so that:

$$c_u \approx \frac{q_c}{N_k} \tag{9.15}$$

Typically, in these conditions, the undrained shear strength is about 1/15th to 1/20th of the cone resistance.

N_k is not a constant, but depends upon cone type, soil type, overconsolidation ratio, degree of cementing, and the method by which undrained shear strength has been measured (because undrained shear strength is sample-size and test-method dependent). The N_k value in an overconsolidated clay will be higher than in the same clay when normally consolidated. Therefore it is normal to use area-specific values of N_k to calculate c. Typically, N_k varies from 10 to 20. Lunne and Kleven have shown that this variation is significantly reduced, giving N_k much closer on average to 15, if a correction ($N_k^*=N_k/\mu$) is made to allow for rate effects, in a similar way to that proposed by Bjerrum for the vane test (see below), but this is rarely done in practice. Higher N_k values are obtained from mechanical cones than from electric cones, because of differences in shape.

The stiffness of a clay can be obtained in the form of constrained modulus from the equation

$$M = \frac{1}{m_v} = \alpha_M q_c \tag{9.16}$$

where M = constrained modulus (normally in MPa), m_v = equivalent oedometer coefficient of compressibility (normally in m^2/MN), and α_M = constrained modulus coefficient.

α_M is soil specific, but approximations can be obtained from published values (for example, see Meigh (1987)). Typically, α_M lies in the range 2—8.

Probing

The use of probing to investigate the variability of the ground has been discussed in Chapter 5. The similarities between the SPT and some forms of probing make any distinction between them seem rather arbitrary. But the interpretation of probing results in terms of soil parameters is, apparently, carried out on the basis of locally derived correlations, none of which appear to have become widely or internationally accepted. Therefore we do not consider probing further in this chapter.

STRENGTH AND COMPRESSIBILITY TESTING

Because strength and compressibility parameters are generally required for engineering calculations, many forms of test have been developed with the specific purpose of determining them in particular soil or rock types. These tests are not as widely used as the penetration tests described in the previous section, but nonetheless many are in common usage. Below we describe the most popular tests in use at the time of writing.

1. *The field vane test*. This is used exclusively to measure the undrained shear strength of soft or firm clays.
2. *The pressuremeter test*. This is used routinely in France to determine strength and compressibility parameters for routine design, for all types of soil and weak rock, but (in its

self-boring form) used in the UK for special projects in overconsolidated clays, to determine undrained strength, shear modulus, and coefficient of earth pressure at rest, K_0.
3. *The plate loading test.* This is used primarily to obtain the stiffness of granular soils and fractured weak rocks.
4. *The Marchetti dilatometer.* This is not yet used commercially in the UK but, at the time of writing, is becoming more widely used in other parts of the world.

There are many other tests to be found in the literature.

In situ strength and compressibility tests are sometimes very much more expensive than laboratory tests. They suffer from the disadvantage that the soil under load has no drainage control (i.e. the true state of drainage during the test is not normally known because, unlike a triaxial test, there is no far drainage boundary), but they are often used because of the many types of soil which do not lend themselves to good- quality sampling. Tests on cohesive soils are loaded rapidly, in order that they can be assumed undrained. This gives rise to significant rate effects. Free-draining soils and weak rocks are assumed to be drained, and are generally loaded more slowly.

The field vane test

Early geotechnical engineers found difficulty in determining the shear strength of very soft and sensitive clays by means of laboratory tests, as a result of the disturbance induced by poor-quality samplers. These difficulties led to the development of the vane shear test. This device made it possible for the first time to determine the in situ shear strength and sensitivity of a soft clay.

The very first vane borer, as far as can be determined, was designed by John Olsson, the secretary of the Swedish Geotechnical Commission. It was used in 1919 during the construction of the Lidingoe Bridge in Stockholm, built between 1917 and 1926. A surface vane borer was used in England as early as 1944 by the Army Operational Research Group to investigate the mobility of military vehicles on the suggestion of the Soil Mechanics Section of the Building Research Station. A laboratory vane apparatus was also developed.

The vane borer as used today was presented for the first time by Lyman Carlsson (Cadling) in 1948 at the Second International Conference in Rotterdam. A report on a more advanced device was published two years later (Cadling and Odenstad 1950). The original Cadling vane borer, which was designed for soft soil, was pushed into the soil without preboring. The rod was encased to eliminate friction, and torque required to rotate the vane was measured at the ground surface by a separate instrument. From the torque and geometry of the vane, the shear strength of the soil could be calculated. The blades were made as thin as possible to reduce the disturbance when the vane was pushed into the soil. The vane was initially unprotected but later provided with a protective sheath to protect the vane from stones in the clay. A recent review of the field vane test can be found in Chandler (1988).

Standard testing

The following standards are known to exist for the vane test:

- USA ASTM D2573—72 (Reapproved 1978)
- UK BS 1377:part 9: 1990
- Australia AS F2.2—1977
- Germany DIN 4096 1980
- India IS 4434—1978.

Apparatus

The vane shear test basically consists of pushing a four-bladed (cruciform) vane, mounted on a solid rod, into the soil and rotating it from the surface. Vane tests may be carried out either in the field or in the laboratory. In the field they may be carried out either from ground level, or from the base of a borehole.

In its conventional form (Fig. 9.13), the field vane has four rectangular blades and a height to diameter ratio of two. In the USA vane blades, which may have tapered ends, are specified as in Table 9.6 (ASTM 2573—72 (Reapproved 1978)):

Table 9.6 USA specifications for vane blades

Casing size	Vane diameter (mm)	Vane height (mm)	Blade thickness (mm)	Diameter of vane rod (mm)
AX	38.1	76.2	1.6	12.7
BX	50.8	101.6	1.6	12.7
NX	63.5	127.0	3.2	12.7
4in. (101.6mm)	92.1	184.1	3.2	12.7

In the UK the dimensions of field vanes are controlled by BS 1377: part 9: 1990, clause 4.4.2. The height must be twice the diameter, and the Standard states that experience has shown that the following overall dimensions are suitable (Table 9.7).

Table 9.7 UK specifications for vane blades

Undrained shear strength (kPa)	Vane diameter (mm)	Vane height (mm)	Rod diameter (mm)
<50	75	150	<13
50—75	50	100	<13

By implication, BS 1377 considers that the field vane will not be suitable for testing soils with undrained strengths greater than about 75 kPa. The vane must be designed to achieve an area ratio of 12% or less (see below). The test is not suitable for fibrous peats, sands or gravels, or in clays containing laminations of silt or sand, or stones.

Four types of vane are in use. In the first, the vane is pushed unprotected from the bottom of a borehole or from ground surface. In the second, a vane housing is used to protect the vane during penetration, and the vane is then pushed ahead of the bottom of the vane housing before the test is started. In the third, the vane rods are sleeved to minimize friction between the ground and the rods during the test. Finally, some vanes incorporate a swivel just above the blades, which allows about 900 of rod rotation before the vane is engaged. This simple device allows the measurement of rod friction as an integral part of the test.

Test procedure

In all tests it is important that the vane is pushed ahead of disturbance caused either by the vane housing or any boring operations. ASTM D2573 specifies that the vane should be pushed five vane-housing diameters ahead of the vane housing before testing, and that when a borehole is used to get down to the test depth the vane should be advanced at least five borehole diameters ahead of the bottom of the borehole.

Once the vane has been pushed into the ground, it is rotated at a slow rate, preferably using a purpose-built test apparatus with an inbuilt geared drive (Fig. 9.13). Torsional force is measured, and is then converted to unit shearing resistance by assuming the geometry of the shear surface, and the shear stress distribution across it.

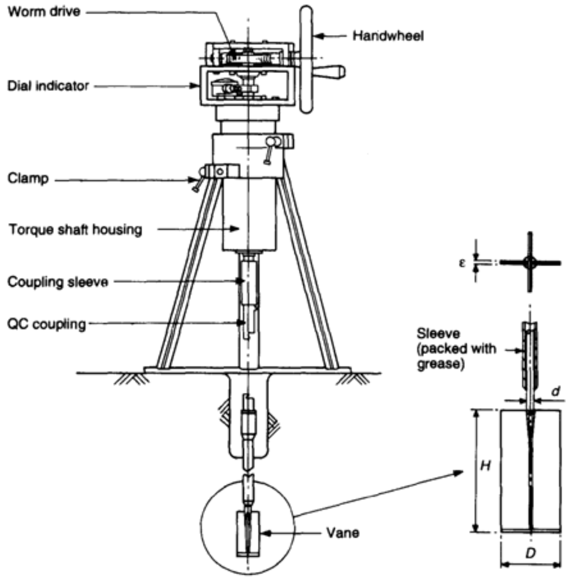

Fig. 9.13 Farnell model 274 field vane apparatus.

The test procedure is as follows.

1. Push the vane slowly with a single thrust from the bottom of the borehole or protected sleeve for the distance required to ensure that it penetrates undisturbed soil. Ensure that the vane is not rotated during this stage.
2. Attach a torque wrench, or preferably a purpose-built geared drive unit, to the top of the vane rods, and turn the rods at a slow but continuous rate. BS 1377:1990 specifies a rate of 6-12°/min whilst ASTM D2573 specifies that the rate shall not exceed 6°/min.
3. Record the relationship between rod rotation (at ground surface) and measured torque by taking readings of both at intervals of 15—30s. Once maximum torque is achieved, rotate the vane rapidly through a minimum of ten revolutions, and immediately (within 1 mm — ASTM D2573) restart shearing at the previous slow rate, to determine the remoulded strength of the soil.

Interpretation

The vane test is routinely used only to obtain 'undisturbed' peak undrained shear strength, and remoulded undrained shear strength. The undrained strength is derived on the basis of the following assumptions:

1. penetration of the vane causes negligible disturbance, both in terms of changes in effective stress, and shear distortion;
2. no drainage occurs before or during shear;
3. the soil is isotropic and homogeneous;
4. the soil fails on a cylindrical shear surface;
5. the diameter of the shear surface is equal to the width of the vane blades;
6. at peak and remoulded strength there is a uniform shear stress distribution across the shear surface; and
7. there is no progressive failure, so that at maximum torque the shear stress at all points on the shear surface is equal to the undrained shear strength, c.

On this basis (Fig. 9.14), the maximum torque is:

$$
\begin{aligned}
T &= \frac{\pi D^2 H c_u}{2} + 2\int_0^{D/2} 2\pi r \delta r \cdot r c_u \\
&= \frac{\pi D^2 H c_u}{2} + \left(\frac{4\pi r^3}{3} c_u \right)_0^{D/2} \\
&= \frac{\pi D^2 H}{2}\left(1 + \frac{D}{3H}\right) c_u
\end{aligned}
\tag{9.17}
$$

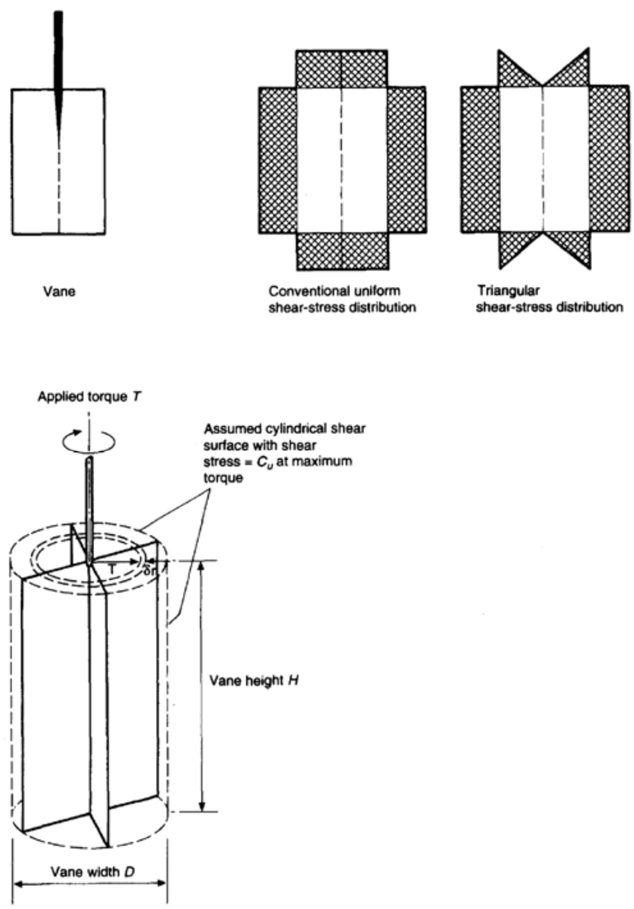

Fig. 9.14 Assumed geometry of shear surface for conventional interpretation of the vane test.

For a vane blade where H = 2D:

$$T = 3.667D^3\, c_u \tag{9.18}$$

If it is assumed that the shear stress mobilized by the soil is linearly proportional to displacement, up to failure, then another simple assumption (Skempton 1948), that the shear stress on the top and bottom of the cylindrical shear surface has a triangular distribution, is sometimes adopted. For the rectangular vane this leads to the equation:

$$T = \frac{\pi D^2 H}{2}\left(1 + \frac{D}{4H}\right) c_u \tag{9.19}$$

For a vane blade where H = 2D:

$$T = 3.53D^3 c_u \tag{9.20}$$

giving only 4% difference in shear strength from that obtained using the uniform assumption.

Discussion

The results of a vane shear test may be influenced by many factors, namely:

1. type of soil, especially when permeable fabric exists;
2. strength anisotropy;
3. disturbance due to insertion of the vane;
4. rate of rotation or strain rate;
5. time lapse between insertion of the vane and the beginning of the test; and
6. progressive/instantaneous failure of the soil around the vane.

It may readily be appreciated that the assumptions involved in the interpretation of the vane test are rarely, if ever, likely to be correct. As a result, as stated above, only a very limited range of soil may be tested. In common with other in situ tests, it is necessary to carry out the vane test rapidly, in an attempt to ensure that the shear surface remains reasonably undrained. The presence of sand or silt lenses or laminations within the test section will certainly make this assumption invalid, but (again in common with many in situ tests) it is not normally possible to know what type of material is about to be tested. Sands and gravels will drain instantly, thus invalidating any test carried out in them. The presence of stones or fibrous peat may mean that the assumption of a cylindrical shear surface with a diameter equal to the vane blade width is invalid. Skempton (1948) noted that if the cylindrical shear surface occurs at a diameter only 5% greater than the blade width (say 1.25mm outside a 50mm vane blade) then this will lead to an increase in the calculated undrained shear strength of 10%.

Most users implicitly assume the existence of a circular failure surface when calculating the undrained shear strength. This assumption is based on observations by Cadling and Odenstad who studied the shape of the surface of rupture by inserting sheets of wet tissue-paper on which a spider-web-like pattern was drawn, in between slabs of soil. By comparing the disturbance in the patterns on the tissue-paper in a series of vane shear tests with increasing rotation of the vane, it was concluded that the diameter of the cylinder of rupture closely coincided with the diameter of the vane. Skempton found that shear strength values measured with the unconfined compression test were lower than those determined with the in situ vane shear test. Reasonable agreement was obtained when the diameter of the failure surface was multiplied by a factor of 1.05. This correction factor, named ‘effective diameter’ by Skempton, was an empirical coefficient based on the (unlikely) assumption that the unconfined compressive test data represented the true in situ shear strength values. Later researchers incorrectly assumed that Skempton actually observed the diameter of the failure surface to be 5% greater than the diameter of the vane unit. Arman *et al.* (1975) found that the failure surface was circular in cross-section with the same diameter as that of the vane unit. Adjacent to the failure surface, they also noticed a very thin, partially sheared zone. They concluded that the actual diameter of the failure surface was slightly larger than the vane diameter, but that the radius of this failure zone was soil-type dependent.

