Chapter 8

Laboratory testing

INTRODUCTION

Laboratory testing is part of the physical survey. As an integral part of site investigation, the need for laboratory tests will often dictate the type and frequency of sample to be taken, and will therefore control the method of forming boreholes. Thus the type of sampling requires a precognition of the soil conditions on site; this has had the effect of leading some writers to recommend at least two stages of field work, with the bulk of laboratory testing being carried out after specific sampling in the second phase of investigation. For routine work such a programme is impractical and rarely used, because of the increases in cost and time that it causes. If two phases of site and laboratory work cannot be included then the investigation must be more carefully planned. With provision for changes during field work, with close engineering supervision and with a knowledge of soil conditions on site based on a first-class desk study, it should be possible to avoid the use of two field investigations.

Soil mechanics, although a -branch of engineering, is often imprecise. Since many problems cannot be solved with accuracy, either as a result of imperfect analytical techniques or complex ground conditions, the use of refined sampling and testing techniques has been questioned. Terzaghi and Peck (1948) have commented ' ... On the overwhelming majority of jobs no more than an approximate forecast is needed, and if such a forecast cannot be made by simple means it cannot be made at all'. But is this attitude always justified?

Certain classes of structure are so costly and the consequences of their failure so serious that, whatever the soil conditions, no effort should be spared in making as accurate a prediction of performance as possible. Where routine jobs are concerned, individual judgement based on low-cost sampling and testing may well suffice in the majority of cases, but such a method has a serious drawback; it does not allow extension of engineering knowledge based on observation and comparison with good quality data. Routine jobs are much more numerous than those for which the cost and time required for accurate and specialist testing can be justified, but can an engineer afford not to develop his experience and can he now afford the consequences of failure? Brunel and Stephenson could do so, for in their day experimental data were almost non-existent in the field of soil mechanics and it could be expected that the almost exclusive use of personal judgement would inevitably lead to some failures. We can no longer enjoy such luxury.

When making predictions about the behaviour of soil, two factors are most important. First, it is normally necessary to judge which elements of soil behaviour will be critical to the satisfactory performance of the structure. Since there are many different ways in which soil behaviour can adversely affect the performance of a structure, it is necessary to appreciate all those facets which may cause problems and then analyse each, however briefly, to determine which are the most critical. Secondly, it is important to appreciate the limits which can be placed on any aspect of soil behaviour, for example, what settlement is tolerable, and is this the total or differential movement? For example, when considering the suitability of a site for spread footings for a multistorey structure it might be necessary to look at the following aspects of design:

1. overall slope stability after the end of construction;
2. stability of temporary slopes during foundation construction;
3. temporary support requirements;
4. amount of seepage inflow into excavations;

5. effects of seepage and loss of ground on adjacent structures;
6. settlement of surrounding ground due to groundwater lowering;
7. maximum allowable foundation bearing pressure;
8. predicted settlements of footings;
9. time for consolidation to occur; and
10. proposed dimensions and layout to keep differential settlements small.

In any one case, it is probable that only a small proportion of these problems would require the acquisition of soil parameters for solution. The quality of the data required would depend on the allowable limits set for the structure. Thus spread footings in weathered rock would not normally experience significant settlements, but if a raft with very little tolerance of differential settlement were considered then even these conditions might give difficulties. Examples of the very small tolerance to differential settlements of sugar silo raft foundations, where doming of 5 mm over a 23 m foundation diameter was the limit to avoid structural distress, have been considered by Burland and Davidson (1976), Kee (1974) and Connor (1980).

Two factors affect the quality of soil testing data required for a satisfactory prediction of soil behaviour. The tests carried out must be appropriate for the acquisition of the required data, or their results must be empirically linked to the required soil parameters with sufficient precision for the required calculation. In addition, sampling and testing must be carried out using techniques and accuracy which will yield parameters which are representative of the bulk of the soil *in situ*. Bearing in mind the small proportion of the on-site soil which will be sampled, (Broms,(1980) suggests 1 in 1000000 by volume), it will never be feasible to obtain representative parameters when soil conditions are variable, however good or expensive the sampling and testing techniques. Under these circumstances only simple laboratory tests should normally be considered, anti field tests may provide more useful data.

THE PURPOSE OF SOIL TESTING

In general, soil is tested in order to assess its variability and in order to obtain parameters for particular geotechnical calculations. These two distinct reasons for testing lead to very different testing programmes. Routine tests carried out to allow the soil on a site to be divided into groups should ideally be scheduled for an initial phase of testing. Subsequent more expensive and complex tests are normally carried out on soil which is thought to be representative of each group; the samples to be tested cannot be so well selected before the results of classification tests are known. For reasons of time and economy, this ideal scheme cannot normally be used. More complex tests require a longer test period. When testing is started at about the same time as samples start to arrive from site, the engineer initially may have to rely completely on soil descriptions for a division of the *in situ* soil.

Soil classification is carried out in order to define a small number of different groups of soil on any site. Each soil group may consist of a stratigraphically defined geological unit. More often it may ignore geological boundaries because the essence of the soil group should be that materials within it have (or are expected to have) similar geotechnical properties. Particle size, plasticity and organic content may be more important to the geotechnical engineer than time of deposition. The three main tools used to classify soil are soil description, particle size distribution analysis and plasticity testing.

AVAILABLE TESTS

This chapter sets out to describe individual test techniques in detail: texts such as Lambe (1951), Bishop and Henkel (1962), Akroyd (1964), Vickers (1978), Head (1980), Head (1982), Head (1986), BS 1377:1990 and ASTM Part 19, should be referred to for the methods used in each test. Soil tests are loosely brought into two groups in this section; the first provides information to allow the classification of soil into arbitrary groups while the second includes all tests which provide parameters which may be used in geotechnical calculation and design (Table 8.1).

Table 8.1 Soil classification tests and test parameters

Soil classification tests	Tests for geotechnical parameters
Sample description (Discussed in Chapter 2)	Strength tests
Particle size distribution tests	Stiffness tests
Plasticity tests	Consolidation tests
Compaction tests	Seepage and permeability tests
Specific gravity tests	

This division is not conventional. Normally plasticity tests, particle size distribution and specific gravity tests are know as soil classification tests (for example, see Head (1980) or BS 1377:part 2:1990).

The British Standard used for soil testing for many years was BS 1377:1975. BS 1377:1975 comprised a single document which covered a wide range of tests for classification and geotechnical parameters. However, in certain areas the scope of the old British Standard was limited. For example, when it was written effective stress strength tests were not considered routine in most commercial laboratories and hence the description of such tests were omitted from the standard. BS 1377:1975 has now been completely revised and is superseded by BS 1377:1990. The new British Standard is divided into nine separate parts:

Part 1 General requirements and sample preparation
Part 2 Classification tests
Part 3 Chemical and electro-chemical tests
Part 4 Compaction-related tests
Part 5 Compressibility, permeability and durability tests
Part 6 Consolidation and permeability tests in hydraulic cells and with pore pressure measurement
Part 7 Shear strength tests (total stress)
Part 8 Shear strength tests (effective stress)
Part 9 *In situ* tests.

Soil classification tests

Soil classification, although introducing a further stage of data acquisition into site investigation, has an important role to play in reducing the costs and increasing the cost-effectiveness of laboratory testing. Together with detailed sample description, classification tests allow the soils on a site to be divided into a limited number of arbitrary groups, each of which is estimated to contain materials of similar geotechnical properties. Subsequent more expensive and time-consuming tests carried out to determine geotechnical parameters for design purposes may then be made on limited numbers of samples which are selected to be representative of the soil group in question.

Particle size distribution tests

BS 1377:1990 gives four methods for determining the particle size distribution of soils (part 2, clauses 9.2—9.5). The coarse fraction of the soil (>0.06mm approximately) is tested by passing it through a series of sieves with diminishing apertures. The particle size distribution is obtained from records of the weight of soil particles retained on each sieve and is usually shown as a graph of 'percentage passing by weight' as a function of particle size (Fig. 8.1).

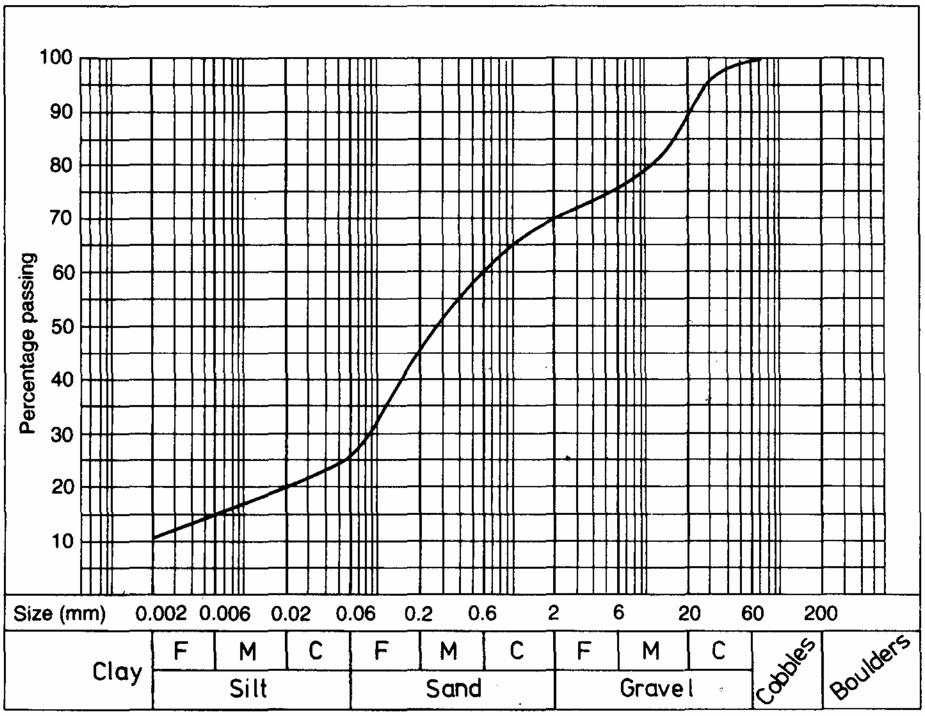


Fig. 8.1 Typical particle size distribution.

Two methods of sieving are defined in BS 1377 (part 2, clauses 9.2, 9.3). Dry sieving is only suitable for sands and gravels which do not contain any clay: the British Standard discourages its use, and since the exact composition of a soil will not be known before testing, it is not often requested. Wet sieving requires a complex procedure to separate the fine clayey particles from the coarse fraction of the soil which is suitable for sieving, as summarized below.

1. Select representative test specimen by quartering and riffling.
2. Oven dry specimen at 105—110°C, and weigh.
3. Place on 20mm sieve.
4. Wirebrush each particle retained on the 20mm sieve to remove fines.
5. Sieve particles coarser than 20 mm. Record weights retained on each sieve.
6. Riffle particles finer than 20mm to reduce specimen mass to 2kg (approx.). Weigh.
7. Spread soil in a tray and cover with water and sodium hexametaphosphate (2 g/l).
8. Stir frequently for 1 h, to break down and separate clay particles.
9. Place soil in small batches on a 2mm sieve resting on a 63 m sieve and wash gently to remove fines.
10. When clean, place the material retained in an oven and dry at 105—110°C.
11. Sieve through standard mesh sizes between 20mm and 6.3 mm using the dry sieving procedure. Note weights retained on each sieve.
12. If more than 150 g passes the 6.3mm mesh, split the sample by riffling to give 100—150g.
13. Sieve through standard mesh sizes between 5mm and 63 tm sieve.

It is important that this procedure is closely adhered to. Inadequate dispersal of the clay particles, poor washing, the overloading of sieves, and insufficient sieving time can all lead to inaccurate results. In particular, extra time and care may be required to ensure full dispersion of clay lumps within the test specimen.

The particle size distribution of the fine soil fraction, between about 0.1 mm and 1 µm may be determined by one of two British Standard sedimentation tests (BS 1377:part 2, clauses 9.4, 9.5). Soil is sedimented through water, and Stokes' law, which relates the terminal velocity of a spherical particle falling through a liquid of known viscosity to its diameter and specific gravity, is used to deduce the particle size distribution.

Sedimentation tests make a number of important assumptions. Since Stokes' law is used, the following assumptions are implied (Allen 1975).

1. The drag force on each particle is due entirely to viscous forces within the fluid. The particles must be spherical, smooth and rigid, and there must be no slippage between them and the fluid.
2. Each particle must move as if it were a single particle in a fluid of infinite extent.
3. The terminal velocity must be reached very shortly after the test starts.
4. The settling velocity must be slow enough so that inertia effects are negligible.
5. The fluid must be homogeneous compared with the size of the particle.

Since Stokes' law applies only in the laminar flow region, for Reynolds numbers of less than 0.2, it cannot be applied to large particles. For quartz spheres (Gs = 2.65) falling in water the critical diameter is 60 µm. Some idea of the minimum particle size that can be measured by sedimentation in water can be obtained by considering the relative displacements per unit time of a small particle due to Brownian motion and gravity settlement. For particles finer than 1 µm Brownian motion exceeds gravitational motion, but in reality since Brownian motion is extremely weak when compared with even the slightest convection current the minimum particle size measurable is about 2tm.