Wilson (1964) noted through a series of photographs of the shearing planes that at the instant of maximum torque the failure surface is not circular in plan, but almost square. Only after considerable deformation takes place does a cylindrical surface form. In a vane test, failure can be expected to start in front of the edge of each wing and to advance gradually across the whole surface of rupture. Cadling and Odenstad in their studies with tissue-paper noted that the deformation in front of each wing seemed to be somewhat greater than behind it, but concluded that the effect of progressive failure was only slight and therefore could be ignored. One of the assumptions for the calculation of undrained strength of soils is that the maximum applied torque has to overcome the fully mobilized shear strength along a cylindrical surface. Hence, the occurrence of progressive failure may influence the final strength value.

Perhaps the most serious problem can result from the disturbance induced in the ground by the insertion of the vane blades. La Rochelle *et al.* (1973) have reported that thicker vane blades resulted in lower undrained shear strength values because of greater soil disturbance and also because of the induced increase in pore water pressure in the soil surrounding the vane. In a typical test, torque is applied shortly after insertion, and this pressure does not have time to dissipate. Hence, the time interval between the moment of vane intrusion and the time of failure is also of importance in influencing measured strength.

BS 1377:1990 specifies that the area ratio (the volume of soil displaced, divided by the volume of soil within the assumed cylindrical shear surface), which is given by:

$$A_r = \frac{[8t(D-d) + \pi d^2]}{\pi D^2} \tag{9.21}$$

where t = vane blade thickness, D = vane diameter, and d = diameter of the vane rod, below any sleeve, and including any enlargements due to welded joints, shall not exceed 12%. La Rochelle *et al.* (1973) used the definition of perimeter ratio first given by Cadling and Odenstad (1950):

$$a = \frac{4t}{\pi D} \tag{9.22}$$

to show (Fig. 9.15) that if Cadling and Odenstad's recommendation of a maximum perimeter ratio of 11% was adhered to then the undrained strength of a sensitive clay might be underestimated by some 30%. The area ratio specified by the British Standard is approximately equal to a perimeter ratio of 5.5%, but nonetheless the disturbance caused by vane insertion is a matter for concern, and engineers should ensure that the dimensions of the vanes that they use are recorded.

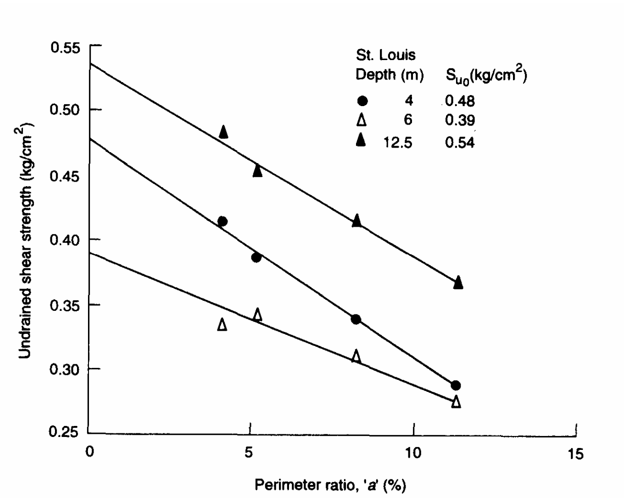

Fig. 9.15 Effect of perimeter ratio on undrained shear strength of Champlain clay (La Rochelle *et al.* 1973).

Another possible cause of variability in results may arise from the rate at which the vane is rotated during the test. Cadling and Odenstad (1950) observed a marked increase in measured strength as the angular velocity of their vane was increased from 0.1°/s to 1.0°/s. and more recent researchers have to a greater or lesser extent confirmed their findings (Aas 1965; Perlow and Richards 1977; Pugh 1978;

Torstensson 1977; Wiesel 1973). It has already been noted that the ASTM and British standards differ in their recommendations as to maximum rate of rotation.

One of the major problems involving the routine use of the field vane arose when Norwegian expertise was transferred to south-east Asia. The vane had been used very successfully in Scandinavia for many years, where excellent correlations had been obtained between vane strengths and backfigured undrained shear strengths for the soft and sensitive low-plasticity clays which are typical of this region. Its use during embankment design in the high plasticity coastal clays of south-east Asia produced unexpected overestimates of undrained shear strength. Embankments with short- term factors of safety as high as 1.65 (Parry and McLeod 1967) were observed to fail, and Bjerrum (1972) subsequently collected a series of case records which showed that the calculated short-term factor of safety of embankments which failed was a function of the plasticity of the soil (Fig. 9.16). Bjerrum interpreted this data in terms of a correction factor, μ, which varies with plasticity and which he recommended should be applied to vane strengths to give more reliable estimates of the stability of foundations and embankment side slopes.

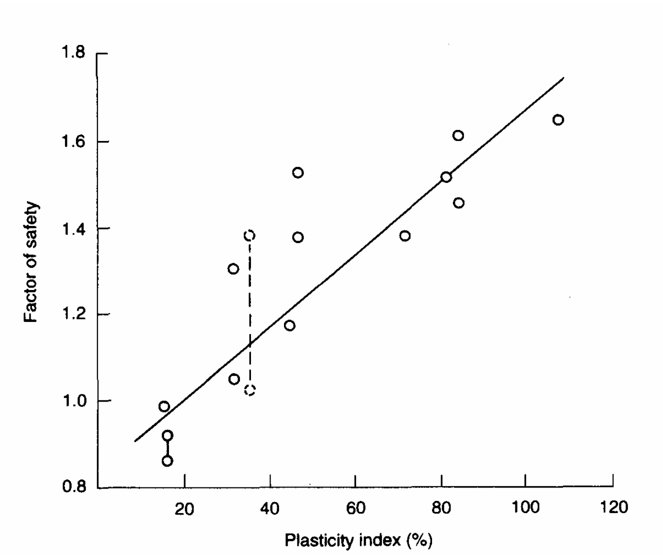

Fig. 9.16 Calculated factors of safety for failed embankments, based on (Bjerrum 1972).

Application of this factor, however, is not always satisfactory. La Rochelle *et al.* (1973) reported disagreement with Bjerrum's correction factors based on field observations. Possible causes for the difference between the various test results are soil anisotropy, differences in strain rate, soil type, and sample disturbance before testing. Most vane units have a height/diameter ratio of two (common dimensions being 130 x 65 x 3mm), which means that most of the shear strength is mobilized along the vertical cylindrical surface. Cadling and Odenstad found that a rate of 6/mm resulted in the lowest shear strength values, and this value is now the strain rate most often used in routine field testing.

Further developments

The vane test has been the subject of considerable development during research, some of which is of practical value and is therefore described below.

Strength anisotropy and non-homogeneity are quite normal in soils, either as a result of fabric and/or structure, or because of the different effective stress levels in the horizontal and vertical directions. A knowledge of the degree of variation of shear strength with orientation of the plane upon which shearing is to take place can often be important (for example, when calculating the stability of embankments). When a rectangular field vane is used, shear stresses are developed on vertical (cylindrical) and horizontal planes. Aas (1965, 1967) developed a method for assessing the degree of anisotropy of a soil by using rectangular vanes of differing heights.

From the equations above, denoting the undrained shear strength in the horizontal and vertical directions as c_{uh} and c_{uv}, respectively:

$$\left(\frac{2}{\pi D^2 H}\right)T = c_{uv} + c_{uh}\left(\frac{D}{3H}\right) \tag{9.23}$$

By plotting $(2/\pi D^2 H)T$ as a function of $(D/3H)$ for a large number of tests carried out in the same deposit, but with vane blades with differing height/diameter ratios, c_{uv}, and (c_{uv}/c_{uh}) can be found directly (Fig. 9.17). But as the height to diameter ratio decreases, the assumption of regarding the distribution of shear stress on the top and bottom of the cylindrical shear surface becomes more important, leading to uncertainties in the interpretation of the data. For example, when H = 0.5D, the difference in calculated undrained shear strength between assuming a triangular and a uniform shear stress distribution rises to 11%. As Menzies and Mailey (1976) have pointed out, the above method gives strengths in only two modes of shearing, namely horizontally on the vertical and the horizontal planes. In the field, shearing vertically on the vertical plane is normally involved.

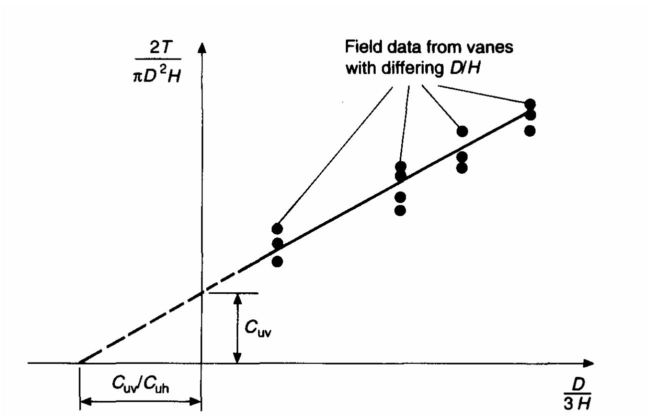

Fig. 9.17 Method of determining undrained strength anisotropy (Aas 1965, 1967).

It is also possible to determine the shear strength of a deposit on a range of surfaces other than in the vertical and horizontal, by using diamond vanes (Aas 1967; Menzies and Mailey 1976), Fig. 9.18. For example, Menzies and Mailey (1976) measured shear strengths with diamond vanes with angles varying from 20° to 70°. The interpretation of the diamond vane is, however, much more dependent upon the assumption of shear stress distribution. Following the method of analysis detailed above, for a uniform shear stress distribution:

$$T = \frac{\pi D^3 c_{u\alpha}}{6\sin\alpha} \tag{9.24}$$

whilst for a triangular distribution:

$$T = \frac{\pi D^3 c_{u\alpha}}{8\sin\alpha} \tag{9.25}$$

giving a 33% difference in calculated shear strength for a given maximum measured torque.

Researchers have also made considerable improvements in the vane equipment. Wiesel (1973) developed an electric vane borer, where torque was measured by strain gauges fastened on the shaft just above the vane. The angle of rotation was recorded by a displacement transducer located about 1.2

m above the vane and the relationship between torque and angle of rotation was automatically drawn by an x—y recorder. The vane was rotated by an electrical motor via a gearbox which allowed various rotation speeds. A similar arrangement was used by Merrifield (1980). These types of refinements will undoubtedly lead to improved data by removing rod friction effects and by ensuring that uniform rates of rotation are applied.

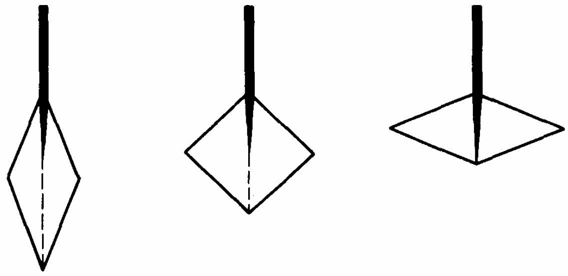

Fig. 9.18 Diamond shear vanes.

Pressuremeter testing

The pressuremeter was developed in France in the early 1950s (Ménard 1957). In its earliest form it was (and remains today) a simple, robust mechanical tool, well- adapted to use in routine investigations. Since its development there has been a considerable growth in the number of designs of pressuremeter that are in use, as will be described below. A recent review of pressuremeter testing is given by Mair and Wood (1987). Higher pressure devices of this type, designed for use in hard soils or rocks, are sometimes referred to as ‘dilatometers’.

Pressuremeter tests can be carried out both in soils and in rocks. The pressuremeter probe, which is a cylindrical device designed to apply uniform pressure to the ground via a flexible membrane, is normally installed vertically, thus loading the ground horizontally (Fig. 9.19). It is connected by tubing or cabling to a control and measuring unit at the ground surface. The aim of a pressuremeter test is to obtain information on the stiffness, and in weaker materials on the strength of the ground, by measuring the relationship between radial applied pressure and the resulting deformation. Conventional self-boring pressuremeters cannot penetrate very hard, cemented or stoney soils, or rocks. In these materials a borehole pressuremeter is normally used.

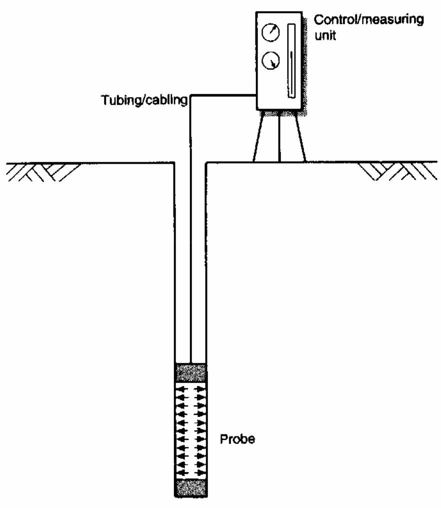

Fig. 9.19 Basic components of the pressuremeter.

Types of pressuremeter

Three principal types of pressuremeter are in use.

1. *The borehole pressuremeter.* Originally developed by Ménard, a borehole is formed using any conventional type of drilling rig capable of producing a smooth-sided test cavity. The pressuremeter has a slightly smaller outside diameter than the diameter of the hole, and can therefore be lowered to the test position before being inflated.

   There are two types of measuring system in use. In the original Ménard system the probe contains a measuring cell which is fluid-filled (Fig. 9.20). The radial expansion of the probe when pressurized is inferred from measurements of volume take made at the ground surface, using the control/measuring unit. A guard cell is incorporated into each end of the probe, in order to ensure, as far as possible, that the measuring cell expands only radially.

   In more recently designed pressuremeters (for example, the Oyo ‘Elastmeter2’) the probe is pressurized using gas, and the radial displacement is sensed electronically by diametrically opposed measuring arms. This type of pressure- meter may also incorporate a pressure transducer in the probe, thus giving better quality (but more complex) pressure measurement.

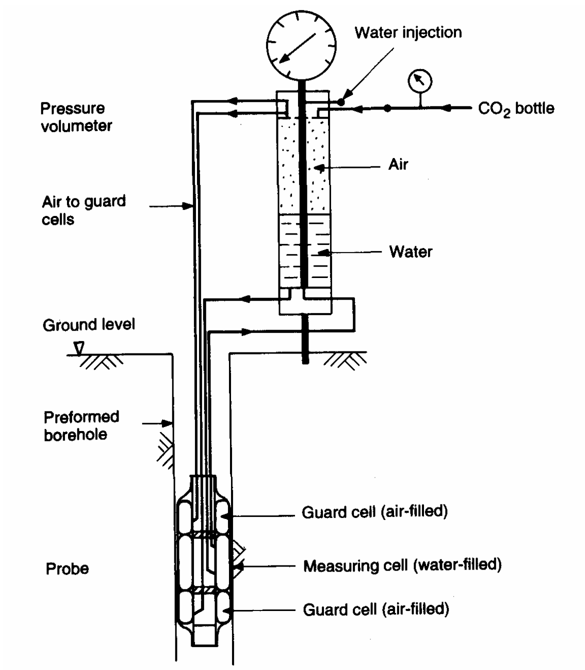

Fig. 9.20 Diagrammatic sketch of the Ménard pressuremeter (Gibson and Anderson 1961).

2. *The self-boring pressuremeter (SBP).* The self-boring pressuremeter has been developed both in France (Baguelin *et al.* 1974) and in the UK (Wroth and Hughes 1973), in an attempt to reduce the almost inevitable soil disturbance caused by forming a borehole. Borehole disturbance can have a very great effect on the soil properties determined from in situ testing, as we have already noted in the case of the SPT. A self-boring pressuremeter incorporates an internal cutting mechanism at its base; the probe is pushed hydraulically from the surface, whilst the cutter is rotated and supplied with flush fluid (Fig. 9.21). The soil cuttings are flushed to the ground surface via the hollow centre of the probe, as the pressuremeter advances. At least four distinct versions of the self-boring pressuremeter have been described:

(i) the original Cambridge self-boring pressuremeter, used widely in the UK for testing overconsolidated clays (Wroth and Hughes 1973);
(ii) the rock self-boring pressuremeter, which incorporates a full-face drilling bit, and can penetrate weak rocks (Clarke and Allan 1989);
(iii) the original French Pressiomètre Autoforeur (PAF) (Baguelin *et al.* 1972);
(iv) a more recent development of the PAF, designed to penetrate hard rocks (the Pressiomètre Autoforeur pour Sol Raide (PAFSOR)).

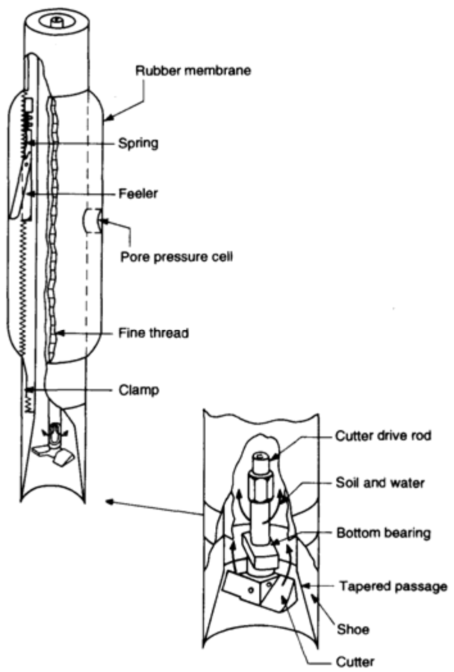

Fig. 9.21 The Cambridge self-boring pressuremeter (after Windle and Wroth 1977).

There are major differences between French and British self-boring devices:

(i) in the French PAF the radial strains are inferred from measurements of fluid take (as in the original Ménard probe), whilst in British devices measurements are made using strain-gauged feeler arms or, in the case of the rock pressure- meter, Hall effect sensors;
(ii) in British self-boring pressuremeters the membrane is supported during installation by an internal rigid metal wall, whereas the French devices are supported by liquid;
(iii) French PAF have cutters which are driven by down-hole hydraulic motors, whereas the British cutter system is driven by rods extending to ground surface; and
(iv) French self-boring pressuremeters are apparently regarded as research tools, whereas in the UK the Cambridge self-boring pressuremeter is frequently used on a commercial basis.

> In theory, the SBP offers the attractive possibility of performing tests on almost undisturbed soil. However, this important advantage of the SBP rests entirely upon its potential to test relatively undisturbed soil. The SBP is a complex device, and in order to achieve high quality installation, it requires operators of considerable skill and experience. If an SBP is inserted into the ground by operators of less skill and experience, the degree of disturbance could be such that the device offers little advantage over the simpler (borehole pressuremeter). Even with skilled operators, some degree of disturbance is inevitable.
>
> *Mair and Wood, 1987*

Factors affecting the amount of disturbance caused by insertion are:

(i) soil type;
(ii) distance of the cutter back from the lower edge of the cutting shoe;
(iii) diameter of cutting shoe relative to the uninflated outside diameter of the pressuremeter membrane;
(iv) the downward force applied during drilling; and
(v) the amount of vibration during drilling.

The degree of disturbance can be minimized by attention to each of these factors at the start of a testing programme. Regrettably this is not often done for commercial investigations.

3. *Displacement pressuremeters.* Displacement pressuremeters have, to date, been used only rarely in conventional, on-shore, site investigations. Two forms are noted. The push-in pressuremeter (PIP) (Henderson *et al.* 1979) was developed at the Building Research Station, UK. The device is illustrated in Fig. 9.22. It is primarily intended for off-shore investigations, where it is used with wireline drilling equipment. The cone-pressuremeter (Withers *et al.* 1986) is a fulldisplacement device mounted above a CPT. At the time of writing this has only recently been developed.

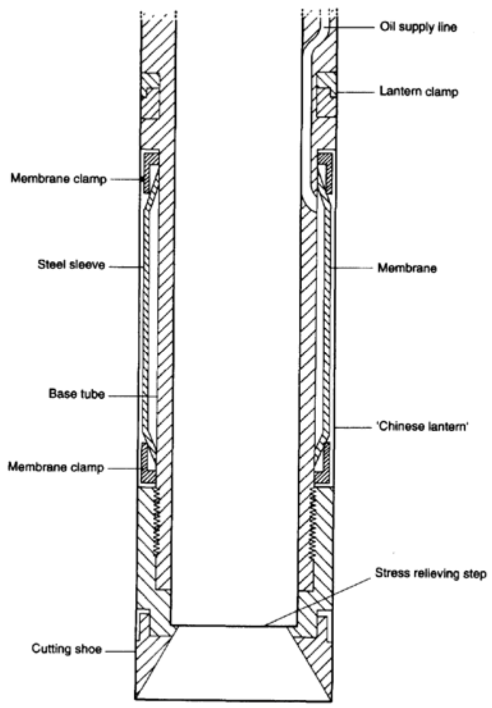

Fig. 9.22 The push-in pressuremeter.

Test methods — borehole pressuremeters

Where the soil is strong enough that a borehole will stand open, uncased, the borehole pressuremeter test may be carried out as boring or drilling proceeds or, more economically, at the completion of the hole. In ground which will not stand unsupported (for example, sands and gravels) a special slotted casing is sometimes used.

The borehole pressuremeter consists of two main elements (Fig. 9.20); a radially- expanding cylindrical probe which is suspended inside the borehole at the required test level, and a monitoring unit (known as a 'pressure-volumeter') which is deployed at ground level. As noted above, the probe consists of three cells. The outer two cells are known as 'guard cells' and are normally filled with pressurized gas. The central, measuring cell is filled with water, and is connected to a sight tube, which records volume change, in the pressure-volumeter. Pressure is provided by means of a CO2 bottle.

The pressure of both gas and water is increased in equal increments of time, and approximately equal increments of pressure. Resulting changes in measuring-cell volume are recorded at 15 s, 30s, 60s and 120 s after each pressure increment is applied. Corrections must be made (Fig. 9.23) for the following:

1. *The resistance of the probe itself to expansion.* The probe normally consists of both a rubber membrane and a thin slotted protective metal cover (sometimes known as a 'Chinese lantern'). A calibration test is carried out with the probe at ground surface to determine the specific relationship (for the pressuremeter in use) between applied pressure and the volumetric expansion of the unconfined probe. At each volume change during subsequent tests in the ground, the calibration pressures are deducted from the measured pressure.
2. *The expansion of the tubes connecting the probe with the pressure-volumeter.* The required corrections can be determined by conducting a surface test in which the probe is confined in a rigid steel cylinder, where all measured volume change results from expansion of the leads and the pressure-volumeter. At each pressure during subsequent tests in the ground, the calibration volume changes are deducted from those recorded at the given pressure.
3. *Hydrostatic effects.* These are due to the fact that the measuring cell and its leads are filled with water, and therefore the pressure in the measuring cell is higher than that recorded by the pressure volumeter. In probe/pressure-volumeter systems where the guard cells contain air, Gibson and Anderson (1961) note that it may become necessary to use two pressure sources in order to give equal pressures in both guard and measuring cells, when working at depths in excess of 30m.

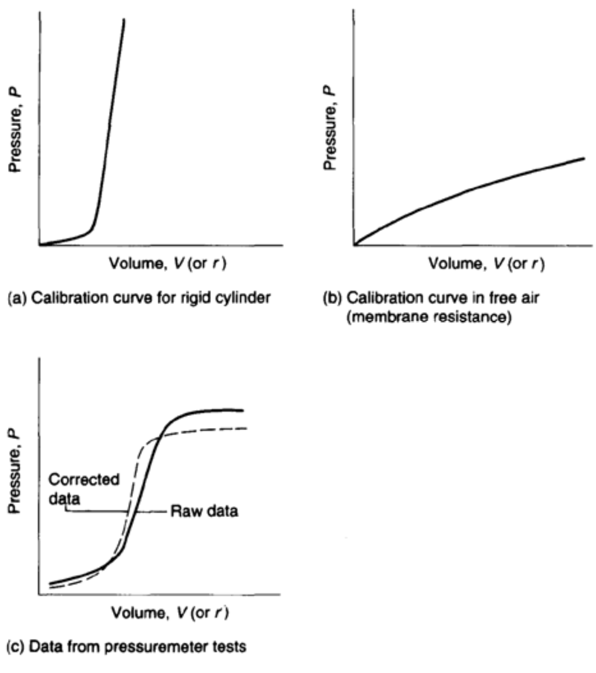

Fig. 9.23 Pressuremeter calibration plots and data correction (Mair and Wood 1987).

Each test consists of about ten approximately equal pressure increments. The number of increments actually achieved will depend upon the accuracy of the operator's prior estimate of the limit pressure. Between 5 and 14 increments are normally considered acceptable. In some circumstances, for example when testing in weak rock, an unload— reload loop may be carried out.

After the application of calibration corrections, the results are plotted (Fig. 9.24) as:

1. a pressuremeter curve (i.e. corrected volumetric expansion (at 120 s) as a function of corrected pressure); and
2. a creep curve (i.e. the measured volume change between 30s and 60s, for each pressure, also plotted as a function of corrected pressure).

The pressuremeter curve can be divided into three phases:

1. bedding of the probe against the borehole wall, and re-establishment of horizontal in situ stress ($p<p_0$);
2. pseudo-elastic linear stress—strain behaviour, with low levels of creep ($p_0<p<p_f$); and
3. plastic deformation, with increasing amounts of creep measured as the soil approaches failure ($p_f <p <p_L$).

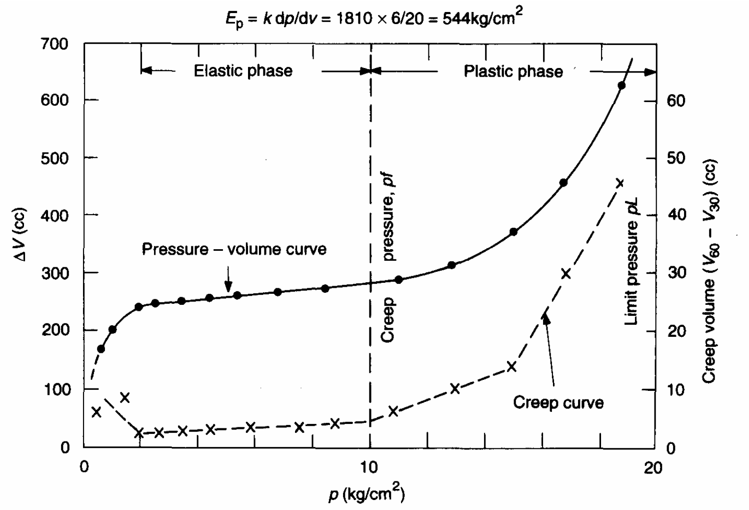

Fig. 9.24 Pressuremeter test curve on mudstone (Meigh and Greenland 1965).

Test method — Cambridge self-boring pressuremeter

Both stress and strain control can usually be applied to this type of pressuremeter, via a computer-controlled pressure system. It is normal to adopt a stress-controlled approach in the early part of the test, followed by strain control once plastic strains commence ($p >p_f$). For clays, Windle and Wroth (1977) suggest that a strain rate of 1%/mm is suitable. High rates of strain are required in order to ensure, as far as possible, that the test remains undrained. During self-boring pressuremeter testing it is normal to include at least one small unload—reload loop, in order to allow stiffness to be calculated. A final unload curve is also normally obtained.

As with the borehole pressuremeter, the results must be corrected for membrane stiffness and system compliance before being plotted. But in this case careful additional calibrations are also necessary for the various electronic instruments (pressure transducers and displacement strain followers) that are used. Mair and Wood (1987) very sensibly recommend that the engineer commissioning pressuremeter tests should require both the raw data and the calibration data to be reported, in order that the accuracy with which the corrections have been applied can be checked.

After application of corrections, self-boring pressuremeter test results are plotted as a curve of corrected pressure (p) as a function of cavity strain (ε_c). Cavity strain is the radial strain of the cavity, i.e.

$$\varepsilon_c = \frac{d - d_0}{d_0} \tag{9.26}$$

where d_0 = original diameter of the pressuremeter just before the start of inflation, under (ideally) the in situ horizontal total stress, and d = current diameter of the cavity, after expansion under pressure p.

Results and interpretation

The principal differences between the three classes of pressuremeter described above lie in the stresses applied to the probe at the start of the test. Borehole pressuremeters start from a horizontal total stress level close to or equal to zero. Self-boring pressure- meters start their test at approximately the horizontal total stress level in the ground before insertion. Displacement pressuremeters (because they push soil aside during installation) start with a horizontal total stress which can be expected to be much greater than originally existed in the ground. The increases in horizontal total stress applied during the test itself take soil to failure, although in rock this may not be achievable.

Conventionally, borehole pressuremeter test results are plotted in the form of change in volume as a function of applied pressure (as, for example, in Fig. 9.24), whilst self-boring pressuremeter results are plotted as applied pressure as a function of cavity strain (see Fig. 9.25a). In Fig. 9.26, results from the three types of test are contrasted schematically. The borehole pressuremeter starts from a zero (or very low) total stress, and initially relatively large radial strains are required to bed in the probe and bring the pressure on the borehole boundary back to the original in situ horizontal stress. The displacement pressuremeter starts at much higher pressures, implying disturbance as a result of pushing aside the soil. It is conventionally assumed that the disturbance caused by a borehole or displacement pressuremeter will have little effect on the measured properties, but because all soils are (to a greater or lesser extent) bonded, this cannot be true. Therefore, in principal, the self-boring test is to be preferred.

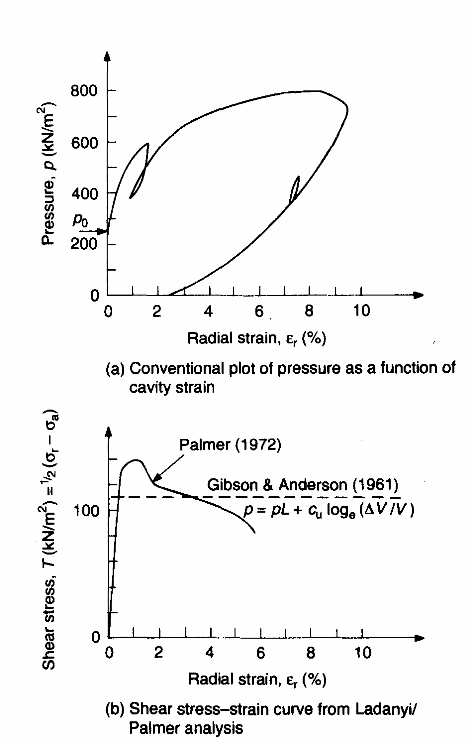

Fig. 9.25 Results from a Cambridge self-boring pressuremeter test in Gault clay (Windle and Wroth 1977).

Although, in theory, tests in all soil types are capable of interpretation in one way or another, in practice the most common methods are as discussed below.

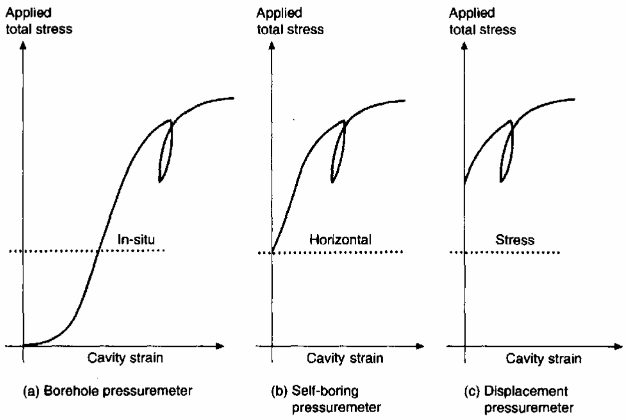

Fig. 9.26 Idealized results from three types of pressuremeter.

Semi-empirical approach

French practice, developed over the past few decades, is used in conjunction with the borehole pressuremeter. In this approach the pressuremeter is not thought of as a means of obtaining fundamental soil parameters, and the results are used in an empirical fashion (see, for example, Baguelin *et al.* 1978).