High concentrations of particles in the fluid create a number of problems. Because of the rigidity of the particles, increasing concentrations result in increases in apparent viscosity of the suspension. Additional problems, occur owing to particle—particle interaction: a cluster of particles may have a much greater terminal velocity than the individual particle settling velocities and at volume concentrations as low as 1% the suspension may settle *en masse*, apparently giving a coarser size distribution. High volume concentrations are also associated with the upflow of displaced fluid, causing an over-estimation of the fines content. Vickers (1978) expresses the view that provided that the concentration of soil is maintained at less than 50 g per 1000 ml and the container used for sedimentation is larger than 50mm dia., errors will generally be negligible. Allen (1975) indicates that concentrations should be less, than 1% by vol. (i.e. about 25g per l000ml).

In addition to the assumptions and problems discussed in detail above, the nature of soil particles causes particular inaccuracies in sedimentation testing. First, the methods of preparation (i.e. mechanical agitation) may modify the particle size distribution. Secondly, the density of each soil particle will not equal the average specific gravity of the soil particles times the density of water: clay particles will contain adsorbed or absorbed water giving particle densities which may approach one half of the calculated value. Finally, few soil particles will be spherical. Clay particles will tend to be platy and will not drop vertically, and indeed may not be capable of achieving steady motion.

Two techniques are available for sedimentation. The British Standard (BS 1377:1990) prefers the use of a fixed-depth pipette (BS 3406:part 2) (sometimes referred to in soil testing literature as an Andreasen pipette) to sample the soil — water mix at a depth of 10cm below the fluid surface at regular intervals after the test has been started by evenly distributing the soil in the water. The rate at which the suspension is drawn into the pipette is most important. In analysis, sampling is assumed to take place instantaneously, but the rapid withdrawal of a sample tends to give a finer distribution. BS 3406 recommends a 20s sampling time, while BS 1377 uses a l0s sampling time.

The weight of soil left at that depth after a known time is determined by oven drying the sample, and Stokes' law is then used to deduce the maximum particle size that can be left at that level.

An alternative technique, which requires less sophisticated glassware, uses the hydrometer to determine the density of the soil — water mix at some depth. This method is less accurate in principle than the pipette, because the hydrometer does not measure density at a fixed point below the surface of the fluid, but determines an average value over the depth of its bulb. It is known that the pipette and hydrometer do not yield the same particle size distributions, and it is generally believed that the pipette is more accurate, but the effects of all of the inaccuracy of assumptions discussed above do not appear to have been assessed in absolute terms.

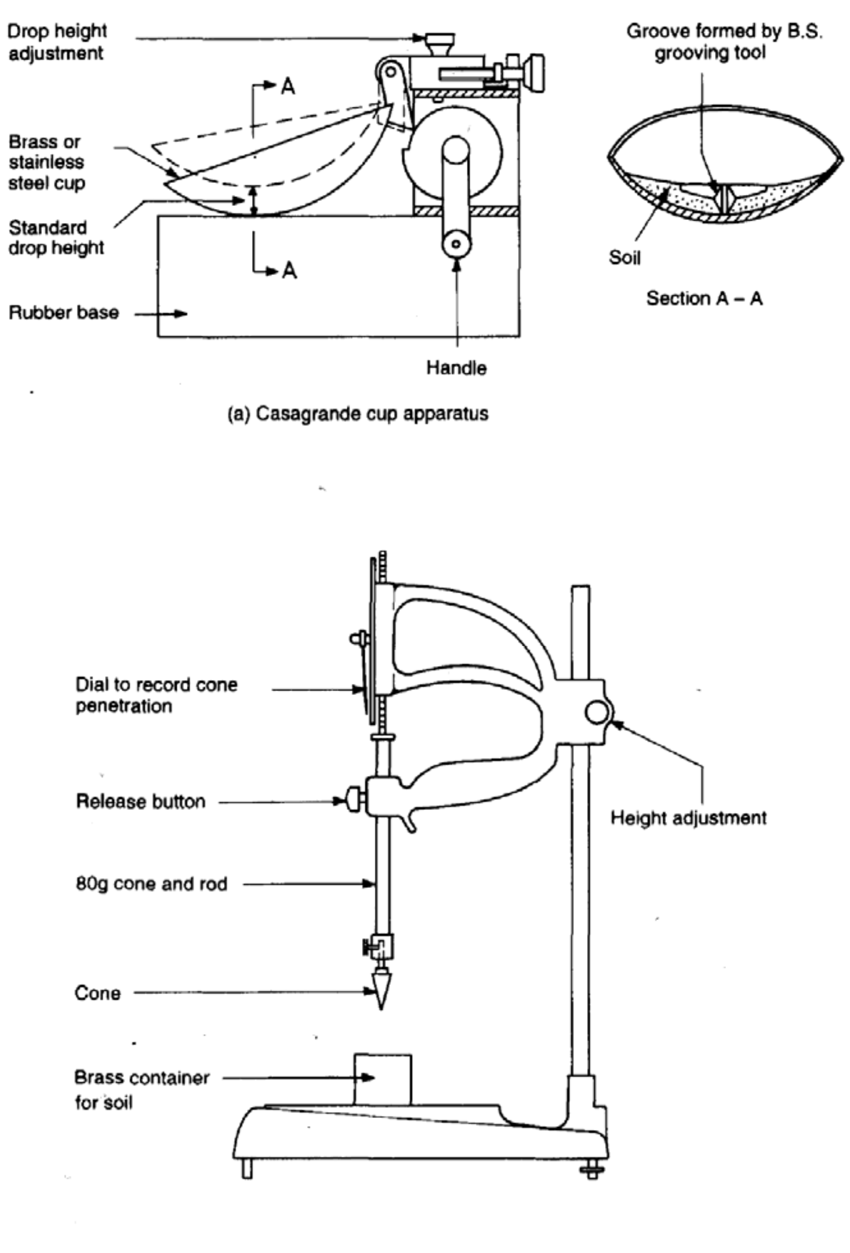
Plasticity tests

The plasticity of soils is determined by using relatively simple remoulded strength tests. The plastic limit is the moisture content of the soil under test when remoulded and rolled between the tips of the fingers and a glass plate such that longitudinal and transverse cracks appear at a rolled diameter of 3 mm. At this point the soil has a stiff consistency.

The liquid limit of a soil can be determined using the cone penetrometer or the Casagrande apparatus (BS 1377:1990:part 2, clauses 4.3, 4.5). One of the major changes introduced by the 1975 British Standard (BS 1377) was that the preferred method of liquid limit testing became the cone penetrometer. This preference is reinforced in the revised 1990 British Standard which refers to the cone penetrometer as the ‘definitive method’. The cone penetrometer is considered a more satisfactory method than the alternative because it is essentially a static test which relies on the shear strength of the soil, whereas the alternative Casagrande cup method introduces dynamic effects. In the penetrometer test, the liquid limit of the soil is the moisture content at which an 80 g, 30^0 cone sinks exactly 20 mm into a cup of remoulded soil in a 5s period. At this moisture content the soil will be very soft. Figure 8.2 shows the cone penetrometer and Casagrande cup. When determining the liquid limit with the Casagrande apparatus, the base of the cup is filled with soil and a groove is then made through the soil to the base of the cup. The apparatus is arranged to allow the metal cup to be raised repeatedly 10mm and dropped freely on to its rubber base at a constant rate of two drops per second. The liquid limit is the moisture content of a soil when 25 blows cause 13mm of closure of the groove at the base of the cup. The liquid limit is generally determined by mixing soils to consistencies just wet and dry of the liquid limit and determining the liquid limit moisture content by interpolation between four points (Fig. 8.3). BS 1377:part 2:1990, clause 4.6 provides factors which allow the liquid limit to be determined from one point (Clayton and Jukes 1978).

The plastic limit test relies heavily on the skill of the operator, and is almost entirely subjective despite attempts by the British Standard to define procedure rigidly. The Casagrande cup method of determining the liquid limit is also rather operator dependent, and in addition suffers from apparatus maintenance problems. These two tests were subject to a comparative testing programme carried out in the UK and reported by Sherwood in 1970. The repeatability of these tests between over 40 laboratories in the UK was tested and gave the results listed in Table 8.2.

The range of results reported for these tests is rather alarming, particularly in view of the fact that it was known by the participating organizations that their results would be compared with those of rival organizations. Sherwood (1970) commented that TRRL attempts to assess the amount of error attributable to defective or worn apparatus in the liquid limit test indicated that the majority of error was due to operator technique. This certainly agrees with our observations which include one of a 15% moisture content error in determining the liquid limit using the Casagrande apparatus as a result of incorrect frequency of drop. When considering the plastic limit test it is surprising that any agreement between laboratories exists. The amount of finger pressure used and the shape of the tips of fingers varies to a great extent and, in addition, operators frequently do not carry out the test using the tips of the fingers (as specified in the British Standard) since these are eminently unsuited to the task.



(a) Casagrande cup apparatus

(b) 80g fall cone apparatus

Fig. 8.2 Casagrande cup and cone penetrometer for liquid limit testing.

Table 8.2 Results of comparative testing programme

	Soil B	Soil G	Soil W
Plastic limit (%)			
Mean	18	25	25
Range	13—24	18—36	20—39
S.D.	2.4	3.2	3.1
Coefficient of variation	13.1	12.8	12.7
Liquid limit (%)			
(Four-point method)			
Mean	34	69	67
Range	29—38	59—84	55—85
S.D.	2.4	5.2	5.3
Coefficient of variation	7.1	7.5	7.9

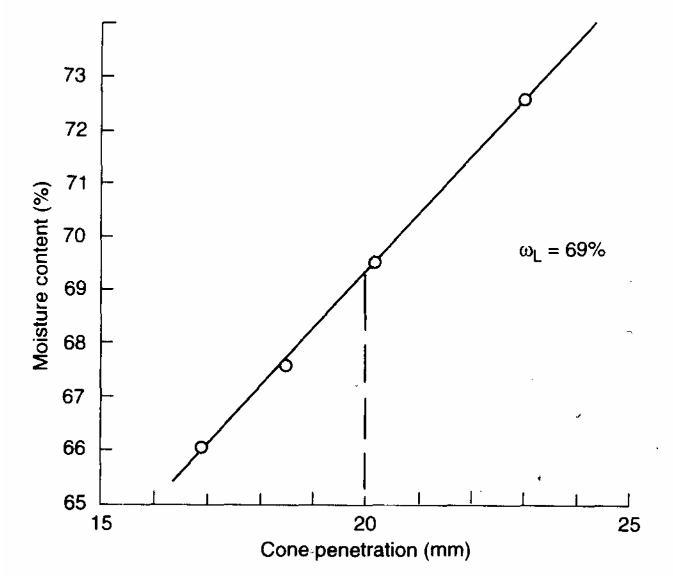


Fig. 8.3 Liquid limit result by four-point cone method.

The definitive method for the determination of liquid limit is the cone penetrometer.

Operator technique can affect this test, particularly since it has been observed that long resting periods, after initially mixing the soil approximately to its liquid limit stage and before carrying out the test, tend to give higher results. (BS 1377:part 2 1990, clause 4.3 Note Three attempts to eliminate this effect by specifying a 24 h rest period between initial mixing of the soil with water, and carrying out the liquid limit test.) The requirement that each part of the test must be repeatable within fixed limits (if observed) however, leads to a much improved result. Tests reported by Sherwood and Ryley (1968), before the introduction of the test as a British Standard, indicated that 'within laboratory' variability is much reduced by the cone penetrometer method. The effects of operator technique between test houses are not known.

Plasticity tests are widely used for classification of soils (Fig. 8.4) into groups on the basis of their position on the Casagrande chart (Casagrande 1948), but in addition they are used to determine the suitability of wet cohesive fill for use in earthworks, and to determine the thickness of sub-base required beneath highway pavements (Road Research Laboratory 1970). The results of wrong decisions in the latter two cases are likely to be much more serious than in the former case; test results from Sherwood (1970) indicate that single plasticity tests, or more than one plasticity test carried out by the same 'biased' operator cannot be used for these purposes.

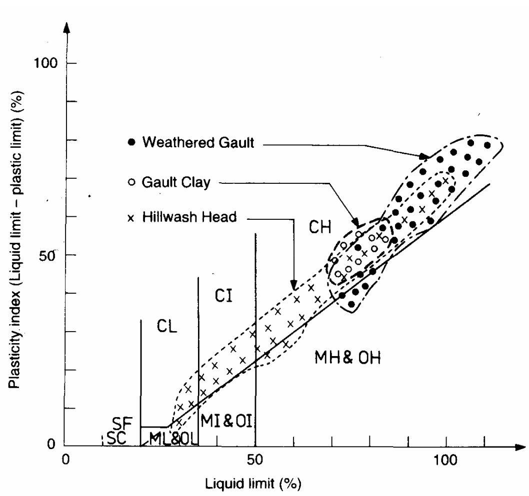


Fig. 8.4 Casagrande plot showing classification of soil into groups.

The extensive use of plasticity testing can be most rewarding, but the low levels of accuracy coupled with high cost tend to discourage use. At the present time liquid and plastic limit tests carried out to the British Standard in the preferred manner will normally take 48—72h to complete, allowing only for resting periods after mixing, and for oven-drying. The result of attempts to improve reproducibility has been a complexity of procedure which has increased expenses as Table 8.3 shows.