The limit pressure (p_L) for a borehole pressuremeter test was defined by Ménard as the pressure necessary to expand the probe to twice its original volume. The net limit pressure (p^*_L) is defined as:

$$p_L^* = p_L - \sigma_{h0} \tag{9.27}$$

where σ_{h0} = in situ horizontal stress in the ground.

In French practice the limit pressure is used empirically in providing design values, for example, for the bearing capacity of foundations. In clays, p_L is related to undrained shear strength (c_u) by:

$$c_u = \frac{p_L^*}{N_p} \tag{9.28}$$

where the factor N_p varies between about 5.5 and 10.0 (Baguelin *et al.* 1978).

A pressuremeter modulus (E_M) is obtained from the gradient of the pressuremeter curve (i.e. the pressure—volume curve) in the pseudo-elastic (straight-line) region by the equation:

$$E_M = A\frac{\Delta p}{\Delta V} \tag{9.29}$$

where A is a function of the probe size and Poisson's ratio (the latter being taken arbitrarily as 0.33).

From elastic analysis:

$$\frac{\Delta V}{V} = \frac{2\Delta p(1+\nu)}{E} \tag{9.30}$$

and therefore

$$E = 2(1+\nu)V\frac{\Delta p}{\Delta V} \tag{9.31}$$

where V = volume of the measuring cell at the point of measurement (= $V_0 + V_m$).

Typically, then, A = 1500—3000 cm^3 for a Ménard-type probe with an increase in volume of 200 cm^3, because:

$$A = 2(1+\nu)(V_0 + V_m) \tag{9.32}$$

Baguelin *et al.* (1978) give a table of α values by which E_M should be divided in order to obtain design values of Young's modulus, E. α varies from 0.25 to 1.0, depending upon E_M/p_L and soil type.

Analytical approach

British practice, developed partly for the borehole pressuremeter but largely for the self-boring pressuremeter, attempts to determine the more fundamental properties of the soil. Analytical techniques are available not only to interpret the results of tests in clays, but also for those in sands. These latter procedures are used rather infrequently at present, and appear still to be developing, and therefore are not considered further. The reader is referred to Mair and Wood (1987).

CLAYS

In clays, the pressuremeter curve starts, at least notionally, at the in situ stress. It then proceeds through an elastic phase, and an elasto-plastic phase. At least one unload— reload cycle is carried out. The interpretation of self-boring pressuremeter tests can be based upon all of these phases, in order to obtain in situ horizontal total stress, stiffness, and undrained strength.

In situ horizontal stress. In situ horizontal stress is normally determined using the liftoff method (Fig. 9.27). The point of lift off is detected by a break in the initial slope of the cavity strain — pressure curve. The initial, stiff part of the pressure strain relationship is a function of strain-arm and membrane compliance. It is normal to examine the curves for each of the three strain arms independently (see, for example, Dalton and Hawkins (1982)). The time between the end of drilling and the start of testing will probably have an influence on the values obtained. Mair and Wood (1987) note that it may be desirable to use a rest period of between 1 and 2 h in order to overcome some of the mechanical disturbance effects associated with poor installation procedures.

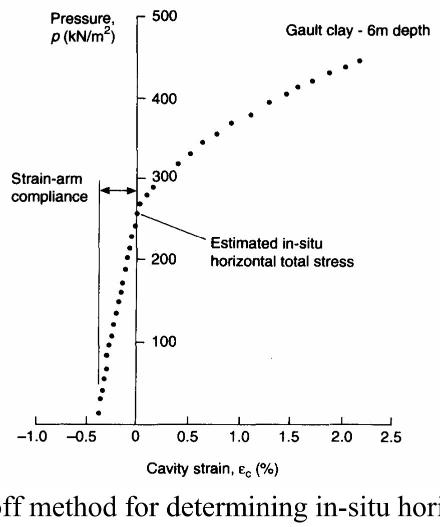

Fig. 9.27 Illustration of lift-off method for determining in-situ horizontal stress (from Dalton and Hawkins 1982).

Stiffness. Stiffness is determined in the form of shear modulus, G (for an isotropic elastic solid G=E/[2(1 + v)]), from the slope of the unload—reload loops in the loading curve. If the soil is presumed to behave elastically, as might be expected for an elastic—perfectly plastic material during a relatively small unload—reload cycle, then the shear modulus is:

$$G = \frac{1}{2}\left(\frac{d}{d_0}\right)\left(\frac{dp}{d\varepsilon_c}\right) \tag{9.33}$$

In practice the term (d/d_0) is often neglected, because it is close to unity when unload—reload loops are carried out at relatively small cavity strains, and therefore:

$$G \approx \frac{1}{2}\left(\frac{dp}{d\varepsilon_c}\right) \tag{9.34}$$

This simplification should not be made at larger cavity strains.

Strength. Two types of analysis have been used to determine the shear strength of clays from pressuremeter data; the Gibson and Anderson analysis, and the Palmer/Ladanyi analysis.

Gibson and Anderson (1961) derived (for the borehole pressuremeter) the expression:

$$p = p_0 + c_u + c_u \log_e\left[\frac{\Delta V}{V} - \left(1 - \frac{\Delta V}{V}\right)\frac{p_0}{G}\right] \tag{9.35}$$

which can be rewritten, for a self-boring pressuremeter, whose volume changes start from p_0 as

$$p = p_0 + c_u\left[1 + \log_e\left(\frac{G}{c_u}\right)\right] + c_u\left[\log_e\left(\frac{\Delta V}{V}\right)\right] \tag{9.36}$$

for an elastic—perfectly plastic soil, once yielding commences (at $p = p_0 + c_u$). As pressure increases the volume of soil undergoing plastic straining increases, and the tangent stiffness decreases, since the increasing volume of material shearing plastically has no tangent stiffness. At infinite strain $\Delta V/V = 1$, and because all strain is plastic the limit pressure, p_L is reached, where:

$$p_L = p_0 + c_u\left[1 + \log_e\left(\frac{G}{c_u}\right)\right] \tag{9.37}$$

This expression can be used to determine the undrained shear strength, c_u, only if the true limit pressure can be determined, which occurs when infinite probe expansion occurs (at $\Delta V/V = 1$). But self-boring pressuremeters tend to have even more limited ranges of expansion than borehole pressuremeters, and it is therefore necessary to extrapolate considerably to determine p_L.

From the above equations it can be seen that during the plastic deformation phase ($p > p_0 + c_u$):

$$p = p_L + c_u \log_e\left(\frac{\Delta V}{V}\right) \tag{9.38}$$

The limit pressure can therefore be estimated by plotting the corrected data for the last part of the test as change in volume over total volume, $\Delta V/V$ as a function of corrected cell pressure, p. When plotted in this way the data should form an approximately straight line—Fig. 9.28. By extrapolating to $\Delta V/V(=V_m/(V_0 + V_m))= 1$, and in the case of a self-boring pressuremeter, by first converting cavity strain through the expression:

$$\varepsilon_c = \left(1 - \frac{\Delta V}{V}\right)^{-1/2} - 1 \tag{9.39}$$

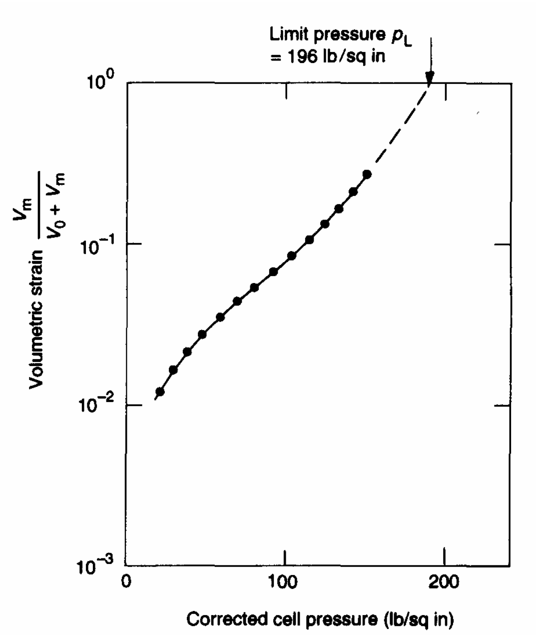

Fig. 9.28 Method of extrapolating to limit pressure (Gibson and Anderson 1961).

A continuous stress—strain curve can be obtained from analytical methods proposed by Ladanyi (1972) and Palmer (1972), who showed that the shear stress, τ, at any stage of the test is:

$$\tau = \frac{dp}{d\left[\log_{10}\left(\frac{\Delta V}{V}\right)\right]} \tag{9.40}$$

where dp and dV are changes from in situ stress levels, rather than from p = 0. An example of this interpretation is given in Fig. 9.25b. The peak shear stress (= undrained shear strength) given by this method is generally higher than that given by the Gibson and Anderson analysis, and does not seem to correlate so well with the results of other test methods. Mair and Wood (1987) do not recommend the use of the Palmer/ Ladanyi analysis.

ROCKS

In rocks, the self-boring pressuremeter has recently been developed to improve penetration (by using a full-face drilling bit), and to give greater sensitivity to cavity strain (by incorporating Hall effect sensors) (Clarke and Allan 1989). This is still a relatively untried tool, however. In contrast, the borehole pressuremeter has long been used to obtain stiffness values in weak rocks and saprolitic soils. Meigh and Greenland (1965) showed good agreement between ultimate bearing pressure obtained from small-diameter plate tests and limit pressure values, with reasonable agreement between modulus values derived from plates and from the pressuremeter. However, the small size of the pressuremeter tests, and the dominant effects of fracture stiffness and orientation on rock mass stiffness suggest that this may often not be the case. Recently, Haberfield and Johnston (1993) have argued that the only parameter that can be determined with any reliability in weak rock is the shear modulus, and that for this to be reliable joint spacing must be less than about 20mm (for a typical pressure- meter of approximately 75 to l00mmdia.). With current technology, they argue that it is doubtful that an accurate estimate of in situ stress can be obtained, and that strength properties are best measured by other methods.

Plate loading tests

Plate loading tests provide a direct measure of compressibility and occasionally of the bearing capacity of soils which are not easily sampled. Probably the most well known use was by Terzaghi and Peck (1948) in the derivation of their settlement charts for footings on sand (Bazaraa 1967), but plate loading tests are also extremely useful in assessing the properties of weak rocks (Marsland 1972; Hobbs 1975).

Techniques for carrying out the plate loading test have been described by CP 2001:1957, ASTM D1194-72, Tomlinson (1980) and BS 5930:1981 and BS 1377:1990. Figure 9.29 gives a typical set of results. In the test, a plate is bedded on to the soil to be tested, either using sand/cement mortar or Plaster of Paris. Load is applied to the plate in successive increments of about one fifth of the design loading, and held until the rate of settlement reduces to less than 0.004mm/mm, measured for a period of at least 60mm. Load increments are applied either until:

1. shear failure of the soil occurs; or more commonly
2. the plate pressure reaches two or three times the design bearing pressure proposed for the full-scale foundation.

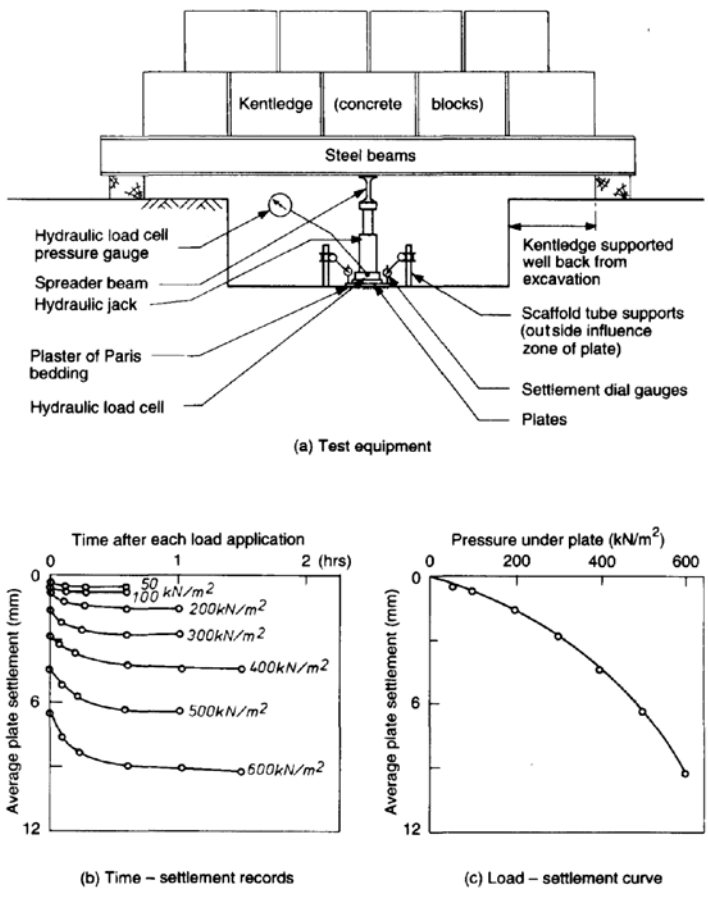

Fig. 9.29 Plate loading test layout and result.

Load is usually applied to the plate via a factory calibrated hydraulic load cell and a hydraulic jack. The hydraulic jack may either bear against beams supporting kentledge, as shown in Fig. 9.29, or reaction may be provided by tension piles or ground anchors installed on each side of the load position. When kentledge is used, the maximum plate size practicable may be considered to be about 1 m dia., since such a plate loaded to two and a half times a design pressure of 200 kN/m^2 will require about 40 tonnes of kentledge.

Where large plates are used they should be made as rigid as possible by stacking successively smaller spreader plates above them and below the load cell. Thus a 1 m plate will typically have 0.75m, 0.50 and 0.30mdia. plates above it. The minimum plate size should be 0.30m.

Settlement is measured using dial gauges reading to 0.05 or 0.01 mm. In order to measure any tilt that may occur it is advisable to use four gauges on the perimeter of the largest plate. These gauges are normally supported on rigid uprights driven firmly into the ground at a distance of at least twice the plate width from the plate centre: a necessary precaution to avoid plate settlement interfering with the datum level.

At each pressure increment, a note is made of the load on the plate and dial gauge readings are made on a 'square of the integer' basis (i.e. 1, 4, 9, 16, 25 mm, etc.) after load application. This will ensure sufficient readings in the early stages of each load application when movement occurs most rapidly.

The results of these measurements are normally plotted in two forms: a time— settlement curve and a load—settlement curve (Figs 9.29b,c). Owing to the natural variability of soil a single test will rarely be sufficient, but due to the relatively high cost of the test many tests will not be possible. Tomlinson (1975) quotes the cost of a single plate test with a 300—600mm plate with 50 tonnes of kentledge as being equivalent to three 12m deep boreholes complete with conventional SPT testing, open-drive sampling and laboratory testing, and yet such a plate test will investigate considerably less than 1 m^3 of ground.

The number of tests that should be carried out depends on both the soil variability and the consequences of poor data on geotechnical design. Tests should not normally be carried out in groups of less than three, and in order to allow assessments of variability any plate testing should be carried out at the end of a site investigation, or as part of a supplementary investigation.

The size and location of plate tests should be assessed on the basis of in situ testing and visual examination of the soil or rock to be investigated. As a general rule of thumb the plate diameter should never be less than either six times the maximum soil particle size or six times the maximum intact rock block size. Thus Lake and Simons (1975) suggested the use of a 600mm dia. plate on grade III chalk (Ward *et al.* 1968) which had an intact block size in the range 50—100mm, based on predicted and observed settlements of a building at Basingstoke, England. The use of the above rule ensures that enough discontinuities or inter-particle contacts exist in the stressed zone to give representative results, but it does not aid in extrapolating results when tests are carried out only at proposed foundation level. Under these conditions, Terzaghi and Peck (1948) observed that predictions made on the basis of uniform compressibility with depth over-estimated the settlement of structures. This would be expected, since although elastic stress distribution predicts that the stressed depth beneath a foundation is proportional to the foundation width, it is commonly observed that soil becomes less compressible with depth. This not only reduces the settlements at depth, but tends to restrict significant stress increases to smaller depth/width ratios (Gibson 1974).

Terzaghi and Peck proposed:

$$\rho_B = \rho_1 \left(\frac{2B}{B+1} \right)^2 \tag{9.41}$$

where ρ_B = settlement of a footing of width B, ρ_1 = settlement of a 1 ft plate, which leads to a maximum settlement ratio of 4, however big the footing. Bjerrum and Eggestad (1963) investigated Terzaghi and Peck's relationship for settlement ratio, and found a considerable scatter, (Fig. 9.30). It can be seen that Terzaghi and Peck's relationship is close to Bjerrum and Eggestad's lower extreme', while Bjerrum and Eggestad's upper bound approximates to (ρ_B / ρ_1)= B for B <l0ft (3m) and gives a settlement ratio of 30 for B 100 ft (30 m). Bjerrum and Eggestad found no differences in correlation for footings on dense, medium or loose sand, but considered that the upper extreme represented very loose, slightly organic sand while foundations on dense sand would give points between the average and lower extremes. This evidence is not supported by more recently published work which gives even higher settlement ratios. D'Appolonia *et al.* (1970) give values of settlement ratio for dense sands which fall above the extreme of Bjerrum and Eggestad, and Sutherland (1975) gives values taken from Levy and Morton (1975) which are even higher.

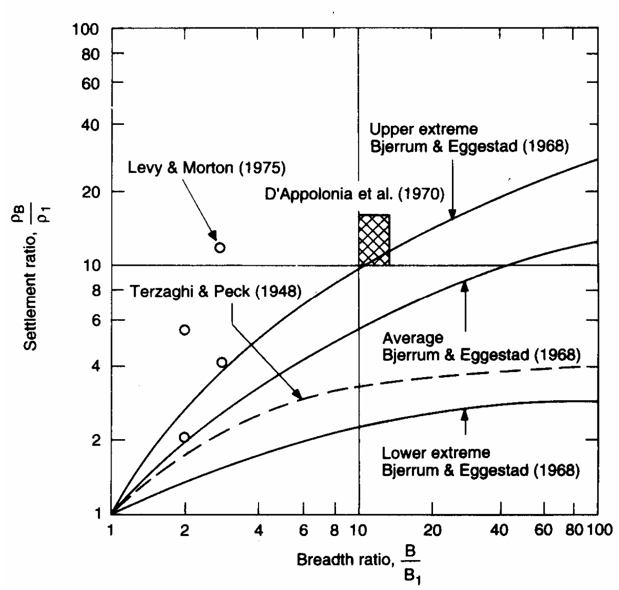

Fig. 9.30 Correlation between plate bearing tests and settlement of foundations (Sutherland 1975).

It is quite clear from the discussion above that extrapolation of settlement from small plates to large loaded areas on granular soils is rather unreliable, and therefore the plate loading test on granular material should be regarded as giving a modulus of compressibility value for the soil immediately beneath the test location. Elastic stress distributions indicate that the soil will only be significantly stressed to a depth below the plate of about 1.0—1.5 times the width of a square or circular loaded area.

Plate tests on rocks appear to present a rather more attractive proposition, because reliable methods of predicting settlements on rocks are almost non-existent, and also because the sorts of structure for which good estimates of settlements on rock are required will normally justify the high expenditure necessary. Most civil engineering structures will be founded in the upper, more weathered rock zones. Ward *et al.* (1968) have shown that in these zones it is the compressibility of the discontinuities, and not of the intact rock, which controls the compressibility of rock in the mass. The compressibility of joints and bedding planes can be assessed visually, based on experience, but actual test values can only be obtained satisfactorily from an in situ loading test.

Lake and Simons (1970) have proposed that the results of plate loading tests on chalk can be extrapolated using the expression:

$$\frac{\rho_f}{\rho_p} = \left(\frac{B_f}{B_p}\right)^{\alpha} \tag{9.42}$$

where ρ_f, ρ_p, are the settlements of foundation and plate respectively, and B_f, B_p are the widths of foundation and plate.

Lake and Simons (1970) considered that when extrapolating from plate tests at foundation level to the full scale foundation it is prudent to adopt α= 1, and this is supported by results presented by Hobbs (1975), which show considerable scatter.

A more logical approach to the problem is to use a plate of sufficient size to determine a modulus value, and either to carry out tests at different levels, or to correlate tests on different materials with their weathering grades. The latter approach is less satisfactory, but the cost of deep plate tests usually makes testing at various levels prohibitively expensive. Borehole plate tests have been used (for example, Lake (1975)) but the problems of disturbance at the base of the hole, and of necessary borehole and plate size do not make this procedure suitable in soft rocks such as the chalk. In the stiff fissured overconsolidated London clay at Wraysbury, Marsland (1972) achieved much better repeatability of ultimate bearing pressures with a plate test than could be obtained with undrained triaxial test results. Figure 9.31a shows the usual scatter of triaxial test results which is normally expected from tests on stiff fissured clays while Fig. 9.31b shows the individual plate test results achieved by testing the base of a bucket-augered borehole. This type of drilling will provide a much cleaner hole bottom than can be achieved either by a light percussion rig using a claycutter or by a continuous auger.

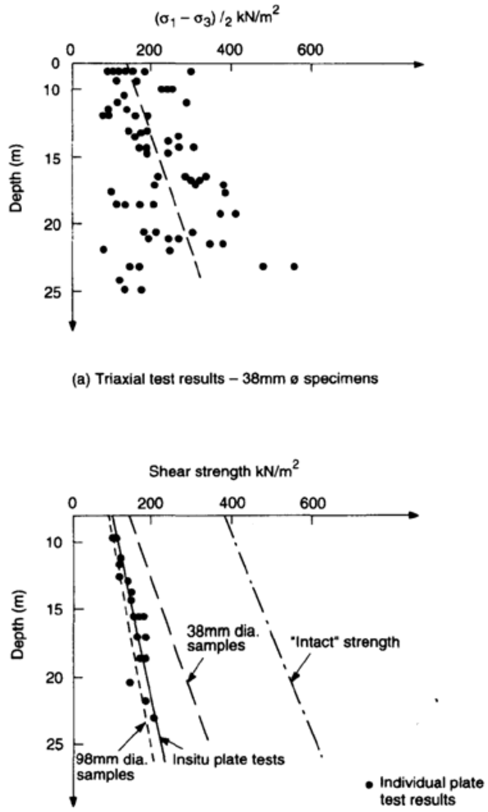

Fig. 9.31 Comparison of plate tests and laboratory test results in London clay (Marsland 1972).

For plate tests intended to give elastic moduli values for soils or rocks BS 5930:1981 recommends the use of the equation for a uniformly loaded rigid plate on a semi- infinite elastic isotropic solid, i.e.

$$E = \frac{\pi\, qB}{4}\frac{\left(1-\nu^2\right)}{\rho} \tag{9.43}$$

where E = elastic modulus, q = applied pressure between plate and soil, B = plate width, ρ = settlement under applied pressure q, and ν= Poisson's ratio.

For granular soils and soft rocks Poisson's ratio will normally be between 0.1 and 0.3, and so the term $(1 - \nu^2)$ has a relatively small effect. Where plate tests are carried out in the stressed zone of a proposed foundation the value of q can be taken as the vertical foundation stress to be applied at the level of the plate test, or alternatively, a safety margin can be incorporated by taking q to be 50% (for example) higher than the estimated applied stress.

Where plate tests are intended to give values of shear strength or bearing capacity in cohesive soils, the load is not applied in stages. The plate is pushed downwards to give a constant rate of penetration, and the undrained shear strength is deduced from the ultimate bearing capacity using eqn 9.44:

$$c_u = \frac{q_{ult} - \gamma H}{N_c} \tag{9.44}$$

where c_u = undrained shear strength, q_{ult} = ultimate bearing capacity, γ = average bulk unit weight of the soil above the test position, H depth at which the test is made, and N_c = bearing capacity factor, normally 6.15 for a circular loaded area at the surface and 9.25 when the test is carried out using a plate in the base of a borehole having the same diameter as the base of the hole (but see Hillier (1992) for a discussion).

Where the ultimate bearing capacity is not obvious from the load/settlement curve, it may be assumed to occur at a settlement equal to 15% of the plate diameter.

Two enhancements are now sometimes used in conjunction with larger, more expensive, plate tests, such as are used for major investigations (reactor foundations or underground caverns) in weak rocks.

1. Multi-point borehole extensometers may be placed under the plate, in order to allow the determination of strain levels at various distances away from the loading (for example, see Marsland and Eason (1973)) and Barla *et al.* (1993)). Stress changes at the measuring points must be determined from elastic theory, even though this may be rather unreliable (Hillier 1992).
2. An oil-filled pad (similar to a flat jack) may be placed between the plate and the rock in an attempt to remove the concentration of stresses at the plate edge, which are produced by its rigidity. When this is done, then estimates of stress change are improved, but interpretation of surface movements should be made on the basis of the settlement of a fully flexible loaded area on an elastic half space, i.e.

$$E = \frac{qB\left(1-\nu^2\right)}{\rho} \tag{9.45}$$

Smaller plate-loading tests are routinely carried out down-hole in some countries (for example, South Africa) as part of investigations which rely upon visual description. This combination has proved particularly valuable above the water table in hard, gravelly or unsaturated and saprolitic soils, all of which can be very difficult to sample and test. The plate test is carried out across a large-diameter hole which is formed using a auger-piling rig. An engineer or geologist is lowered down the hole to describe the ground and produce a borehole record. Test depths are then selected, and diagonally opposed faces are hand trimmed to provide flat areas upon which the small-diameter (100, 200 or 300 mm) plate test will bear. Details can be found in Wrench (1984).

Another adaptation of the plate test is the ‘skip test’. Here a heavy-duty waste- disposal skip is used to simulate the relatively low levels of loading produced, for example, by low-rise housing. This type of test is now the subject of a standard (BS 1377:part 9:1990, clause 4.2, *Determination of the settlement characteristics of soil for lightly loaded foundations by the shallow pad maintained load test*). Settlements are measured using levelling.

The Marchetti dilatometer (DMT)

The Marchetti dilatometer test, also known as the DMT, is carried out by pushing or hammering a special dilatometer blade (Fig. 9.32) into the soil, whilst measuring penetration resistance, and then using gas pressure to expand a 60mm dia. thin steel membrane (mounted on one side of the blade) approximately 1mm into the soil. The operator measures various pressures during the inflation—deflation cycle, before advancing the blade to the next test depth. The test is generally well adapted to normally consolidated clays and uncemented sands, where the force required for penetration is relatively low, but it is also finding increasing use in overconsolidated cohesive deposits. Typically a hydraulic CPT rig is used to advance the probe, although conventional boring equipment, together with an SPT trip hammer can also be used.

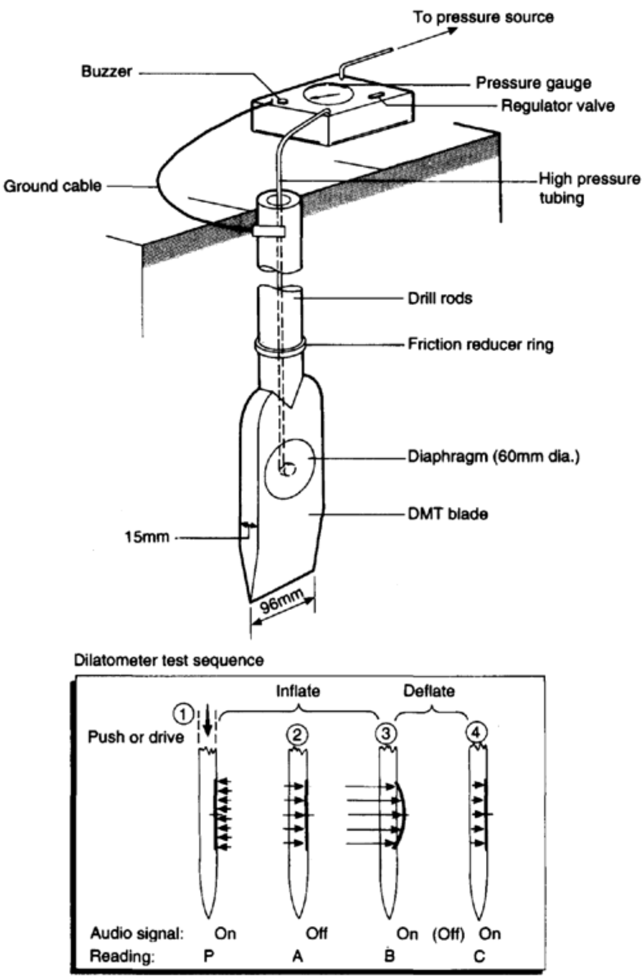

Fig. 9.32 Marchetti dilatometer.

The test was first reported by Marchetti (1975). A 'Suggested Method' of test (in effect a draft ASTM standard) has been published (Schmertmann 1986). The test has not yet been widely used in the UK, although it has obvious potential. Some guidance on its performance in UK ground conditions can be found in Powell and Uglow (1988) and Uglow (1989). In other parts of the world, and most noticeably in the USA, the use of the test has increased dramatically in the last decade. A state-of-the-art report on the test was given at the First International Symposium on Penetration Testing (ISOPT-1) by Lutenegger (1988).

Equipment

The test equipment consists of a blade (Fig. 9.32) conforming to the dimensions given by Marchetti (1980), together with rods and a control unit. In most situations the blade is pushed from ground surface, without the need to make a borehole, and drilling disturbance is therefore avoided.

The blade is 95mm wide, 14mm thick, with a base apex angle of about 12—16°. Mounted on one side of the blade is a stainless-steel membrane, which is expanded by gas (preferably dry nitrogen) pressure supplied through the control unit, by a small gas cylinder at ground surface. Behind the membrane a spring-mounted electrical sensor is used to detect two positions, when:

1. the centre of the membrane has lifted off its support and moved horizontally 0.05 (+0.02—0.00) mm; and
2. the centre of the membrane has moved horizontally 1.10 (±0.03) mm from its support.

The electrical sensor is a switch, and this is generally used to sound an audible tone in the control box. As the membrane expands away from its support the tone should cease cleanly at 0.05 mm, returning once a deflection of 1.05 mm is achieved.

The blade is connected to the rods to the ground surface, and by a pneumatic- electrical cable to the control box. The small control box contains a dual-range, manually read Bourdon pressure gauge, and valves to control gas flow and vent the system. An electrical ground cable is used to ensure continuity between the control box and the blade.

A simple calibration unit is required, in order that the pressures necessary to achieve the 0.05mm and 1.10mm membrane movements in free air may be measured. At the same time, the displacements at which the switch is tripped can be checked.

Test method

The test method originally described by Marchetti (for example, in Marchetti (1980)) and in the ASTM Suggested Method (Schmertmann 1986) differ in detail. The description given below is based upon the ASTM Suggested Method.

Calibration of the unrestrained membrane should take place at ground surface before and after each DMT sounding. About 5 mm is required. Apart from checking the correct functioning of the switch, two values of pressure are measured.

- ΔA is the gauge pressure necessary to suck the membrane back against its support. During initial preconditioning of new membranes, which is carried out by cycling them about 20 times, the membrane develops a permanent deformation such that its at-rest position (with no pressure or suction applied) lies somewhere between the support and the 1.10mm deflection position. ΔA is recorded as a positive pressure, even though it is applied as a suction.
- ΔB is the gauge pressure necessary to move it outward to the 1.10mm position.

The blade is pushed into the soil at between 10mm/s and 30mm/s. Penetration resistance is measured (usually at the ground surface, but preferably by using an electrical load cell mounted in the rod directly above the blade) during the last 10mm of penetration before stopping to carry out an inflation of the membrane. During the advance the membrane should be forced back against its support, and therefore at this stage the control box should be producing its audible signal.