Table 8.3 Relative costs of plasticity tests

Plasticity tests	Cost of typical tests, divided by cost of one set of three 38mm dia. specimens tested in undrained triaxial Plasticity tests compression (1995)
Four-point cone penetrometer liquid limit test, plastic limit, plasticity index and natural moisture content BS 1377:part 2, clauses 3, 4.3, 5	0.85
Four-point Casagrande liquid limit test, plastic limit plasticity index and natural moisture content BS 1377:part 2, clauses 3, 4.5, 5	0.79
One-point Casagrande liquid limit test, plastic limit, plasticity index and natural moisture content BS 1377:part 2, clauses 3, 4.6, 5	0.60

The low level of repeatability of the plastic limit test and the high cost and time-consuming nature of the four-point cone penetrometer liquid limit test make these tests unsuitable for construction control or for soil grouping. Clayton and Jukes (1978) have considered the possibility of a one-point cone penetrometer liquid limit, and concluded that such a test could provide a cheap but relatively accurate alternative to the one-point Casagrande method.

Compaction tests

British Standard BS 1377: 1990:part 4 provides three specifications for laboratory compaction:

1. 2.5 kg rammer method;
2. 4.5 kg rammer method; and
3. vibrating hammer method for granular soils.

Compaction has been defined as 'the process whereby soil particles are constrained to pack more closely together through a reduction in the air voids, generally by mechanical means' (Road Research Laboratory 1952). Compaction is therefore a rapid process which does not normally involve a significant change in moisture content.

Laboratory compaction tests are intended to model the field process, and to indicate the most suitable moisture content for compaction (the 'optimum moisture content') at which the maximum dry density will be achieved for a particular soil. The 2.5 kg rammer method is derived from the work of Proctor (1933) which introduced a test intended to be relevant to the compaction techniques in use in earthfill dam construction in the USA in the 1930s. The test subsequently became adopted by the American Association of State Highway Officials (AASHO), and was known as the Proctor or AASHO compaction test. In the original test a mould of capacity 1/30 ft^3 with an internal diameter of 4 in. was filled with soil at a fixed moisture content in three approximately equal layers. Each layer was compacted by 25 blows of a 2 in. dia. 5.5 lb rammer dropping through a height of 12 in. After compaction, the soil was trimmed to the level of the top of the mould, and the wet weight of soil and its moisture content determined. The process was repeated for several increasing moisture contents, and a compaction curve (i.e. dry density as a function of moisture content) was obtained (Fig. 8.5).

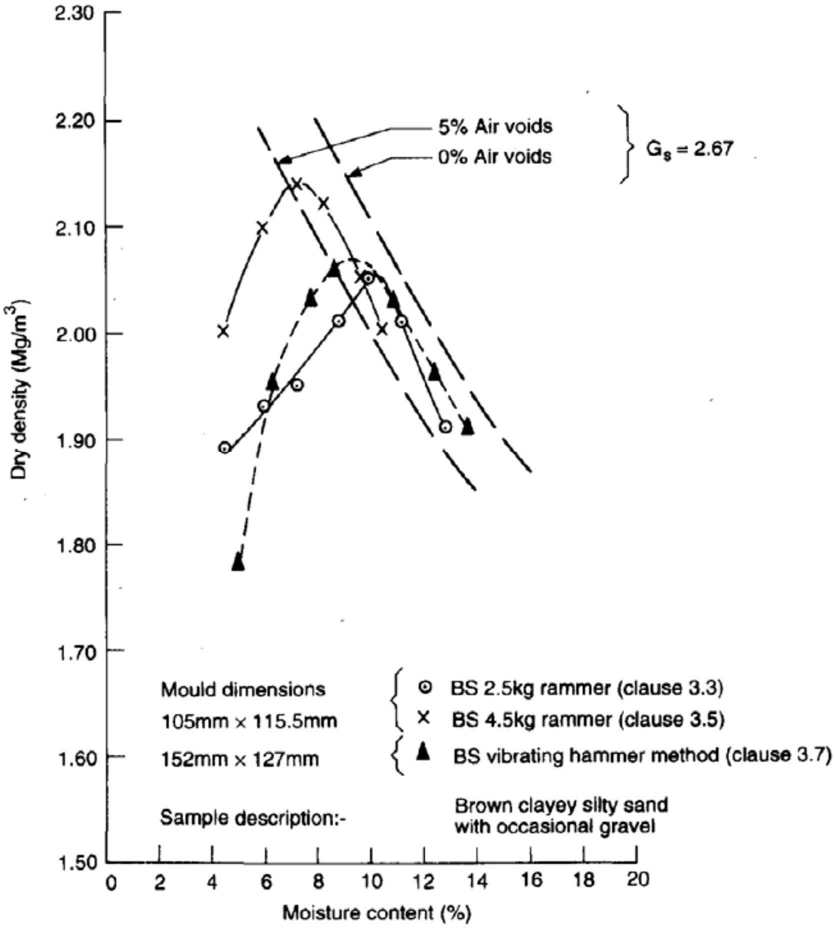


Fig. 8.5 Compaction curves.

Subsequent tests have been developed either to model advances in compaction plant, or as a result of metrication. The Modified AASHO test was developed during World War II to model the heavier standard of field compaction in use during airfield construction. A comparison of these tests is given in Table 8.4.

The vibrating hammer compaction test was introduced in the 1967 revision of BS 1377. This test uses an electrical vibrating hammer with a tamper of approximately the same diameter as the mould (c.f. 145mm and 152mm); the electrical hammer is required to consume between 600 and 750W with an operational frequency between 25 and 45 Hz, and has a dead weight of between 300 and 400 N.

Because of the limited size of the moulds in use, laboratory compaction tests require the exclusion of coarse soil particles. The conventional non-vibratory compaction tests that were covered by the 1975 British Standard made use of 1 litre moulds, necessitating the removal of particles held on the 20mm sieve. In the revised 1990 British Standard the specification for these tests have been extended to include soils with coarse gravel-size particles (BS 1377:part 4:1990, clauses 3.4, 3.6). These compaction tests are suitable for soils containing no more than 30% by mass of material retained on the 20mm sieve, which may include some particles retained on the 37.5 mm sieve. When compared with the conventional tests (clauses 3.3 and 3.5) where coarse gravel-size particles are removed it w1l be seen from the table above that a larger mould (CBR mould) together with a greater number of blows per layer (62) are specified. The vibrating hammer test (BS 1377:part 4:1990, clause 3.7) uses the CBR mould which is suitable for coarse gravel-size particles up to 37.5 mm in diameter.

Table 8.4 Comparison of compaction tests

Test	Soil type	No. of layers	Blows/ layer	Height of drop (in)	Height of drop (mm)	Weight of rammer (lb)	Weight of rammer (kg)	Volume of mould (ft^3)	Volume of mould (cc)
Proctor		3	25	12	305	5.5	2.5	1/30	944
Modified AASHO		5	25	18	457	10	4.55	1/30	944
BS 1377: part 4: 1990									
Clause 3.3	Particles up to medium-gravel size	3	27		300		2.5		1000
Clause 3.4	Soils with some coarse gravel-size particles	3	62		300		2.5		CBR mould *c.* 2300
Clause 3.5	Particles up to medium-gravel size	5	27		450		4.5		1000
Clause 3.6	Soils with some coarse gravel-size particles	5	62		450		4.5		CBR mould *c.* 2300
Clause 3.7		3	Vibratory 25-45 Hz	Vibratory			*c.* 30—40		CBR mould *c.* 2300

The repeatability of the 2.5 kg and 4.5 kg rammer methods of compaction, between laboratories, has been discussed by Sherwood (1970). Typically, the maximum variation of reported optimum moisture content was 4—8%, with reported maximum dry densities varying by up to 0.19 Mg/rn3. Inaccuracies of this magnitude make the tests unsuitable for either design or control, even if it is assumed that their results are relevant to field compaction conditions.

In the UK, compaction tests are used for a variety of purposes. In their simplest application they are used to determine the optimum moisture content and maximum dry density expected of a soil; as a result, soils which have moisture contents widely different from the laboratory optimum may not be used in the construction of a fill and material which is found by *in situ* density measurement (see BS 5930:1981) to have a dry density considerably lower than the laboratory maximum (say less than 95%) may have to be removed from a fill and recompacted. The low level of repeatability and time-consuming nature of compaction tests make them unsuitable for control tests, but in addition there is little evidence to suggest that their results give some optimum condition for the soil.

Different types of soil react in very different ways to each type of roller. It is commonly known that increasing levels of compactive effort tend to produce higher maximum dry density values in conjunction with progressively lower optimum moisture contents, but results from Foster (1962) show that the 'lines of optimums' developed in field compaction trials with different plant are not coincident (Fig. 8.6). The objects of field compaction are to obtain sufficient strength, eliminate collapse, and reduce compressibility of fill to an acceptable level; it is doubtful if these aims can be achieved by the limited use of an empirical test with poor repeatability. This may explain the increasing use of specifications which either define the method of compaction in the field, or limit the air void content of the fill after compaction.

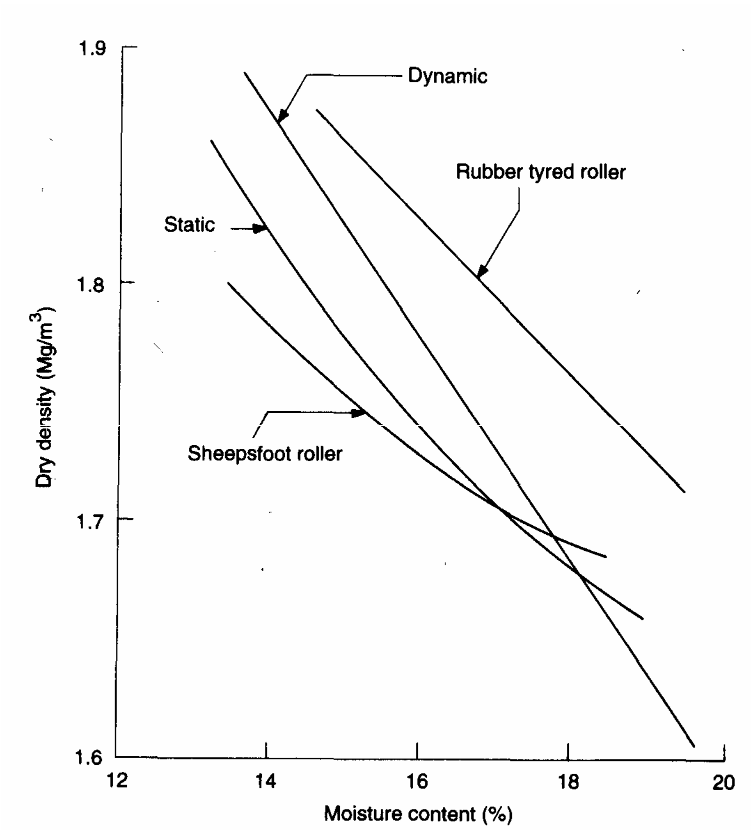


Fig. 8.6 Lines of optimum moisture content/maximum dry density for laboratory compaction methods and two types of field compaction (Foster 1962).

Particle density (specific gravity) determination

Specific gravity values for a soil are not normally used strictly for classification purposes, but are used in the calculation and interpretation of other test results. The specific gravity tests specified in the British Standard (BS 1377:part 2:1990, clause 8) are relatively simple and are based upon determination of the dry weight of a sample of the soil, and the weight of the same sample plus water in a container of known volume. The volume of the container is obtained by weighing the container empty, and full of water. The main problems in conducting the test are of accurate weighing, and complete removal of the air from the soil after the addition of water.

The method still used by most test houses to determine the particle density of fine-grained soil utilizes a 50ml density bottle (BS 1377:part 2:1990 clause 8.3). Unfortunately there is no simple means of knowing when all the air has been removed from the bottle and hence the soil must be de-aired under vacuum. The use of de-aired water will help but it is still necessary to leave the sample in the density bottle under vacuum for several hours. The major difficulty with this test is the provision of a satisfactory vacuum and measuring the length of time required to remove the air completely. These factors can clearly lead to errors in specific gravity determinations. Krawczyk (1969) found that the difficulties of de-airing the soil could be overcome by shaking the sample instead of placing it under vacuum. The advantages of shaking are that the shaking action is easily standardized and the removal of air is more rapid than by the application of a vacuum. Krawczyk proposed that the test should be carried out in a 1 litre gas jar to make the same test suitable for fine-, medium- and coarse-grained soils and the shaking action provided by an end-over-end shaker. This alternative method has been included in the British Standard (BS 1377:part 2:1990, clause 8.2) and should be treated as the preferred method, since in providing a more reliable technique of de-airing the soil it yields more repeatable results.

Results quoted by Sherwood (1970) for three clays tested with especial care to de-air by eight Road Research Laboratory operators are compared in Table 8.5 with the values from some 30 other test houses: they indicate that the specific gravity of British clays may be considerably higher than the 2.65—2.70 values typically expected by experienced engineers.

Table 8.5 Comparison of particle density results

Clay type	RRL results			Other test houses		
	Mean	Mm.	Max.	Mean	Mm.	Max.
Bagshot beds	2.70	2.69	2.72	2.66	2.54	2.81
Gault clay	2.75	2.74	2.77	2.70	2.58	2.84
Weald clay	2.79	2.78	2.81	2.71	2.60	2.85

The results of particle density tests are used in the interpretation of sedimentation test results, to check the results of laboratory compaction tests (BS 1377:1975, clauses 4.1.4, 2.1), and to find the voids ratios of samples during consolidation tests. The test results quoted in Table 8.5 indicate a typical error in particle density determination of about 0.05. Incorrect particle density values affect the position of the voids ratio vs. logarithm of pressure plot for an oedometer consolidation test but they do not affect the values of the coefficients of consolidation (c_v) or compressibility (m_v). A change in particle density leads to a different particle size distribution from the sedimentation test, but the difference is not large and is probably considerably less than the effects of natural soil variability or the assumptions involved in the test.

The major problem arising from an incorrect particle density determination is that of the credibility of compaction tests carried out on the same soil. A low particle density value will push the zero air voids line on a dry density/moisture content plot down and to the left, and may show compaction test results to be apparently impossible (and therefore inaccurate) as they cross over the zero air voids line.

Tests for geotechnical parameters

A wide range of tests has been used to determine the geotechnical parameters required in calculations for example, of bearing capacity, slope stability, earth pressure and settlement, but as testing techniques have changed some tests have been abandoned.

Geotechnical calculations remain almost entirely semi-empirical in nature; it has been said that when calculating the stability of a slope one uses the ‘wrong’ slip circle with the ‘wrong’ shear strength to arrive at a satisfactory answer. For this reason testing requirements differ considerably from region to region. In Scandinavia the *in situ* vane is widely used to determine the undrained shear strength of clays while in Britain this parameter is normally determined using the unconsolidated undrained triaxial compression test. Bearing in mind the Norwegian Geotechnical Institute’s experience in applying Scandinavian techniques to the design of embankments in Asia, some caution should be exercised in introducing familiar techniques to unfamiliar ground conditions.

Clearly each region develops its own testing techniques and comes to appreciate the necessary ‘factor of safety’ applicable to each type of calculation and each method of obtaining parameters. This section relates to laboratory practice in the UK at the time of writing.

Strength tests

The principal tools available for strength determination in a good UK geotechnical testing laboratory are the California Bearing Ratio (CBR) apparatus, the Franklin Point Load Test apparatus (Franklin *et* *al.* 1971; Broch and Franklin 1972), the laboratory vane apparatus and various forms of direct shear and triaxial apparatus.

California bearing ratio (CBR) test

The CBR and Franklin point load tests are empirical in nature. The CBR test is primarily used to assess the strength of materials used in or beneath flexible highway or airfield pavements. The test may be carried out *in situ*, or in the laboratory:

BS 1377:part 4:1990, clause 7 gives a detailed description of the British Standard test method while the Road Research Laboratory publication (1952) describes the development of the test and some previous applications of its results.

The CBR test was specifically developed by the California State Highway Department for the evaluation of sub-grade strengths in the investigation of existing pavements of known performance in use (Porter 1938, 1942). This led to an empirical method of pavement design.

The test is carried out by forcing a standard plunger (approximately 50mm dia.) into the soil at a more or less constant rate of 1.25 mm/mm. Measurements of applied load and plunger penetration are made at regular intervals, and a curve is plotted for penetrations of up to 12.5 mm. Figure 8.7 shows the laboratory apparatus, and a typical result. The California Bearing Ratio is obtained by dividing the plunger loads at penetrations (after bedding correction) of 2.5 and 5.0 mm by the loads given at the same penetrations on a standard crushed stone. The loads given by the soil under test are expressed as percentages of the standard load, and the highest value is taken as the CBR value for design.

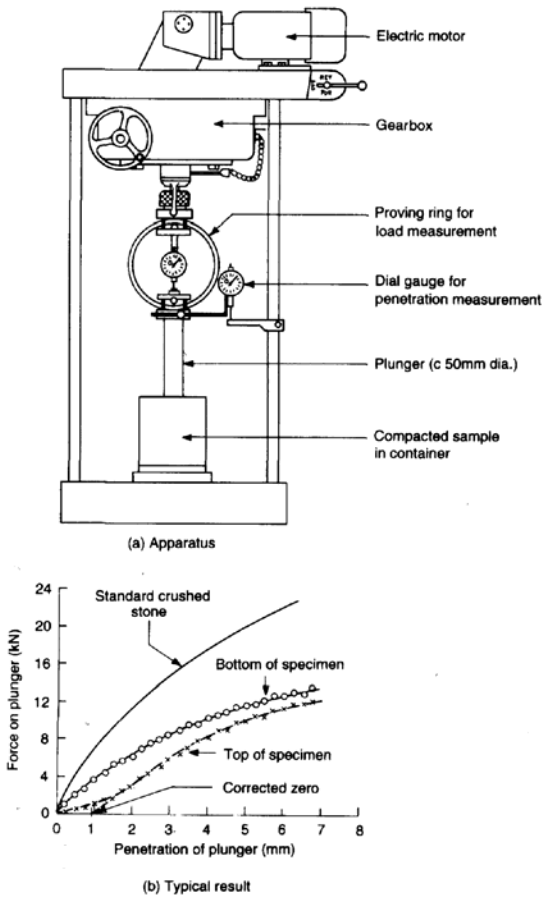


Fig. 8.7 CBR apparatus and typical result.

The CBR test primarily involves shear deformation of the soil beneath the plunger, but its results cannot be accurately related to any of the fundamental shear strength parameters. Its use is therefore restricted to the design of road and airfield pavements. Because of the empirical nature of such designs it is of the utmost importance that the test is carried out precisely in the manner used to develop the particular design method in use. In the UK the CBR test is no longer widely used, because pavement design carried out based on the observed performance of pavements in the UK (Road Research Laboratory 1970) allows the CBR value to be obtained from particle size or plasticity index. Apart from a limited amount of testing to check the quality of sub- grade during construction, the only other use is to determine the strength of granular sub-base (Department of Transport 1976).

Franklin point load test

The Franklin point load test (Broch and Franklin 1972; ISRM 1985) was developed at Imperial College, London to provide a quick and reliable measurement of the strength of unprepared rock core samples, both in the field and the laboratory. The apparatus consists of a small loading frame which is activated by an hydraulic hand pump and ram (Fig. 8.8a). Rock core is placed between pointed platens of standard dimensions and loaded until failure occurs. The point load strength index:

$$I_s = \frac{P}{D^2} \tag{8.1}$$

where P = force required to break the specimen, and D = distance between the platen contact points. The results give a measure of the tensile strength of the rock. Tests may most reliably be carried out across the core diameter, but results can also be obtained when discs of core are loaded axially. Under this latter condition, corrections to the point load strength index will be required which will depend on the aspect ratio of the specimen (Broch and Franklin 1972). Under extreme circumstances, when only irregular lumps of rock are available, the test can be carried out along the shortest axis of the lump, but results will be less reliable. The value of diametral point load strength has been shown by Broch and Franklin to be dependent on the core size, with larger diameter cores giving smaller values of point load index. It has therefore been proposed that a standard classification be adopted by correcting all values to a reference diameter of 50mm. A correction chart for this purpose is given in Fig. 8.8b, based on the results of tests on five rock types at diameters between 10 and 80 mm.

Laboratory vane test

The principles involved in the vane test are discussed in Chapter 8, under '*In situ* testing'. Whilst the field vane typically uses a blade with a height of about 150 mm, the laboratory vane is a small-scale device with a blade height of about 12.7mm and a width of about 12.7 mm. The small size of the laboratory vane makes the device unsuitable for testing samples with fissuring or fabric, and therefore it is not very frequently used. The laboratory vane test is described in BS 1377 :part 7:1990, clause 3.

Direct shear test

The vane apparatus induces shear along a more or less predetermined shear surface. In this respect the direct shear test carried out in the shear box apparatus (Skempton and Bishop 1950) is similar. Figure 8.9 shows the basic components of the direct shear apparatus; soil is cut to fit tightly into a box which may be rectangular or circular in plan (Akroyd 1964; Vickers 1978; ASTM Part 19; Head 1982; BS 1377:1990), and is normally rectangular in elevation. The box is constructed to allow displacement along its horizontal mid-plane, and the upper surface of the soil is confined by a loading platen through which normal stress may be applied. Shear load is applied to the lower half of the box, the upper half being restrained by a proving ring or load cell which is used to record the shear load. The sample is not sealed in the shear box; it is free to drain from its top and bottom surfaces at all times.

The cross-sectional area over which the specimen is sheared is assumed to remain constant during the test.

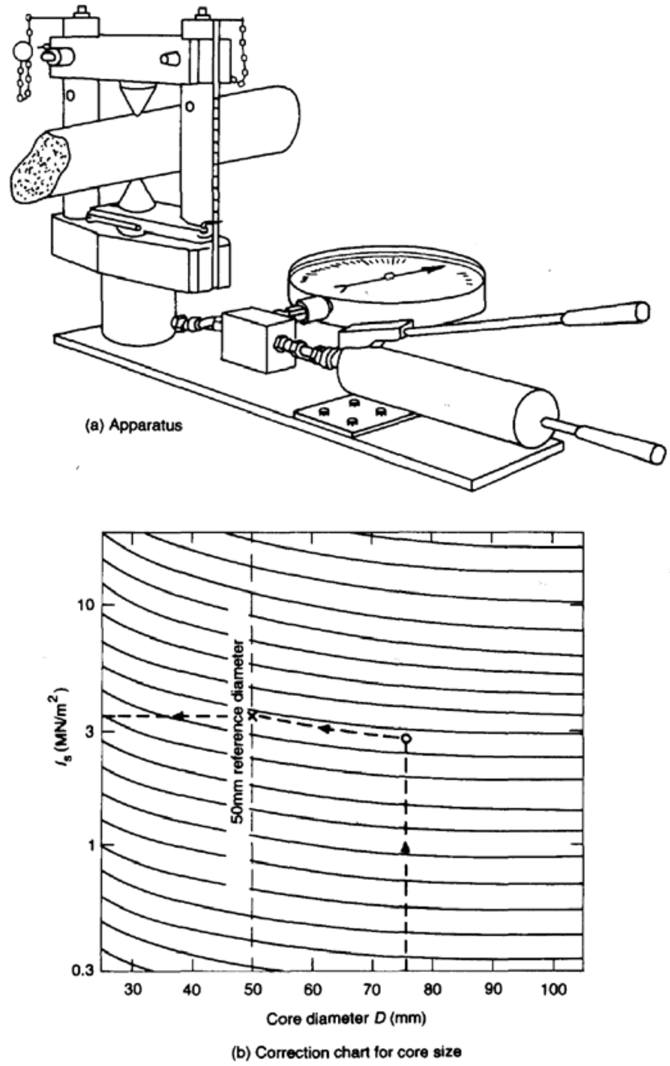


Fig. 8.8 Point load apparatus (Broch and Franklin 1972; Brown and Phillips 1977).

The direct shear test has been used to carry out undrained and drained shear tests, and to determine residual strength parameters. Morgenstern and Tchalenko (1967) reported the results of optical measurements on clays at various stages during the direct shear test, and it is clear that at peak shear stress and beyond, failure structures (Reidels and thrust structures) are not coincident with the supposed imposed horizontal plane of failure. In addition, the restraints of the ends of the box create an even more markedly non-uniform shear surface. Since the direction of the failure planes, the magnitude and directions of principal stresses and the pore pressure are not determinable in a normal shear box experiment, its results are open to various interpretations (Hill 1950), and this test is now rarely used to determine undrained or peak effective strength parameters. Triaxial tests may be performed more conveniently and with better control.

In the UK, shear box tests are now used mainly to determine residual shear strength parameters for the analysis of pre-existing slope instability (Skempton 1964; Skempton and Petley 1967). In this application the technique of cut-plane shear box testing described by Petley (1966) gives results which have been found to be satisfactory, based on back-analysis (for example see Foster (1980)). A specimen of clay is placed in the shear box and allowed to swell for 24h under the weight of the load hanger. Following this, the specimen is consolidated under the required normal pressure and measurements of vertical compression are made. The two halves of the box are then separated sufficiently to allow a cheese wire to cut smoothly through the specimen. The two halves of the specimen are then separated, and the soil surfaces smoothed by rubbing a glass plate lubricated by distilled water over the surfaces. When smooth, the lower half of the soil specimen is raised by packing it with a few layers of filter paper: the box is reassembled, and after applying a normal stress, the specimen is subjected to large displacements on the preformed shear surface by repeatedly reversing the travel of the box. The maximum shear stress obtained for each stage of shearing should be plotted against the logarithm of cumulative displacement, and shearing should continue until this curve levels out. The lowest maximum shear stress values (in the final shear stage of each test) are plotted against their imposed normal stresses to obtain the residual effective strength parameters (c'_r (normally zero) and φ'_r) for a soil. Typical values are given in Table 8.6.

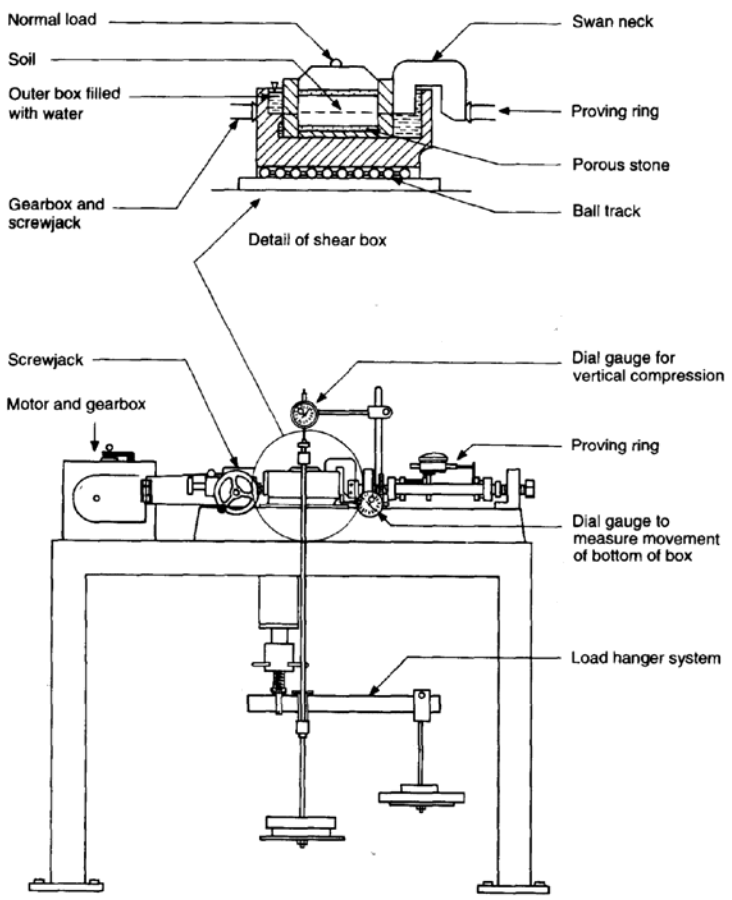


Fig. 8.9 Bishop direct shear box.