Within 15s of reaching the test depth the rods are unloaded, the control-box vent is closed, and the gas-control valve is used to pressurize the membrane. The cessation of the audible signal indicates the point at which membrane lift-off has occurred, and the A-pressure is then recorded. This should occur within 15—30s from the start of pressurization.

The gas pressure is smoothly increased so that in the next 15—30s the membrane inflates to 1.10 mm, and the audible signal returns. The B-pressure is then recorded. The vent on the control box is immediately opened, in order to prevent damage to the membrane as a result of over-expansion, and the gas control valve is closed. Alternatively, a controlled depressurization may be carried out to determine the point at which the membrane returns to its original position, which is recorded as the C-pressure.

The blade is pushed to its next test depth, and the procedure repeated. The interval between test depths is typically between 0.15 and 0.30m. Each test sequence takes about 2 mm, so that a 30 m deep DMT sounding can be carried out (provided no obstructions are encountered) in a few hours.

Reduction of test data

The A- and B-pressure readings are corrected, using the calibration pressures to give:

$$p_0 = 1.05\,(A - z_m + \Delta A) - 0.05\,(B - z_m - \Delta B) \qquad (9.46)$$

$$p_1 = B - z_m - \Delta B \qquad (9.47)$$

$$p_2 = 1.05\,(C - z_m + \Delta A) - 0.05\,(B - z_m - \Delta B) \qquad (9.48)$$

where p_0 = corrected pressure on the membrane before lift-off (i.e. at 0.00mm expansion), p_1= corrected membrane pressure at 1.10 mm expansion, p_2 = corrected pressure at which the membrane just returns to its support after expansion, A = recorded A-pressure reading in soil (at 0.05mm), z_m = gauge pressure reading (error) when vented, ΔA calibration pressure recorded at 0.05 mm membrane expansion in air (a positive value), B = recorded B-pressure reading in soil (at 1.10 mm membrane expansion), ΔB = calibration pressure recorded at 1.10 mm membrane expansion in air (a positive value), and C = recorded C-pressure, at the point at which the audible signal returns during controlled deflation.

The corrected C-pressure can give a measure of the in situ pore pressure, u, in free-draining granular soils, or in sand layers within clays (I_D (see below) >2, approximately). In other soils the initial in situ pore pressure (i.e. before insertion of the dilatometer) will require estimation.

The quasi-static dilatometer penetration resistance (q_D) is obtained from:

$$q_D = \frac{P_D}{A_D} \qquad (9.49)$$

where P_D = measured penetration force, and A_D = plan area of the dilatometer (95mm x 14mm = 13.3 cm^2, as compared with the CPT plan area of $10cm^2$). Approximately, q_D can be expected to equal the CPT cone resistance, q_c.

From an estimate of the bulk density profile and the in situ pore pressure before DMT penetration, the in situ vertical effective stress ($\sigma'_v = \sigma_v$ - u) is calculated. Then four DMT indices are calculated.

1. Material index (a normalized modulus which varies with soil type):

$$I_D = \frac{(p_1 - p_0)}{(p_0 - u_0)} \tag{9.50}$$

2. Horizontal stress index (a normalized lateral stress):

$$K_D = \frac{(p_0 - u_0)}{\sigma'_v} \tag{9.51}$$

3. Dilatometer modulus (an estimate of elastic Young's modulus):

$$E_D = 34.7(p_1 - p_0) \tag{9.52}$$

4. Pore pressure index (a measure of the pore pressure set up by membrane expansion):

$$U_D = \frac{(p_2 - u_0)}{(p_0 - u_0)} \tag{9.53}$$

Results, interpretation and use

A typical result from a DMT sounding is shown in Fig. 9.33. Plots of the main dilatometer indices and the dilatometer modulus are given as a function of depth. Results are normally processed on a portable computer (for example, using a spreadsheet program) and therefore can be rapidly made available for use in engineering decisions and designs.

In their relatively short life, dilatometer results have become used in a large number of applications:

SOIL PROFILING AND IDENTIFICATION

Marchetti and Crapps (1981) provided the soil identification chart shown in Fig. 9.34. A particularly promising method of identifying shear surfaces below landslides in overconsolidated soils has recently been proposed by Totani (1992).

DETERMINATION OF SOIL PARAMETERS

The DMT can be used to estimate unit weight (Marchetti and Crapps 1981; see also the soil identification chart in Fig. 9.34), undrained shear strength (Marchetti 1980; Lacasse and Lunne 1983; Rogue *et al.* 1988), effective angle of friction (Schmertmann 1982; Marchetti 1985), see Fig. 9.35, drained constrained modulus (Marchetti 1980), elastic modulus, and the very small-strain shear modulus, G_{max}.

In clays, the undrained shear strength can be estimated from a form of the bearing capacity equation:

$$s_u = \frac{(p_1 - \sigma_{h0})}{N_D} \tag{9.54}$$

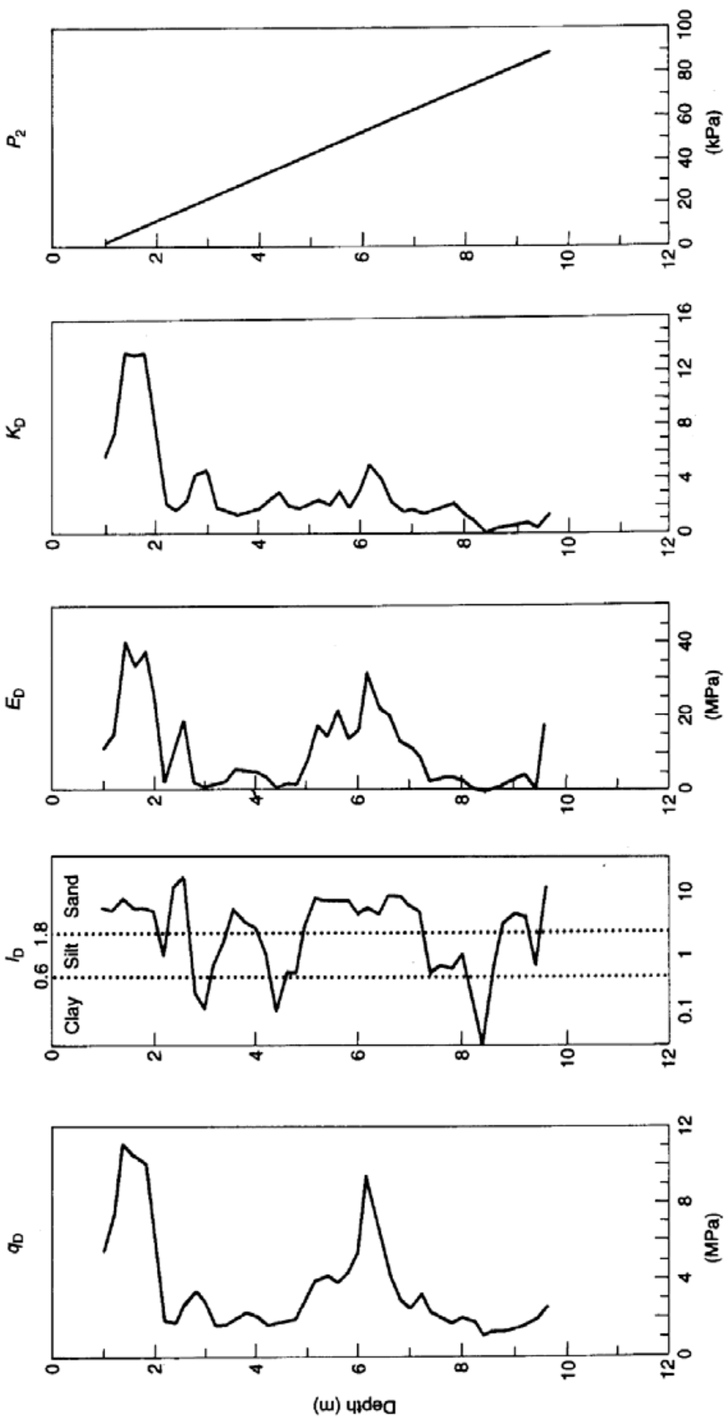

Fig. 9.33 Result of DMT sounding in clayey silts and silty sands (data from Schmertmann 1986).

Rogue *et al.* (1988) have proposed the ND values given in Table 9.8.

Table 9.8 Values proposed by Rogue *et al.* (1988)

Soil type	N_D
Brittle clay and silt	5
Medium clay	7
Non-sensitive plastic clay	9

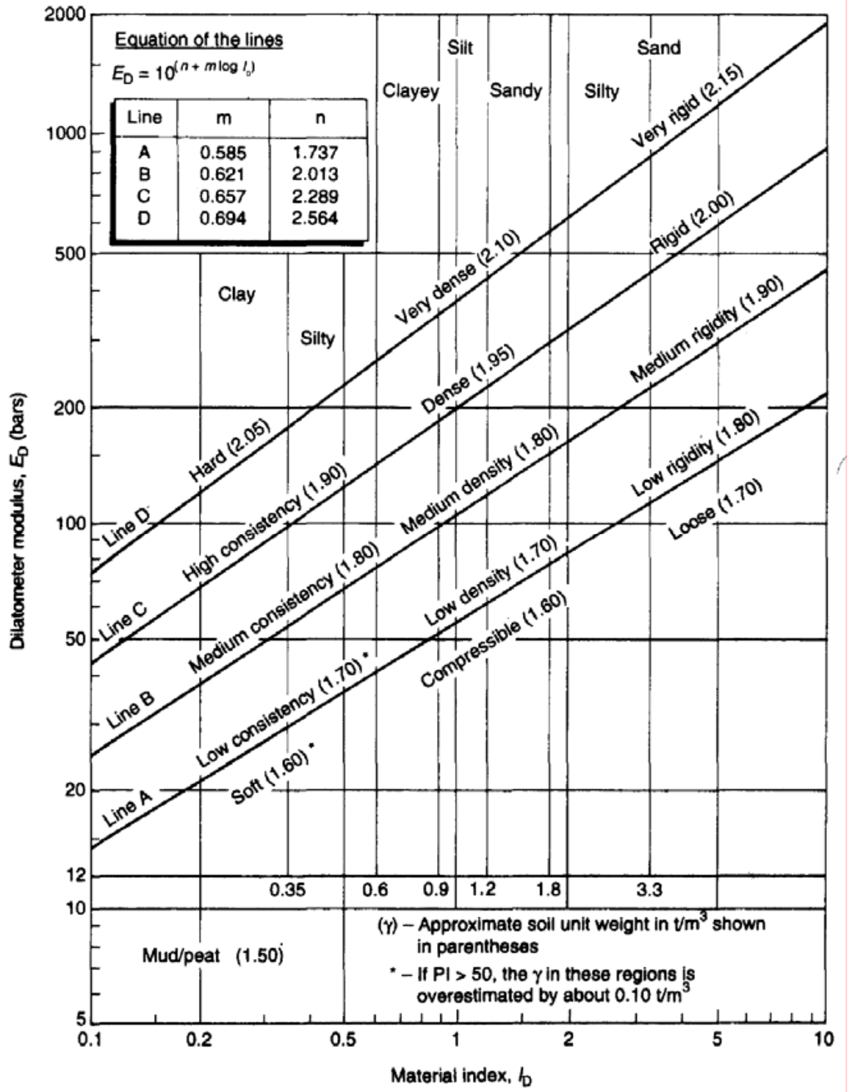

Fig. 9.34 Chart for determination of soil description and unit weight (from Marchetti and Crapps 1981).

In sands, the drained constrained modulus (M) can be obtained from the expression:

$$M = R_M E_D \tag{9.55}$$

where E_D = dilatometer modulus, and R_M = coefficient given as a function of the horizontal stress index, K_D.

Marchetti (1980) gives values of R_M according to I_D. Leonards and Frost (198k) found that Marchetti's values are too low, and suggested factoring these up, but Marchetti (1991) has subsequently argued against this. In clays, Lunne *et al.* (1989) recommend the use of Marchetti's (1980) correlation.

ESTIMATION OF IN-SITU PORE PRESSURE AND HORIZONTAL STRESS

Lutenegger and Kabir (1988) and Robertson *et al.* (1988) have found that in sands the p_2 pressure is equal to the in situ pore pressure, u. This is because in the minute or so after loading, sufficient drainage occurs to re-establish equilibrium pore pressures. This will not be the case in clays or other slower-draining soil types.

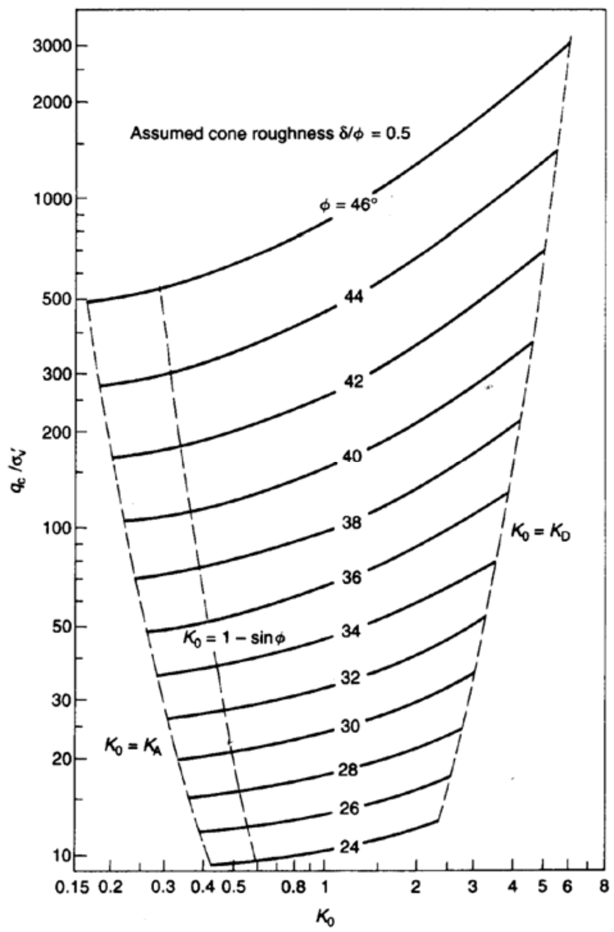

Fig. 9.35 Chart for determining effective angle of friction from CPT q_c and K_0 (from DMT data) (Marchetti 1985).

In normally consolidated, young clays Lunne (1990) has proposed that, for $K_D < 4$:

$$K_0 = 0.34 K_D^m \tag{9.56}$$

where m is a coefficient varying from 0.44 (high-plasticity clay) to 0.64 (low-plasticity clay).

Briaud and Miran recommend that this equation be used for soft and medium to stiff clays having $I_D \leq 1.2$ and $K_D < 4$. Lunne *et al.* (1990) and Powell and Uglow (1988) have shown that the correlation between K_D and K_0 are different for young and for old clays.

In sands, correlations been K_D and K_0 have been proposed by Schmertmann (1983) and Marchetti (1985). Schmertmann's method is complex. Marchetti's method (Fig. 9.36) requires an estimate of q_c, which he suggests should be obtained from a nearby CPT profile. However, because q_D is similar to q_c, it is suggested that this can be used.

DESIGN

Lutenegger (1988) has compiled a list of reported design applications using DMT data, and this has been further added to by Briaud and Miran (1992). A compilation is given in Table 9.9.

In general, the assessments of the accuracy of predictions that have been made,, for example by Lutenegger (1988), show that the DMT is a promising tool. It must be remembered, however, that correlations between its results and the various soil parameters that it produces are site specific.

Interpretation should be carried out with care, particularly when new or unfamiliar ground conditions are encountered. No doubt, as our database grows, these considerations will become less significant.