A better form of test to find residual parameters is carried out on an annular specimen in the ring shear apparatus, described by Bishop *et al.* (1971). Because of its cost and complexity this apparatus has failed to find a place in site investigation testing laboratories, but a simpler form of ring shear test described by Bromhead (1979) has been adopted by BS 1377:part 7:1990, clause 6.

The simplified ring shear test is carried out on an annular specimen of remoulded clay 5mm thick, with internal and external diameters of 70mm and 100mm respectively. The specimen is confined radially between concentric rings and the vertical normal stress is applied via two porous bronze loading platens (Fig. 8.10). Relative rotary motion takes place between the confining rings (which are fixed to the lower loading platen) and the upper platen. This causes the sample to shear,, the shear surfaces forming close to the upper platen. The loading platens are roughened in order to prevent slip at the platen—soil interface. The upper platen reacts against two matched proving rings (or load cells) which provide a measurement of the torque transmitted through the soil specimen.

Table 8.6 Effective strength parameters for some UK soils

Soil type	w_l (%)	w_p (%)	Peak shear strength c'_r (kN/m2)	Peak shear strength φ'_r (deg)	Residual shear strength c'_r (kN/m2)	Residual shear strength φ'_r (deg)
Sand and gravel						
Loose			0	36—42		
Medium dense			0	40—48		
Sand						
Loose			0	30—34		
Dense			0	37—43		
Silt						
Loose			0	28—32		
Dense			0	30—34		
Chalk						
Senonian, remoulded	28	22	0	30—34		
Cenomanian, intact	42	17	600—1000	32—37		
Granular glacial till	15—24	12—14	0—40	35—42		
Oxford clay	57	27	172	28	4	13
Weald clay	60—65	25—32	8	22	0	9—15
Gault clay	55	23	53	22	0	18
Unweathered London clay	71	29	125	26	0	10
Weathered London clay	70—90	25—30	16—20	19—21	0	9—14

The main advantage of the Bromhead ring shear apparatus is that it is relatively simple but still allows an infinite relative displacement without the necessity of reversing the direction of relative motion along the shear plane developed in the soil sample. Preparation of a sample for the test involves kneading remoulded soil into the annular cavity formed by the confining rings and the lower platen. Surplus material is struck off such that the top surface of the soil is level with the top of the confining rings. With the upper platen placed on top of the soil, and the surrounding water bath filled to prevent evaporation during the test, the specimen is consolidated under a normal stress by weights applied to the load hanger.

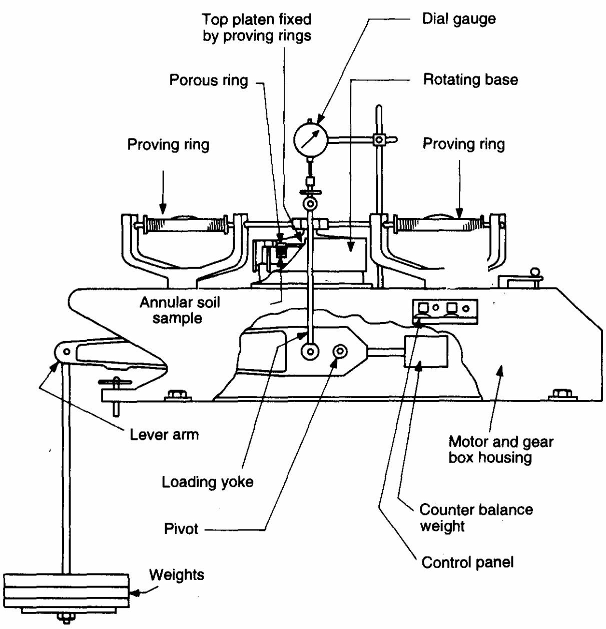


Fig. 8.10 Bromhead ring shear apparatus (Bromhead 1979).

The rate of shearing during *drained direct shear testing* must be slow enough to ensure that no excess pore water pressure exists on the failure plane by the time shear strength measurements are to be made. In practice it is normal to shear the specimen slowly enough so that excess pore pressures are insignificant when the peak shear strength is developed. The time to failure can be determined from the time/consolidation data obtained before the start of the shear stage, from which the coefficient of consolidation of the soil can be obtained, because:

$$c_v = \frac{T_v d^2}{t} \tag{8.2}$$

where T_v = time factor at some specified percentage of consolidation, d = average drainage path distance for the specimen, and t = time taken for the specimen to reach the specified percentage of consolidation (see, for example, Terzaghi (1923, 1943)).

Taylor and Merchant's method (Taylor and Merchant 1940;BS 1377:part 5:1990) can be used to determine the point on a plot of compression as a function of square root of time at which 90% consolidation has occurred. At 90% consolidation of time factor, T_v = 0.848. Gibson and Henkel (1954) have expressed the minimum time to failure in the shear box as:

$$t_f = \frac{h^2}{2c_v(1-\overline{U}_c)} \tag{8.3}$$

where h = half the specimen height, c_v = coefficient of consolidation, determined above, and $\overline{U}_c$ = mid-plane pore-pressure dissipation ratio. A minimum dissipation ratio of 0.95 (i.e. 95% dissipation of excess pore pressures) is normally used, and the rate of deformation in the experiment is then calculated using an estimated value of deformation at failure, based on experience. If, subsequently, failure occurs before the required minimum time to failure then the results of the test may be invalid.

BS 1377:1990 recommends that the time to failure should be based on the elapsed time for 100% consolidation (t_{100}) using the relationship:

$$t_f = 12.7t_{100} \tag{8.4}$$

A similar approach based on t1 is also recommended by the 1990 British Standard for determining the rate of displacement for ring shear tests. However, most *ring shear tests* are carried out at a speed of 0.048% mm when using the Bromhead apparatus. The British Standard suggests in a note in part 7, section 6.4.5.1 that this speed has been found satisfactory for a large range of soils. Faster rates are likely to disrupt the sliding surface and result in erroneous values of φ_r.

The residual angle of shearing resistance φ_r varies with the effective normal stress (σ'_n) acting on the sliding surface (Bishop *et al.* 1971; Bromhead 1979; Lupini *et al.* 1981). For London clay, Bishop *et al.* found that φ_r was 14° at low effective normal stress ($\sigma'_n <$ 30kPa) and reduces to less than 9° at effective normal stresses greater than 100 kPa. Such variations in residual strength make it necessary to measure φ_r at a range of effective normal stresses. The procedure adopted for use with the Bromhead ring shear apparatus by most commercial laboratories is as follows.

1. Place the initial (lowest) load on the hanger and, with the gear disengaged, create a shear surface in the upper part of the specimen by rotating the handwheel through one or two revolutions. After rotating the handwheel ensure that any load on the proving rings or load cells is taken off by reversing the direction of handwheel rotation.

2. Take a set of initial readings, engage the gear, start the motor and shear for several hours until a constant torque measurement is achieved. Generally, shearing overnight has been found satisfactory.
3. Place the next load on the hanger and continue to shear whilst taking readings at regular intervals until the torque remains constant for 20 mm. The whole stage should take about 1 h.
4. Repeat stage (3) at least three times with increasing normal loads.
5. When the maximum load is reached, stop the motor and manually reverse the rotation until a zero torque reading is achieved. If the torque is not taken off, the release of proving ring energy into the specimen causes disruption of the shear surface.
6. Reduce the load on the hanger back to the initial (lowest) load and commence shearing again until a constant torque reading is achieved.
7. Compare the torque reading from stage (6) with that at the end of stage (2). If the readings are the same, the test is complete; if they differ significantly then the test must be repeated.

This procedure, although in common usage, is not given in the 1990 British Standard. A typical ring shear test performed in the above manner should take about 24 h to complete. Typical values of normal stress used in such tests include 25, 50, 75 and 100 kPa.

Triaxial test

The triaxial apparatus has been described in great detail by Bishop and Henkel (1962). The test specimen is normally a cylinder with an aspect ratio of two, which is sealed on its sides by a rubber membrane attached by rubber ‘O’ rings to a base pedestal and top cap (Fig. 8.11). Water pressure inside the cell provides the horizontal principal total stresses, while the vertical pressure at the top cap is produced by the cell fluid pressure and the ram force. The use of an aspect ratio of two ensures that the effects of the radial shear stresses between soil, and top cap and base-pedestal are insignificant at the centre of the specimen.

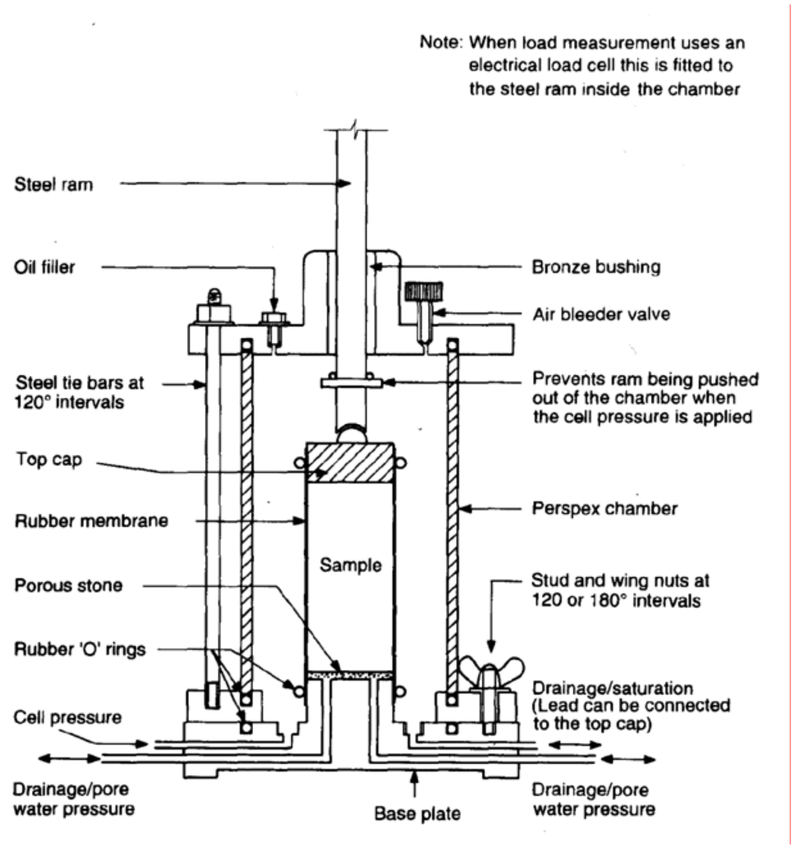


Fig. 8.11 Triaxial cell.

The triaxial apparatus requires one or two self-compensating constant pressure systems, a volume change measuring device and several water pressure sensing devices. The ram force may be measured outside the cell using a proving ring, but most modern systems now use an internal electrical load cell mounted on the bottom of the ram. The ram is driven into the triaxial cell by an electrical loading frame which will typically have a capacity of 5000 or 10000 kgf and is capable of running at a wide range of constant speeds; triaxial tests are normally carried out at a controlled rate of strain increase.

When this apparatus is used to measure strength the specimen is normally failed in triaxial compression, that is with the intermediate principal stress held constant and equal to the minor principal stress and with the major principal stress increased to bring about failure. Under these conditions the height of the specimen decreases during shearing. The three most common forms of test are:

1. the unconsoldiated undrained triaxial compression test, without pore water pressure measurement (BS 1377:part 7:1990. clause 8);
2. the consolidated undrained triaxial compression test, with pore water pressure measurement (BS 1377:part 8:1990, clause 7); and
3. the consolidated drained triaxial compression test, with volume change measurement (BS 1377:part 8:1990, clause 8).

The unconsolidated undrained triaxial compression test is carried out on 'undisturbed' samples of clay in order to determine the undrained shear strength of the deposit *in situ*. Pore pressures are not measured during this test and therefore the results can only be interpreted in terms of total stress. Three test specimens, which may be either 38mm or 102mm dia. and will normally have an aspect ratio of 2, are extruded from a core and sealed using a rubber membrane, 'O' rings and top and bottom caps. Once a specimen is inside the triaxial cell, the cell pressure is increased to a predetermined value and the specimen is brought to failure by increasing the vertical stress; during this period regular readings of the ram load and specimen height decrease are made. The cell pressures used will normally increase by a factor of two between each of the three specimens, with the middle pressure approximately corresponding to the vertical total stress at the level of sampling in the ground. Thus for a sample taken from 5 m depth cell pressures of 50, 100 and 200 kN/m^2 would be used.

The rate of strain used during the test will normally be 2%/mm. This rate is based on the specifications given in the 1975 British Standard for the maximum strain (20%) and the maximum test duration (10mm.). However, BS 1377:1990 (part 7, clause 8) recommends that the rate of axial deformation should produce failure within a period of 5—15 mm. The recommendation concerning the maximum axial strain remains unchanged. If the same criteria for selecting a rate of strain are adopted using these recommendation the rate of strain should be 1.5%/mm. It should be pointed out that the undrained strength is not a fundamental property of the soil and the measured strength is sensitive to the rate at which the soil is sheared. It is therefore advisable to adopt the same rate for all tests of this type.

Since the major total principal stress (acting in a vertical direction) is composed of two components, i.e.

$$\sigma_1 = \sigma_3 + \frac{P}{A} \tag{8.5}$$

where σ_3 = horizontal total stress (the cell pressure), P= ram force, and A = specimen cross-sectional area. The principal stress difference (or deviator stress), $\sigma_1 - \sigma_3$, is simply equal to the ram force divided by the cross-sectional area. Because the test is carried out undrained, with no volume change allowed, the specimen diameter increases during the test. In order to calculate the cross-sectional area at any time during testing it is assumed that the specimen deforms as a right cylinder and so:

$$V = A_0H_0 = AH = AH_0\,(1 - \varepsilon_a) \tag{8.6}$$

where V= specimen volume (constant), A_0 and H_0 are the original specimen area and height, A and H are the specimen area and height at some time during the test, and ε_a is axial strain at some time during the test. Thus $A=A_0/(1-\varepsilon_a)$.

The results of the test are plotted as curves of principal stress difference against strain. For conditions of maximum principal stress difference (taken as failure) Mohr circles are plotted in terms of total stress. The average undrained shear strength should be quoted, and the failure envelope drawn tangential to the Mohr circles in order to find the undrained 'cohesion intercept' and undrained 'angle of shearing resistance'.

A correction should be applied to the measured maximum deviator stress to allow for the restraining effect of the membrane (BS 1377:1990). For a barrelling type of failure which occurs in a plastic soil the correction (σ_{mb}) is given by:

$$\sigma_{mb} = \frac{4M\varepsilon_a(1-\varepsilon_a)}{D}\,(kN/m^2) \tag{8.7}$$

where M = compression modulus of the membrane material per unit width, ε_a = axial strain at failure, and D = initial diameter of specimen.

The compression modulus of the membrane material, M, is assumed to be equal to its extension 'modulus. The method by which the extension modulus is measured is described by Bishop and Henkel (1962) and Head (1982).

In soils which exhibit brittle failure a different membrane correction may be necessary, although not mentioned in the British Standard. This correction is described by Head (1982).

For soils of high strength, such as stiff clays, the effect of the membrane restraint is small and is often neglected. For soft and very soft clays the membrane effect can be significant and omission of the correction could lead to errors on the unsafe side.

The membrane correction described above is deducted from the maximum measured deviator stress.

The size of specimen tested in the undrained triaxial test can have a significant effect on the resulting shear strength (Bishop *et al.* 1965; Agarwal 1968; Marsland and Randolph 1977). While larger specimens may give parameters which are more relevant, for example, to slope stability calculation (for example, Skempton and La Rochelle (1965)) because of their inclusion of fissures or fabric, it is important to recognize that some empirical or semi-empirical design methods were specifically designed on the basis of undrained shear strengths measured on small diameter specimens. Strutted excavations (Peck 1969) and the adhesion on bored piles (Skempton 1959) are examples of this type of problem.

The decision to test large diameter specimens can cause particular problems when, as is often the case, deposits are more variable in the vertical than the horizontal direction. Three 204mm high specimens cannot be taken from a standard 450mm long open-drive tube sample. To overcome the problems of shortage of material the aspect ratio of the specimens may be reduced to one by using lubricated end platens (Rowe and Barden 1964) or each specimen may be sheared at three cell pressure levels (Taylor 1950; Parry 1963; Anderson 1974). This latter technique is known as 'multi-staging', and has been found to be particularly useful in boulder clay materials where stone content makes the preparation of undisturbed specimens difficult, and test results from individual specimens typically give a large strength variation. Multistage tests are described in Head (1982) and BS 1377:part 7:1990, clause 9.

Peak effective strength parameters (c' and φ') may be determined either from the results of consolidated undrained triaxial compression tests with pore pressure measurement or from consolidated drained triaxial compression tests. The former test is normally preferred because it can be performed more quickly and therefore more economically.

The consolidated undrained triaxial compression test is normally performed in several stages, involving the successive saturation, consolidation and shearing of each of three specimens. Saturation is carried out in order to ensure that the pore fluid in the specimen does not contain free air. If this occurs, the pore air pressure and pore water pressure will differ owing to surface tension effects: the average pore pressure cannot be found as it will not be known whether the measured pore pressure is due to the pore air or pore water, and at what level between the two the average pressure lies. Perhaps more importantly, the presence of air in the pore pressure measuring system can lead to time lags, which for relatively incompressible over-consolidated clay soils can be very significant. Bishop and Henkel (1962) quote theoretical times for 98% equalization of pore pressure for undisturbed 38 mm London clay specimens which vary from about 1 mm to 6h, depending on the compressibility of the pore pressure measuring system.

Saturation is normally carried out by leaving the specimens to swell against an elevated back pressure. The use of a back pressure on dense specimens which are expected to dilate has the additional advantages of extending the range of applied stress for which pore pressure measurements can be made and, in drained tests, of preventing the formation of air locks in the triaxial pedestal and pipework leading to the specimen. Back pressure (which is simply an imposed pore pressure) is applied through a volume change gauge to the top of the specimen, while a cell pressure of slightly higher value is also applied. Both cell pressure and back pressure are normally increased in increments of about 50 kN/m2, allowing time for equalization at each stage.

The degree of saturation can be expressed in terms of Skempton's pore pressure parameter (Skempton 1954):

$$B = \frac{\Delta u}{\Delta \sigma_3} \tag{8.8}$$

where Δu = change in pore pressure for an applied cell pressure change of $\Delta\sigma_3$.

For a saturated soil, B equals unity. In practice it has been found that B approximates to unity (say $B \geq 0.98$) when a back pressure of 200—300 kN/m2 has been used on natural clays, but compacted samples may require back pressures of 400—800 kN/m2. Once a reasonable back pressure has been achieved the B value can be checked by measuring the response of the pore pressure to an applied cell pressure change. BS 1377:part 8:1990, clause 5 recommends that a value of B greater than or equal to 0.95 must be achieved before the specimen may be considered as fully saturated and the consolidation stage started.

The consolidation stage of an effective stress triaxial test is carried out for two reasons. First, three specimens are tested and consolidated at three different effective pressures, in order to give specimens of different strengths which will produce widely spaced effective stress Mohr circles. Secondly, the results of consolidation are used to determine the minimum time to failure in the shear stage. The effective consolidation pressures (i.e. cell pressure minus back pressure) will normally be increased by a factor of two between each specimen, with the middle pressure approximating to the vertical effective stress in the ground.

When the consolidation cell pressure and back pressure are applied to the specimen, readings of volume change are made using a volume change device in the back pressure line. The speed at which volume change takes place depends on the effective pressure increment, the coefficient of consolidation of the soil and the drainage conditions at the specimen boundaries. Normally, pore pressure will be measured at the specimen base, with drainage to the back pressure line taking place through a porous stone covering the top of the specimen. The speed at which heavy clays consolidate and may be sheared can be significantly increased by the use of filter paper drains on the radial boundary of the specimen (Bishop and Henkel 1962). The coefficient of consolidation of the clay can be determined by plotting volume change as a function of the square root of time. Theoretical considerations indicate that the first 50% of volume loss during consolidation should show as a straight line on this plot. This straight line is extended down to cut the horizontal line representing 100% consolidation, and the time intercept at this point (termed 't_{100}' by Bishop and Henkel) can be used to obtain the coefficient of consolidation as shown below (in fact, t1 is equal to $4 \times t_{50}$, and cannot equal the infinite time theoretically required for complete consolidation, see Fig. 8.12).

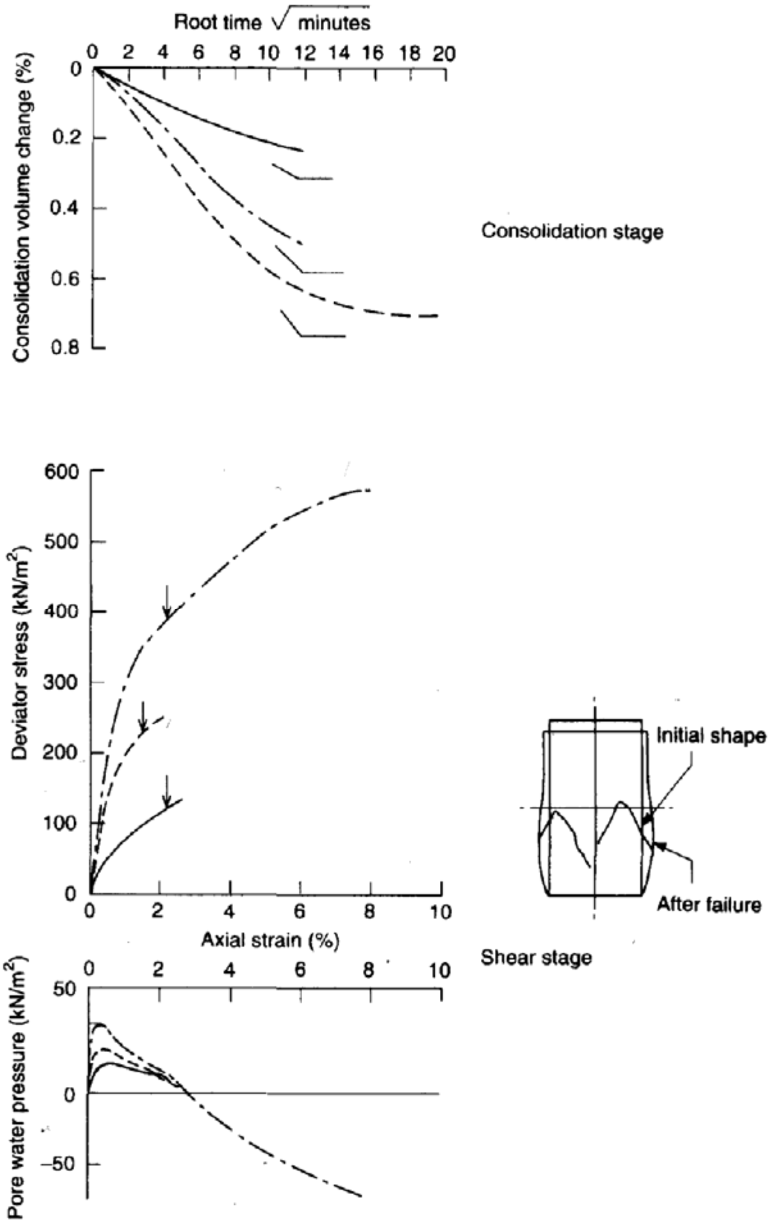


Fig. 8.12 Consolidation and shear stage results for a consolidated undrained triaxial compression test with pore pressure measurement. (Arrows denote principal stress ratio failure, $(\sigma_1'/\sigma_3')_{max}$.)

For drainage from one end only:

$$c_v = \frac{\pi h^2}{t_{100}} \tag{8.9}$$

For drainage from both ends and radial:

$$c_v = \frac{\pi h^2}{100 t_{100}} \tag{8.10}$$

where c_v is the coefficient of consolidation of the clay and h is one-half of the specimen height.

The filter drains used on the radial boundary of triaxial specimens do not cover their entire periphery, and are not infinitely permeable. Work by Bishop and Gibson (1964) indicates that the equations above may significantly under-estimate the coefficient of consolidation of more permeable clays, or silts, tested at high effective pressures, and it will therefore be unwise to use these results indiscriminately for other types of engineering calculation.

A minimum time to failure during the shear stage is necessary, not only to allow for the time lag in the pore pressure measuring system, but also to allow equalization of pore pressures within the specimen. Friction between the specimen and the porous stones creates non-uniformity of stress and strain conditions between the centre and ends of the specimen, and as a result the pore pressures set up during an undrained test are non-uniform. Testing must be carried out slowly enough so that almost complete equalization of pore pressures at the centre and ends of the specimen takes place.

For drainage from one end only:

$$t_f = \frac{1.67 h^2}{c_v} \tag{8.11}$$

For drainage from both ends and radial boundary:

$$t_f = \frac{0.071 h^2}{c_v} \tag{8.12}$$

for 95% equalization of pore pressure in specimens of diameter, *h*, and height, *2h*.

The minimum time to failure (t_f), or to the first valid effective stress readings if a stress path is required, can be obtained by combining the equations above to give:

$t_f = 0.53\ t_{100}$	for drainage from one end	(8.13)
$t_f = 2.26\ t_{100}$	for drainage from the entire boundary	(8.14)

The equations are strictly only valid if the major assumptions in their derivation are correct. It is assumed, *inter alia*, that the pore pressure differences are parabolic over the specimen height and are proportional to the applied load. When brittle failure is expected to take place over a narrow failure zone the rate of testing should be of the order of 10 times slower (La Rochelle 1960).