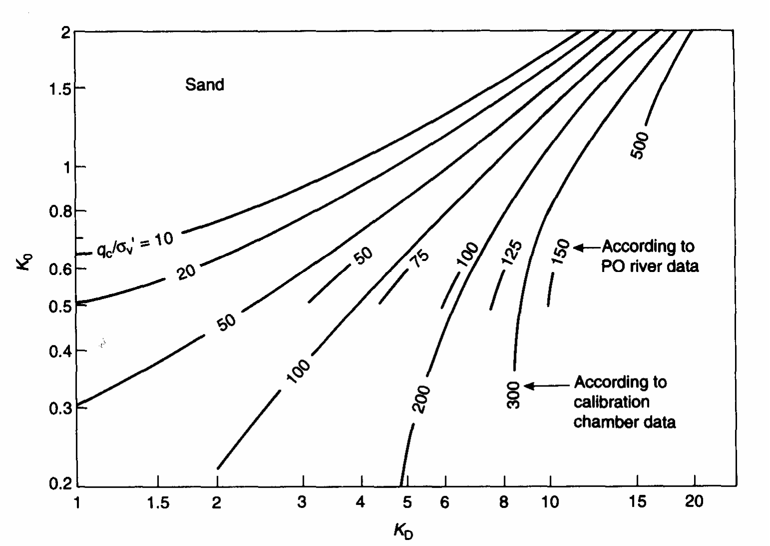

Fig. 9.36 Correlation between K_0 and DMT K_D for sand (Marchetti 1985).

Table 9.9 Design using the Marchetti dilatometer (DMT)

Design application	Reference
Settlement prediction	Schmertmann (1986), Hayes (1986), Saye and Luttenegger (1988), Briaud and Miran (1992)
Bearing capacity of shallow foundations	Briaud and Miran (1992)
Laterally loaded piles	Schmertmann and Crapps (1983), Robertson *et al.* (1988), Briaud and Miran (1992)
Skin friction on axially loaded piles	Marchetti *et al.* (1986)
Liquefaction potential of sands	Marchetti (1982), Robertson and Campanella (1986)
Compaction control	Schmertmann (1982), Schmertmann *et al.* (1986), Lutenegger (1986), Lacasse and Lunne (1986)
Ultimate uplift of anchor foundations	Lutenegger *et al.* (1988)
Transmission tower foundation design	Bechai *et al.* (1986)
End bearing, side friction and settlement of drilled shafts	Schmertmann and Crapps (1983)
Assessment of pre-existing slope instability	Totani (1992)

PERMEABILITY TESTING

The permeability of a soil can only rarely be obtained with sufficient accuracy from laboratory tests on specimens from normal diameter boreholes, and therefore the in situ permeability test is common.

In situ permeability tests can be carried out in soils or rocks, in open boreholes, in piezometers, or in sections of drillhole sealed by inflatable packers. The three most common types of test, which are considered in this chapter, are:

1. rising and falling head tests;
2. constant head tests; and
3. packer or Lugeon tests.

Rising or falling head tests

The rising or falling head test is generally used in relatively permeable soils. It is usually carried out in a cased borehole or a simple piezometer such as the Casagrande low-air entry open-tube type. Where the groundwater level exists above the base of the borehole, the water level in the borehole or piezometer tube may either be reduced or increased. Water level measurements are then taken at suitable time intervals until the water level returns to equilibrium (see Fig. 9.37).

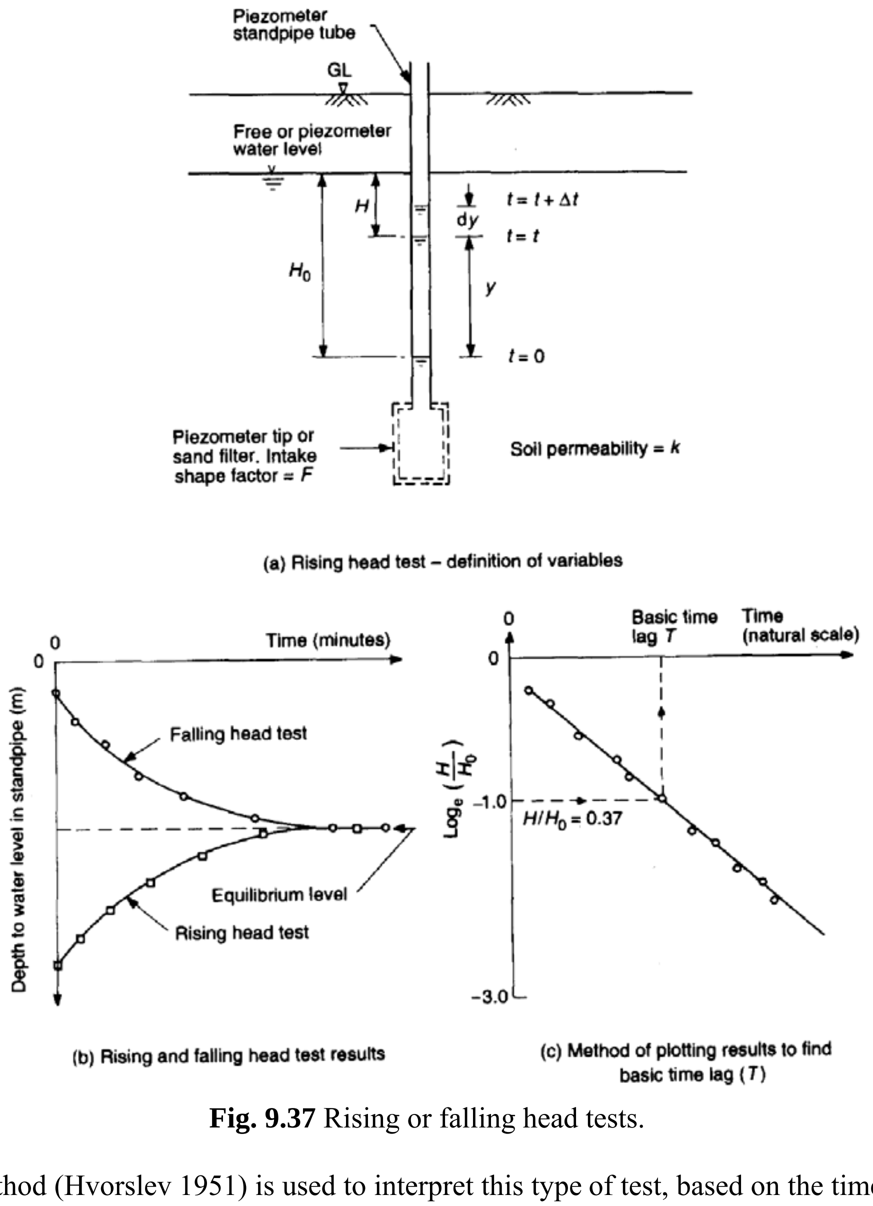

Fig. 9.37 Rising or falling head tests.

Hvorslev's method (Hvorslev 1951) is used to interpret this type of test, based on the time lag required for water pressures to equalize.

Assumptions

Soil does not swell or consolidate. Other test errors, such as those due to air in the soil or pipes, do not occur. There is no smear. At time t, the driving head = H. Therefore, from Darcy's law the rate of flow into the piezometer is given by:

$$q = FkH = Fk(H_0 - y) \tag{9.57}$$

where F= piezometer shape factor and k = coefficient of permeability of the soil (see Fig. 9.37a).

In small time, Δt, the volume of flow into the piezometer tip equals the volume entering the standpipe:

$$qdt = Ady \tag{9.58}$$

therefore, combining with eqn. 9.57:

$$\frac{dy}{H_0 - y} = \frac{Fkdt}{A} \tag{9.59}$$

Hvorslev introduced the concept of basic time lag. This is the time that would be taken for equilibrium to be established if the initial flow rate were maintained throughout the test. (In fact, since the head is reduced by the flow, the rate of flow is progressively retarded during the test.)

For constant groundwater or piezometric level, the basic time lag is defined as:

$$T = \frac{V}{q_{t=0}} = \frac{AH_0}{FkH_0} = \frac{A}{Fk} \tag{9.60}$$

therefore:

$$\begin{gathered} \frac{dy}{H_0 - y} = \frac{dt}{T} \\ \int_0^{H_0 - H} \frac{dy}{H_0 - y} = \int_0^t \frac{dt}{T} \; gives \; \frac{t}{T} = \log_e\left(\frac{H_0}{H}\right) \end{gathered} \tag{9.60}$$

where (t/T) is the time lag ratio and $(H/H_0)[=e^{-t/T}]$ is the head ratio.

In order to determine the coefficient of permeability, the time factor, T, must be found. One simple method which can be widely applied is shown in Fig. 9.37c.

When the time equals the basic lag, then:

$$\frac{H}{H_0} = e^{-1} = 0.368 \tag{9.61}$$

If $\log_e (H/H_0)$ is plotted as a function of time, the basic time lag can be found from the straight line at $\log_e (H/H_0) = -1.0$.

This method requires a knowledge of the stabilized water level, in order to find H_0. In soils of low permeability, the test may take so long that H_0 cannot be found. Obviously, the equalization time is a function of the volume required to reduce the driving head to zero. Hvorslev (1951) quotes times to 90% equalization on which the figures in Table 9.10 are based.

Where in situ tests are carried out, but the groundwater or piezometric level cannot be determined it may be found by inserting trial values of H_0 in the above equations, and repeatedly plotting the graph of loge (H/H_0) vs. time. When the correct value of H_0 is inserted, a straight line will result: incorrect values yield curves.

Table 9.10 Times to 90% equalization (based on Hvorslev (1951)) in hours

Piezometer type Coefficient of permeability(cm/s)	Soil type						
	Sand			Silt		Clay	
	10^{-2}	10^{-3}	10^{-4}	10^{-5}	10^{-6}	10^{-7}	10^{-8} k
204 mm dia. borehole (flat bottomed)	0.37	3.69	36.90	369.00			
Casagrande piezometer l50mmdia. X 9l4mm long with 10mm bore standpipe			0.02	0.22	2.20	22.00	220.00
Closed hydraulic piezometer with 100 m of tubing				0.03	0.25	2.50	25.00

Once H_0 is known, the shape factor must be calculated to allow the coefficient of permeability to be determined from the basic time lag. Hvorslev (1951) gives shape factors for a variety of geometries, but during most site investigations only a few cases are normally used (Table 9.11).

Table 9.11 Shape factors (from Hvorslev (1951))

Geometry	F
Cased borehole (diameter, D) soil flush with bottom of casing, in uniform soil	2.75D
Cased borehole, soil flush with bottom of casing. Soil above base of hole impermeable	2D
Cased borehole, with uncased length L, in uniform soil, or cylindrical piezometer	$\dfrac{2\pi L}{\log_e\left[\dfrac{L}{D}+\sqrt{\left(1+\left(\dfrac{L}{D}\right)^2\right)}\right]}$

If F is known, then the coefficient of permeability, k, can be found because:

$$k = \frac{A}{FT} \tag{9.62}$$

Constant head testing

Constant head testing is required in all soils where stress changes will result in significant consolidation or swelling. When clay is subjected to an in situ permeability test the effective stresses in the soil are modified by the increase in pore water pressure normally applied. As the soil swells is takes in water, and thus test records normally indicate a higher permeability than, in fact, exists.

Gibson (1963, 1966, 1970) and Wilkinson (1968) have considered the use of the constant head test in clay strata. The object of the test is to find the rate of flow under steady seepage conditions, after swelling has occurred.

Under constant head conditions, the rate of water flow (q) at various times (t) after the test start is plotted as a function of ($1/\sqrt{t}$), (see Fig. 9.38). As time passes swelling reduces and q decreases. After some time it should be possible to extrapolate to find the rate of flow at infinite time ($q_{t=\infty}$), the steady flow. The test results may plot concave up or down, depending on the A value of the soil (Gibson 1966), and generally they will not give a straight line on the ($1/\sqrt{t}$) plot.

The coefficient of permeability may be found from Hvorslev's equations. For example for a cylindrical piezometer:

$$q = \frac{2\pi LkH}{\log_e\left[\frac{L}{D} + \sqrt{\left(1 + \left(\frac{L}{D}\right)^2\right)}\right]} \tag{9.63}$$

Alternatively, Maasland and Kirkham (1959) have proposed:

$$q = \frac{3\pi LkH}{\log_e\left[\frac{1.5L}{D} + \sqrt{\left(1 + \left(\frac{1.5L}{D}\right)^2\right)}\right]} \tag{9.64}$$

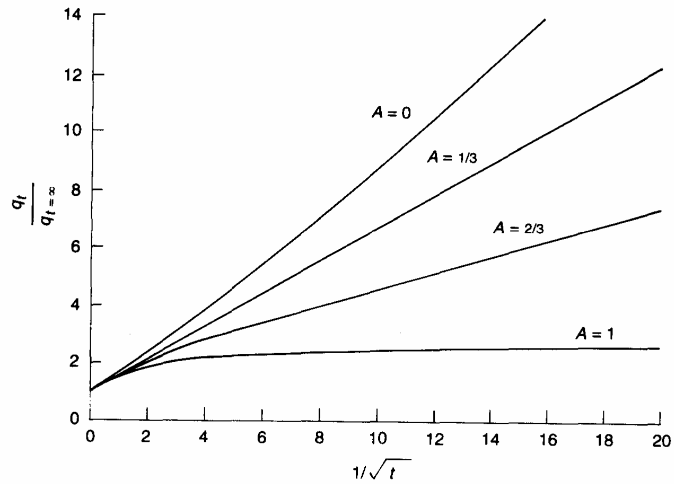

Fig. 9.38 Effect of Skempton's 'A' value on in-situ permeability test results (Gibson 1970).

It has been suggested that coefficient of consolidation values can theoretically be obtained from the slope of the q vs. ($1/\sqrt{t}$) curve, but since this will typically be curved (depending on the A value, see Gibson (1970)), it will be better to obtain them by combining coefficient of permeability values with coefficient of compressibility values obtained from laboratory tests at the same effective stress levels.

Gibson (1966) has considered the effect of the permeability of the piezometer tip and any surrounding filter sand on the measured value of soil permeability. In the above equation, it is assumed that the permeability of the piezometer installation is infinite, and Gibson has concluded on theoretical grounds that this assumption will be reasonable only if the piezometer ceramic and any surrounding filter sand are at least ten times more permeable than the surrounding soil. Gibson gives two examples of soil permeability limits as shown in Table 9.12.

Table 9.12 Soil permeability limits

Type of high air entry ceramic	Max. k soil (m/s)
Aerox 'Cellaton' Grade 6 ceramic	10^{-9}
Doulton Grade P6A ceramic	10^{-10}

Wilkinson (1968) has considered the effects of smear and trapped air on the results of the test. Air trapped in the piezometer pocket or leads during piezometer construction may lead to high initial flow rates, but should not seriously effect predictions based on $q_{t=\infty}$. On the other hand, smear may have a very serious effect, particularly where the piezometer is pushed directly into the soil and no sand pocket is used. This means that 'drive-in' piezometers may not be successfully used to determine in situ permeability in soils exhibiting fabric.

Further errors may arise due to leakage past grout seals used to isolate the top of the sand pocket from the upper part of the borehole. Vaughan (1969) considers that leakage effects are only of major consequence when the soil permeability is low, and of the order of 10^{-10}m/s, or less.

The use of high pressures during constant head tests may lead to 'hydraulic fracture', a process whereby the water pressures rise to such a level that they exceed the in situ total stresses. In theory if the soil is normally consolidated (i.e. $K_0 < 1$) vertical cracks will be formed, but in heavily overconsolidated soil cracking will be horizontal because vertical stress levels are smaller than those on the vertical plane (Bjerrum *et al.* 1972).

The following maximum increases in water pressure are suggested:

In situ coefficient of earth pressure at rest (K_0)	0.3	0.5	0.7	1.0
$Max.\left(\dfrac{increae\ of\ water\ pressure}{initial\ effective\ stress}\right)\left(\dfrac{\Delta u}{p_0}\right)_{max.}$	0.4	0.7	0.9	1.0

When the water pressure increases above these levels cracks develop in the soil, and the apparent permeability rapidly rises through several orders of magnitude, giving totally misleading results (Fig. 9.39).