Once consolidation is complete, the specimen may be isolated from the back pressure and the rate of vertical movement of the compression machine platen set. For this, the minimum time to failure is divided into the estimated axial sample deformation at failure (or at the time of the first valid readings). Soil will normally fail at axial strains of between 2 and 20%, and the actual figure used is largely based on experience of testing similar soil types. During the shear stage the vertical stress is increased by the loading ram, and measurements are made at regular intervals of deformation, ram load and pore pressure. These are converted to graphs of principal stress difference (σ_1- σ_3) and pore pressure as a function of strain (Fig. 8.12), and failure is normally taken as the point of maximum principal stress difference. The effective stress Mohr circles are plotted for the failure conditions of the three specimens, and the gradient and intercept of a straight line drawn tangential to these circles defines the effective strength parameters c' and φ' (Fig. 8.13).

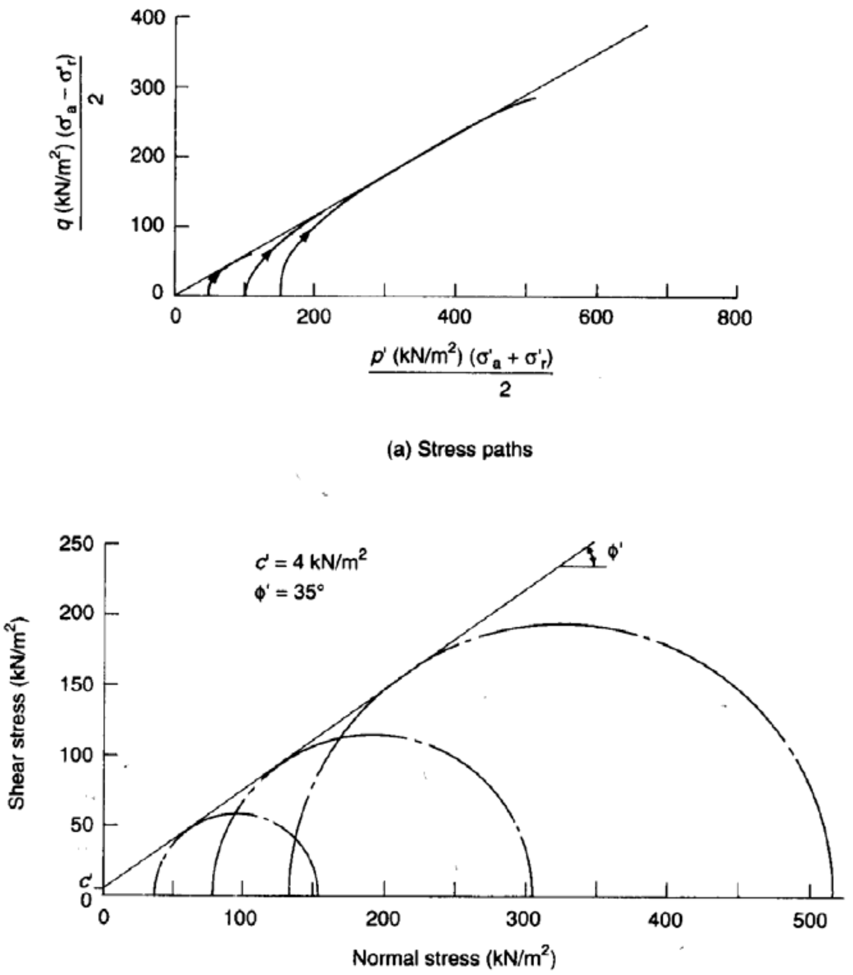


Fig. 8.13 Stress paths and Mohr circles at failure for a consolidated undrained triaxial compression test with pore pressure measurement.

Effective stress triaxial tests are far less affected by sample size effects than undrained triaxial tests, but the problems of sampling in stoney soils still make multistage testing an attractive proposition under certain circumstances. The effectiveness of this technique in consolidated undrained triaxial testing has been reported by Kenney and Watson (1961), Parry (1968) and Parry and Nadarajah (1973).

The consolidated drained triaxial compression test, with volume change measurement during shear is carried out in a similar sequence to the consolidated undrained test, but during shear the back pressure remains connected to the specimen which is loaded sufficiently slowly to avoid the development of excess pore pressures. The coefficient of consolidation of the soil is derived in the manner described above from the volume change measurements made during the consolidation stage. Gibson and Henkel (1954) found that the average degree of consolidation at failure is related to the time from the start of the test by the equation:

$$t_f = \frac{h^2}{\eta c_v (1 - \overline{U}_f)} \tag{8.15}$$

where 2h = specimen height, c_v = coefficient of consolidation, and $\overline{U}_f$ = average degree of consolidation at time t.

For a specimen with an aspect ratio of 2, η equals 0.75 for drainage from one end only and 40.4 for drainage from both ends and the radial boundary. Thus, to achieve 95% consolidation:

$t_f = 8.48\ t_{100}$ for drainage from one end only (8.16)

$t_f = 15.78\ t_{100}$ for drainage from both ends and the radial boundary (8.17)

Thus the shear stage of a drained triaxial test can be expected to take between 7 and 15 times longer than that of an undrained test with pore pressure measurement. 100mm dia. specimens of clay may require to be sheared for as much as one month. Once shearing is complete, the results are presented as graphs of principal stress difference and volume change as a function of strain, and the failure Mohr circles are plotted to give the drained failure envelope defined by the parameters c_d' and φ_d'.

The effective strength parameters defined by drained triaxial testing should not be expected to be precisely the same as those for an undrained test, since volume changes occurring at failure involve work being done by or against the cell pressure (Skempton and Bishop 1954). In practice the resulting angles of friction for cohesive soils are normally within 1—2°, and the cohesion intercepts are within 5 kN/m^2. The results of tests on sands can vary very greatly (for example, Skinner 1969).

Stiffness tests

From the 1950s through to the early 1980s there has been a preoccupation in commercial soil testing with the measurement of strength with less emphasis being paid to the measurement of detailed stress—strain properties such as stiffness. This is reflected in both the 1975 and the 1990 editions of BS 1377, both of which fail to consider the measurement of stiffness.

During the last decade, two important parallel developments have taken place which have resulted in the measurement of stiffness being considered more important than that of strength in geotechnical design, particularly for sensitive structures. These developments are:

1. methods of measuring strain locally on laboratory test specimens have shown that the stress—strain behaviour of many soils and weak rocks is significantly non-linear with very high stiffness at the small strains operational around most engineering structures (Jardine *et al.* 1984); and
2. certain features of field measurements of ground deformation around full-scale structures, which could not be modelled using linear elastic theory, are resolved if non-linear formulations are used incorporating very high initial stiffness (Simpson *et al.* 1979).

These developments have resulted in the application of finite element models in geotechnical design becoming commonplace, with stiffness parameters derived both from special laboratory stiffness tests and from field geophysics.

In most soils any discontinuities such as fissures will generally have a stiffness that is similar to that of the intact soil such that the intact soil stiffness may be used to predict with reasonable accuracy ground deformations and stress distributions. This means that laboratory triaxial tests on good quality ‘undisturbed’ specimens may yield adequate stiffness parameters for design purposes. However, conventional measurements of axial deformation of triaxial specimens, made outside the triaxial cell, introduce significant errors in the computation of strains.

The conventional method of measuring axial deformation is shown schematically in Fig. 8.14a. The errors in the computation of strain that arise from this method of measurement result from the fact that apparatus is compliant; the load cell, porous stones, lubricated end platens (when used) and filter papers will all compress under increasing axial load (Baldi *et al.* 1988). Further errors are associated with bedding caused by lack of fit or surface irregularities at the interfaces between the specimen and loading surfaces (Daramola 1978; Burland and Symes 1982). Although the errors due to apparatus compliance can be evaluated with reasonable certainty by careful calibration, the bedding error can be very difficult to assess since its magnitude depends on the way in which the ends of the specimen are prepared. Thus the only way to obtain accurate determinations of axial strain is to carry out the measurement remotely from the ends of the specimen, and preferably on its middle third (Fig. 8.14b). This type of measurement is referred to as 'local strain' measurement. A comparison of local and axial strain measurements made on the same test specimen are shown in Fig. 8.15. It will be seen that the errors are greatest during the early stages of the test.

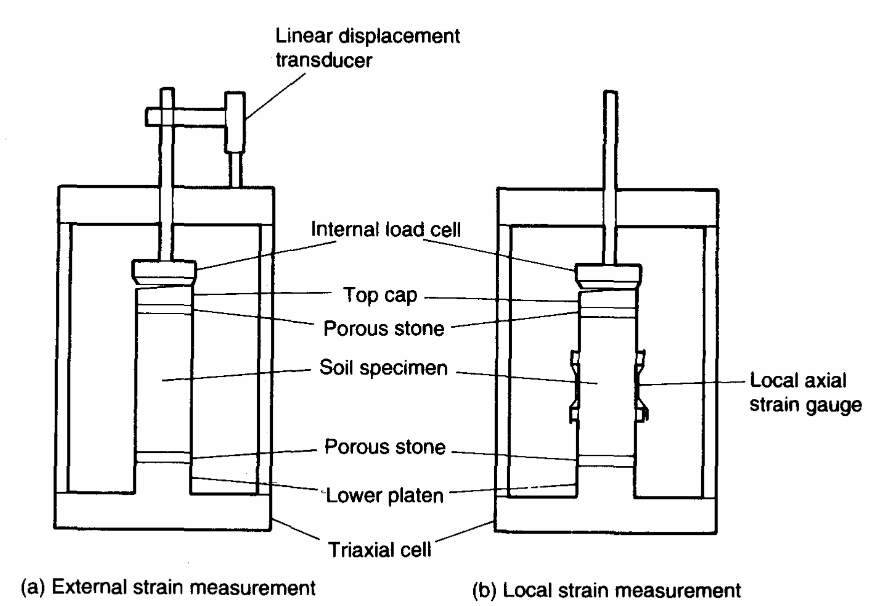


Fig. 8.14 Schematic diagram illustrating external and local strain measurement in the triaxial apparatus.

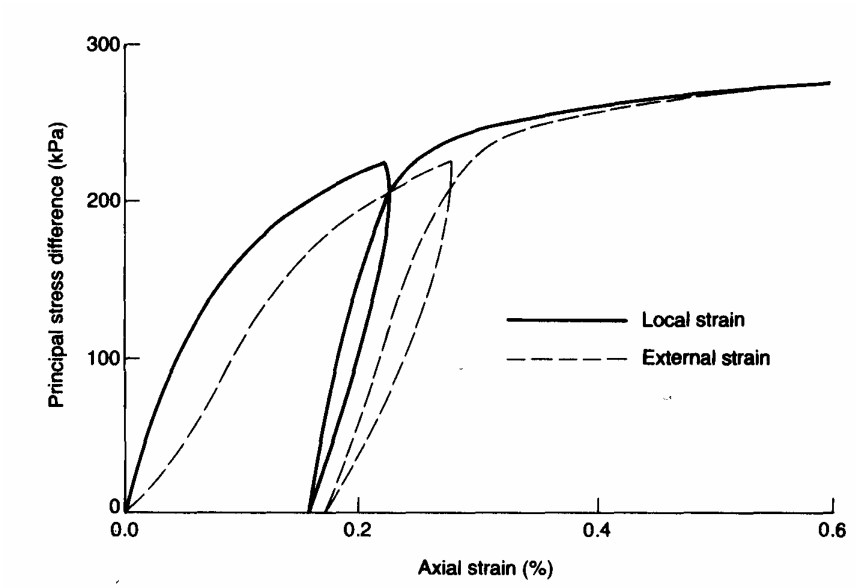


Fig. 8.15 Comparison of local and external strains (Clayton and Khatrush 1986).

During the 1980s the accurate measurement of soil stiffness at small strains (<0.1%) was one of the most challenging topics in most geotechnical research laboratories. The instrumentation for these local strain measurements includes:

1. miniature displacement transducers;
2. proximity transducers;
3. electrolevel gauges (Burland and Symes 1982; Jardine *et al.* 1984);
4. Hall effect semiconductor (Clayton and Khatrush 1986; Clayton *et al.* 1989); and
5. strain gauged metal strips (LDT).

Of these the electrolevel gauges and the Hall effect semiconductors (Fig. 8.16) are in use in commercial laboratories in the UK and local strain gauges (axial and radial) based on the Hall effect semiconductor are available commercially.

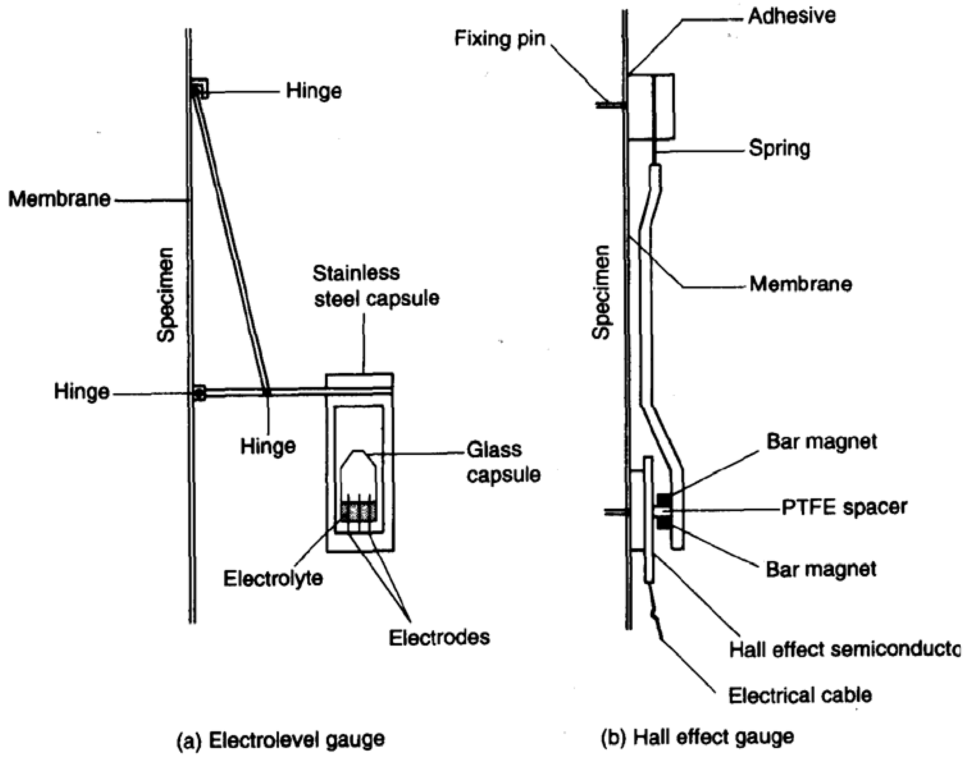


Fig. 8.16 Principle of operation and detail of electrolevel and Hall effect local strain gauges (Jardine *et al.* 1984, and Clayton and Khatrush, 1986).