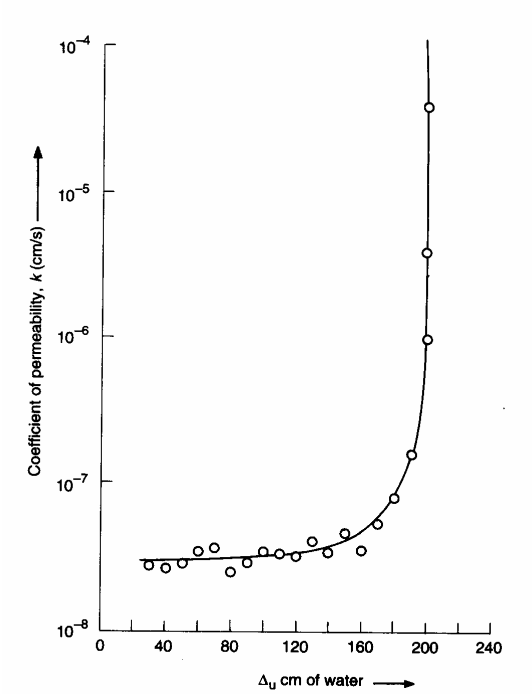

Fig. 9.39 Hydraulic fracture leading to permeability increase (Bjerrum *et al.* 1972).

Of course, the constant head permeability test requires a field apparatus to provide the constant head, and to measure the rate of flow. Two configurations are in use, the more sophisticated using a water cylinder pressurized by an air/water bladder, and flow measurement by variable area conical float flowmeters. Constant air pressure is supplied via a 12 V electrical compressor and a diaphragm-type pressure regulator. This type of apparatus is only suitable for measuring permeabilities in certain restricted ranges, depending on the specific design, and in addition problems may occur where the test has to be continued for a long period and the total flow volume exceeds 3—4 litres.

A simpler but less precise method of test may easily be built as shown in Fig. 9.40 which basically consists of a large (501) polythene water container, connected by push couplings and a tap to a 100cc glass burette. The head is maintained constant by topping up the polythene drum to a mark, and flow measurement is achieved by turning off the tap and noting the volume change in the burette over a measured time. This type of test is theoretically less accurate than using the more sophisticated type, because the head does not remain constant. It has the advantage however, that large bore pipes can be used throughout, thus allowing measurement of a wide range of permeabilities. With other systems, the restrictions in the constant pressure system may sometimes be greater than the effect of soil permeability.

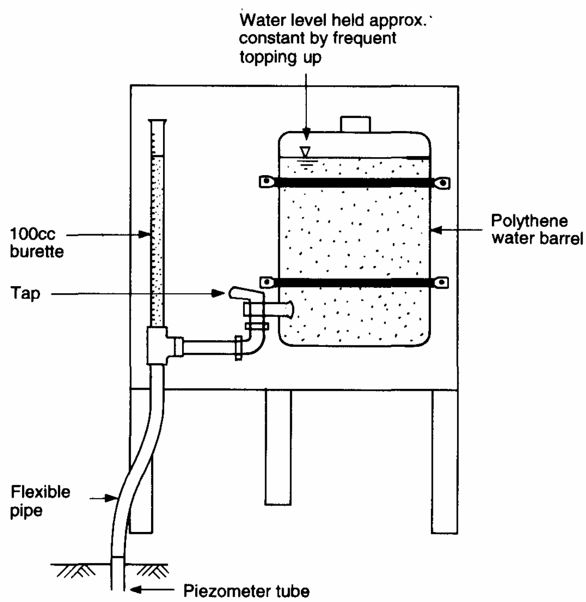

Fig. 9.40 Simplified field constant head apparatus.

The packer or 'Lugeon' test

The rock equivalent of the soil constant head permeability test is the packer test, also sometimes known as the Lugeon test (Lugeon 1933). The test may be carried out in the base of a drillhole using a single inflatable packer to seal off the test section, or after the hole is complete, testing may be carried out at a variety of depths using a double packer to seal the test section top and bottom.

The construction of the packers is critical if leakage is to be avoided, and the longer the packers used, the more effective will be the test. Details of the construction of packers developed at Imperial College, London, may be found in Harper and Ross- Brown (1972), Hoek and Bray (1974), and Pearson and Money (1977).

The test is carried out by lowering the packer or packers to the required depth and inflating them using gas pressure supplied from a nitrogen bottle. The length of each packer should be at least five times the borehole diameter, when expanded; recent researchers have used length to diameter ratios of between 20 and 40. The test section is often about 3m long.

The packers are supported on drill rods, which are also used to supply water under pressure to the test section, and at the top of the borehole the rods are connected via a water swivel or 'gooseneck' to a 'Christmas tree' and flush pump (Fig. 9.41).

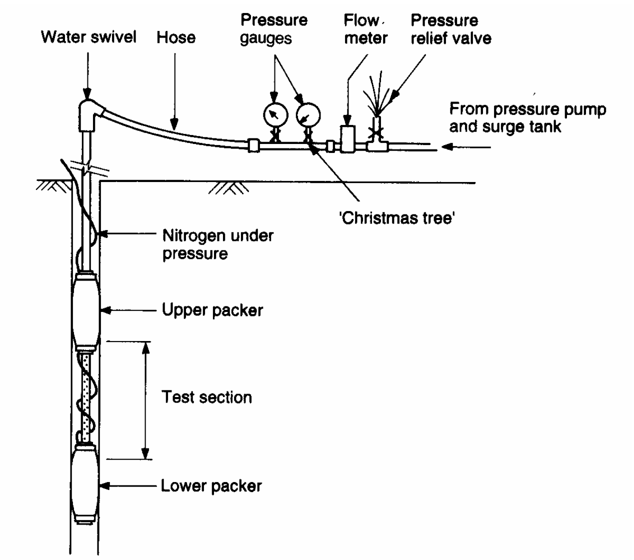

Fig. 9.41 Basic equipment for the double packer test.

The flush pump is capable of producing water at high pressure, but even though this type of pump is often fitted with a gearbox the rate of flow cannot be controlled sufficiently to allow a constant pressure to be applied to the test section. In addition, many pressure pumps run on the piston principle and the output pressure varies with the position of the piston. The 'Christmas tree' therefore needs to include a pressure relief valve, and may contain a surge tank to smooth the pressure from the pump. In addition, at the end coupled to the swivel hose one or two Bourdon pressure gauges and a volumeter are included to allow the measurement of water flow and pressure in various ranges.

The test is carried out in stages, being cycled up to a maximum head and then down again. Typically a maximum head is specified to avoid hydraulic fracture. The allowable net dynamic head (H_t) is often specified as:

$$\frac{overburden\ pressure\ at\ test\ depth}{unit\ weight\ of\ water} = H_t$$

The test is normally carried out using stages such as 1/3, 2/3, 1, 2/3 and 1/3 of the maximum allowable gauge pressure on the 'Christmas tree'. At each pressure stage, the pressure is held constant and the volume measured over a period of 5 min. If the volume measured over two consecutive 5 min periods differs by more than 10%, then measurement should be made for a further 5min period before the pressure is changed. The permeability is calculated from the volume of flow and the net dynamic head applied to the test section.

The net dynamic head (H_t) is:

$$H = (H_p + H_m + H_w) - H_c \tag{9.65}$$

where H_p = pressure head (from the pressure gauge), H_m = head due to the height of the pressure gauge above the ground level at the top of the drillhole, H_w = distance to the groundwater from the top of the drillhole, and H_c = head loss in the test equipment.

H_c, the head loss, must be obtained by calibration of every piece of equipment between the pressure gauge and the test section as a routine before each test. The flexible swivel hose, the swivel, rods and perforated section should be connected, laid out on the ground, and tested by pumping water at different rates while recording the pressure required to sustain the flow. The results may be plotted either with H_c as a function of flow (Fig. 9.42), or H_c as a function of packer stem length. Failure to calibrate properly may lead to errors in permeability of about one order of magnitude, particularly in highly permeable rocks. In rocks with a permeability of less than 1 x 10^{-7} m/s head losses in the equipment are not likely to be significant.

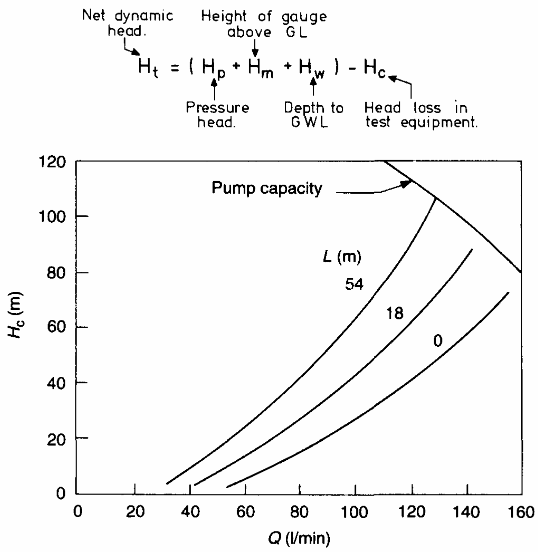

Fig. 9.42 Calibration curves for packer test equipment with various rod lengths (Dick 1975).

The formulae for determining the coefficient of permeability from packer test results are given in the United States Bureau of Reclamation *Earth Manual* (1963) as:

$$k = \frac{Q}{2\pi LH_t}\log_e\left(\frac{L}{r}\right) \qquad \text{for } L \geq 10r \qquad (9.66)$$

and

$$k = \frac{Q}{2\pi LH_t}\sinh^{-1}\left(\frac{L}{2r}\right) \qquad \text{for } 10r > L \geq r \qquad (9.67)$$

where k = permeability, Q = constant rate of flow into the hole, L = test length, and r = radius of hole tested.

Hoek and Bray (1974) also give the solution for tests carried out with rock joints normal and parallel to the length of the test section. In this case:

$$k = \frac{\left(\frac{mL}{r}\right)}{\frac{Q\log_c}{2\pi LH_t}} \qquad (9.68)$$

where m = $(k/k_p)^{½}$, k = permeability at right angles to borehole, and k_p = permeability parallel to the borehole, which if cross-flow is ignored equals the intact rock permeability. For most applications, Hoek and Bray consider a reasonable value of k/k_p to be 10^6, whence m = 10^3. This has the effect of increasing the value of the permeability calculated from the USBR equations by about half an order of magnitude. Interpretation in terms of the 'Lugeon coefficient' is less contentious in deep deposits since this is, by definition, 'the water absorption measured in litres per metre of test section per minute at a pressure of 10 kg/cm^2 (= 1000 kN/m^2).

The pressure restrictions necessary to prevent hydraulic fracture in shallow deposits, however, require extrapolation of low pressure test results. Thus:

$$\frac{Q_2}{Q_1} = \left(\frac{p_2}{p_1}\right)^n \tag{9.69}$$

where Q = rate of flow caused by a pressure p, where n = 1 for laminar flow or 0.5 for turbulent fissure flow. Lancaster-Jones (1975) concludes that under normal conditions, flow tends to be turbulent.

Where cycled tests are performed, usually with an ABCBA pressure pattern, results are sometimes presented graphically either as: (1) a head/permeability diagram; or (2) a head/flow ('Lugeon') diagram.

Examples of the interpretation of these diagrams are given by Lugeon (1933), Little *et al.* (1963), Morgenstern and Vaughan (1963), Muir Wood and Caste (1970), Dick (1975) and Pearson and Money (1977). In practice, results typically fall into three groups (Fig. 9.43).

1. The 'ideal' case: Darcy's law dictates that flow will be directly proportional to pressure, and therefore predicts a horizontal line on the head/permeability plot. On the Lugeon diagram, a straight line passing through the origin should be found.
2. Permeability appears to rise with increasing pressure.
3. Permeability appears to fall with increasing pressure.

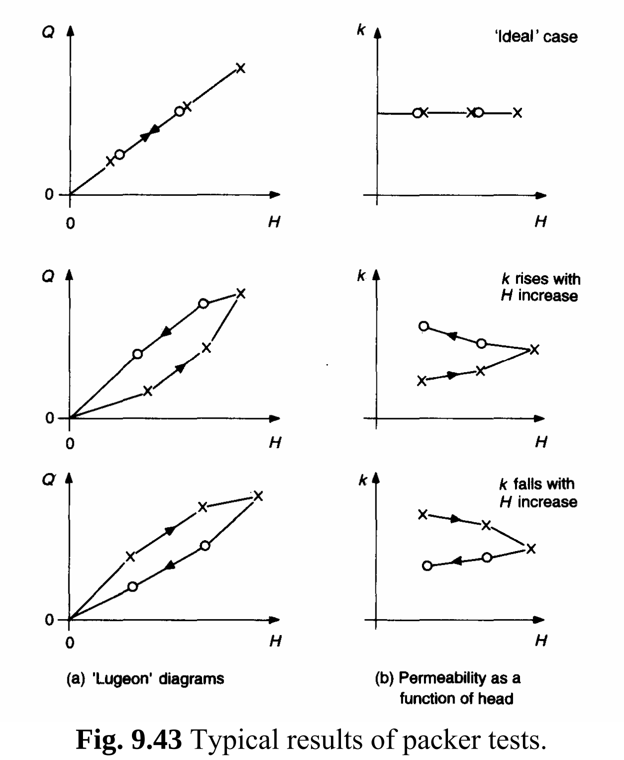

Fig. 9.43 Typical results of packer tests.

Pearson and Money (1977) have observed that these anomalies can also be divided into two other groups: the effects of flow of water in the rock mass, and effects of the test system or technique. The former are unavoidable, but the latter can usually be overcome by careful test technique, coupled with improved instrumentation.

The effects of water flow may lead to either increasing or decreasing permeability with increasing pressure. Increasing permeability will result from erosion of fractures, or dilation as a result of test water pressures. This latter effect can lead to very large increases in apparent permeability if test pressures rise to hydraulic fracture levels. Decreasing permeability is normally associated with either turbulent flow, or siltation or clogging of fissures. Fissure siltation may occur as a result of migration of fines within the rock mass, but is often associated with the use of dirty test water.

A further effect which appears to give decreasing permeability during the decreasing pressure stages of a test has been described by Little *et al.* (1963) as a 'back pressure' effect. The increasing pressure stages act to charge the rock fissures with high pressure water which reduces the head drop between the test section and the rock mass; flow in the later stages of the test, when the applied head is being reduced, leads to apparently low permeabilities because the true head gradient cannot be assessed. There is little point in conducting head decrease stages if this effect is observed.

The effects of test system and technique have been partly discussed in previous sections. Quite clearly packer leakage and the use of dirty water are highly undesirable, and the losses in the system must be assessed if head measurement is to take place at the top of the hole. In addition to these problems, basic systems such as are shown in Fig. 9.41 suffer from a variety of other defects.

1. *Long-term surging.* The use of piston pumps will lead to very rapid surging, as noted above, but long-term surging can also occur if petrol engines are used to drive pumps.
2. *Air injection.* Slightly faulty suction hosing in the pump system may lead to air being pushed into the test section.
3. *Flow measurement.* Basic systems typically use either a reciprocating chamber or an impeller to activate a mechanical counter. These devices measure total flow, rather than rate of flow and cannot detect sudden changes in flow rate which may indicate the onset of faults such as packer leakage. In addition, such devices tend to stick at low flow rates.

In addition to the effects above, the action of forming the drillhole may lead to a considerable amount of smear over the test section, particularly in soft argillaceous rocks. Under these conditions, 'pumping-in' tests may be expected to yield much lower values of permeability than tests which are based on extraction (such as the rising head test). A further complicating effect arises when tests are conducted above the groundwater level. The fracture pattern of the rock mass gives it a storage capacity which leads to initially high flows, and these flows may take a very considerable time to level off.

The packer test in its present usage is undoubtedly far from perfect, and the equipment is rather difficult to use successfully. Various authors have advocated the use of electronic measuring equipment to measure the actual pressure in the test section and the rate of flow (for example, Pearson and Money (1977)). It is likely that the introduction of such equipment would bring considerable advantages, but the added complexity of the equipment would undoubtedly increase both the cost and difficulty of the test.