In soil mechanics it has become traditional to emphasize the non-linear stress— strain behaviour by plotting secant modulus (E_{sec} or G_{sec}) against log axial strain. The relationship between secant modulus and log strain for most soils is shown in Fig. 8.17. It will be seen from Fig. 8.17 that at very small strains (i.e. <0.001%) the stiffness is constant indicating a linear stress—strain relationship. It is thought that the soil behaves elastically at these strains. Between 0.001% strain and 0.1% strain the stiffness of the soil may drop by an order of magnitude. The ground strains that have been measured around structures are generally between 0.2% and 0.5%. Thus, the portion of the curve that is of greatest interest is where the stiffness is most sensitive to strain level.

In soil mechanics practice, for materials considered to be relatively unaffected by cementing it is common to normalize E or G with respect to the mean effective stress (p'_0) immediately before shearing, but for materials which are considered to be cemented to a significant degree it is thought better to normalize with respect to the undrained strength (c_u). Examples of such normalized curves are shown in Fig. 8.18.

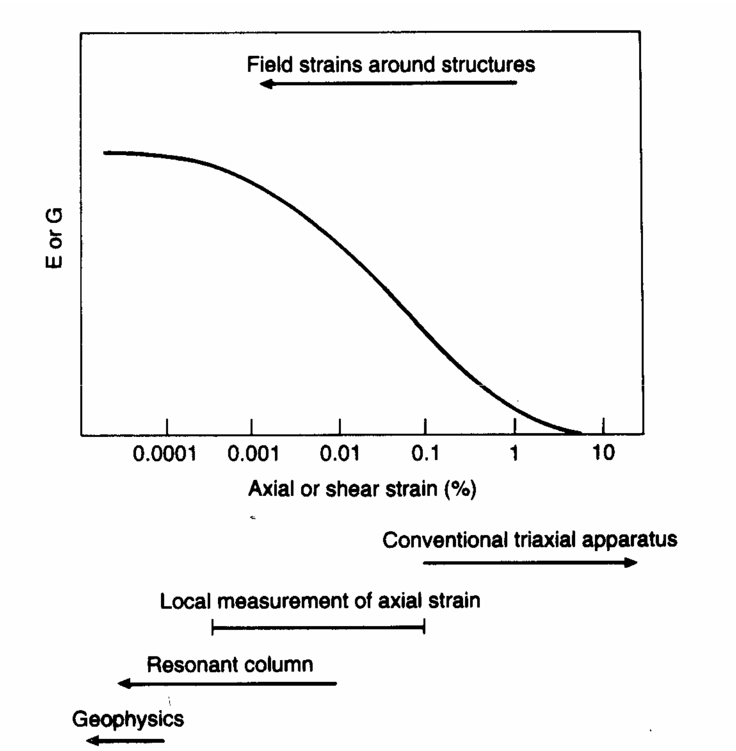


Fig. 8.17 Typical relationship between stiffness and strain for soils.

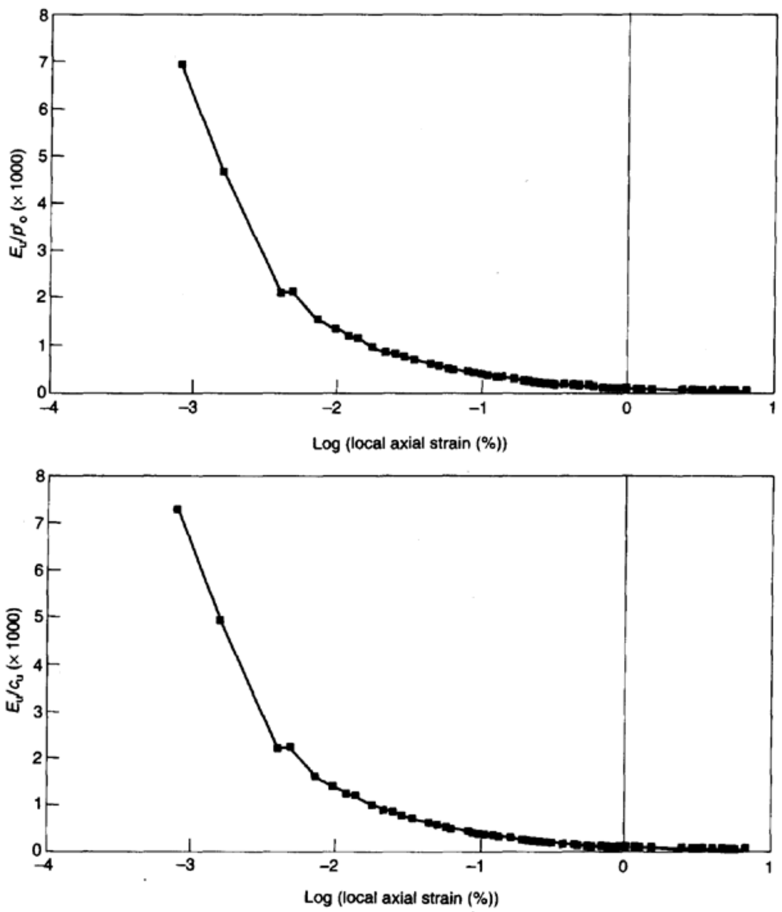


Fig. 8.18 Normalized stiffness—log strain curves.

Recent research has shown that stiffness measured in the triaxial apparatus is affected by the following factors (in descending order of importance):

1. cementing (bonding or structuring);
2. effective stress (in less cemented materials, such as most lightly and moderately overconsolidated clays);
3. sample disturbance (see Chapter 6) which results both in changes in effective stress and in destructuring;
4. history (i.e. overconsolidation);
5. stress path and stress-path rotation
   The stress path followed will have a significant influence on the measured stiffness. In particular, changes in direction of the stress path such as a loading path followed by an unloading path will result in an increase in stiffness (Simpson 1992);
6. Ageing (i.e. creep and rest period)
   Rest period refers to the period during which the soil remains at a constant stress between the end of the most recent stress path and the start of the current path. The duration of the rest period can have a significant effect on the measured stiffness.

It is beyond the scope of this book to provide a detailed discussion on the other factors which affect the measurement of stiffness. A comprehensive discussion of these factors may be found in Atkinson and Sallfors (1991).

The above factors which affect the measurement of stiffness indicate the importance of providing adequate similitude between the test and the field prototype. In many cases this will involve conducting a stress path test (for example Lamb (1967), Head (1986)). These tests are not standardized, but are specified by engineers on the basis of experience and the needs of their own project. It is common practice for local strain measurement in such tests to be combined with mid-plane pore pressure measurement (Hight 1982), in order to provide more reliable pore pressures.

CONSOLIDATION TESTS

Consolidation tests are frequently required either to assess the amount of volume change to be expected of a soil under load, for example beneath a foundation, or to allow prediction of the time that consolidation will take. The effect of predictions based on consolidation test results can be very serious, for example leading to the use of piling beneath structures, and the use of sand drains or stage construction for embankments. It is therefore important to appreciate the limitations of the commonly available test techniques.

Three pieces of apparatus are in common use for consolidation testing in the UK. These are:

1. the oedometer (Terzaghi 1923; Casagrande 1936);
2. the triaxial apparatus (Bishop and Henkel 1962); and
3. the hydraulic consolidation cell (Rowe and Barden 1966).

Casagrande oedometer test

The Casagrande oedometer test is most widely used. The apparatus (Fig. 8.19) consists of a cell which can be placed in a loading frame and loaded vertically. In the cell the soil sample is laterally restrained by a steel ring, which incorporates a cutting shoe used during specimen preparation. The top and bottom of the specimen are placed in contact with porous discs, so that drainage of the specimen takes place in the vertical direction when vertical stress is applied; consolidation is then one-dimensional.

The most common specimen size is 76mm dia. x 19mm high, since this allows the highly disturbed edges of a 102 mm dia. sample to be pared off during specimen preparation. Where the specimen preparation process may be prevented by the presence of stones, the specimen diameter must be equal to that of the sampler.

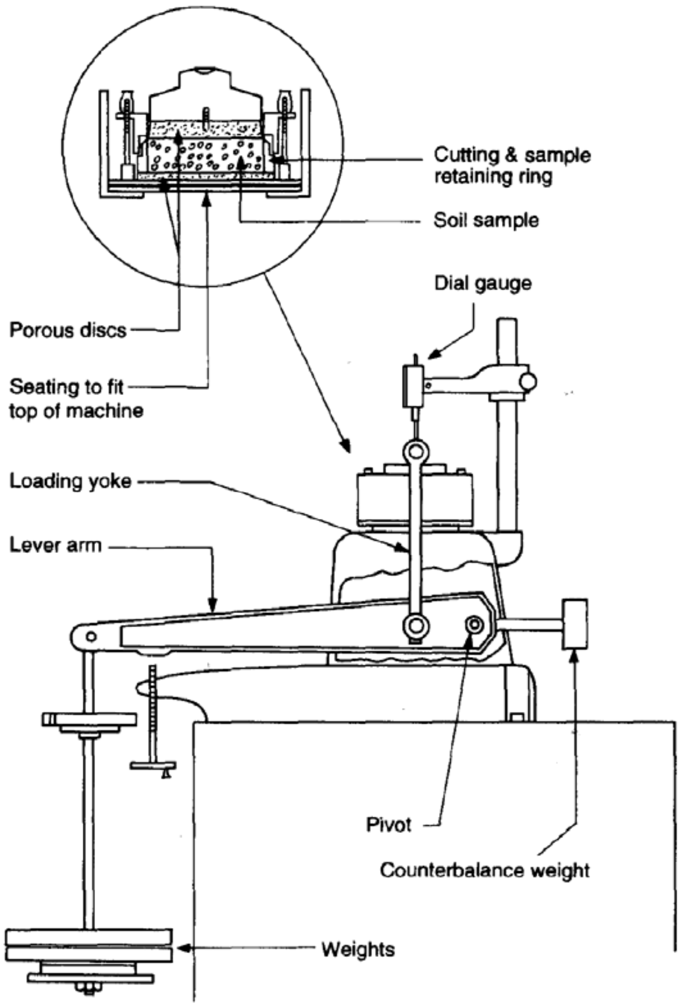


Fig. 8.19 Casagrande oedometer apparatus.

BS 1377:part 5:1990, clause 3 gives a standard procedure for the test. In this procedure the specimen is subjected to a series of pre-selected vertical stresses (e.g. 6, 12, 25, 50, 100, 200, 400, 800, 1600, 3200 kN/m^2) each of which is held constant while dial gauge measurements of vertical deformation of the top of the specimen are made, and until movements cease (normally 24 h). Dial gauge readings are taken at standard intervals of time after the start of the test (i.e. 0, 15 and 30s, 1, 2, 4, 8, 15, 30 and 60mm, 2, 4, 8 and 24h). At the same time that the first load is applied, the oedometer cell is flooded with water, and if the specimen swells the load is immediately increased through the standard increments until swelling ceases.

Swelling pressures in stiff plastic overconsolidated clays are of considerable importance to the foundations of lightly loaded structures, and the technique suggested by BS 1377:part 5:1990, clause 4 allows an assessment of them to be made. The procedure involves balancing the swelling pressure once the water is added by keeping the dial gauge reading stationary by the careful application of weights to the hanger.

The results of each loading stage of an oedometer test are normally plotted as the dial gauge readings either as a function of square root of elapsed time, or as a function of the logarithm of elapsed time.

The coefficient of consolidation (c_v) used in calculations of settlement can be obtained from these curves, using Taylor and Merchant's method, or Casagrande's method respectively.

The c_v values obtained from the results of tests on the relatively small oedometer soil specimen will normally be very significant under-estimates. Rowe (1968b, 1972) gives ratios between the coefficient of consolidation when determined from *in situ* tests and from the oedometer which vary between a factor of 3 and 10^3, with good correspondence only for clays with absolutely no fabric. Since these types of material are rare, it will be wise to check oedometer c_v values using some more reliable method. *In situ* permeability test results (Chapter 9) can be combined with oedometer coefficients of compressibility (m_v) values or a larger laboratory test may be used.

The results of all the oedometer load stages are normally combined in one graph of void ratio as a function of the logarithm of effective pressure (Fig. 8.20), constructed on the basis of the calculated void ratios at the end of each of the load stages. These results are also used to calculate the coefficient of compressibility ($m_v = \Delta e/(1 + e_o).\ (1/\Delta p)$, where Δe is the void ratio change for a pressure change Δp which is used to predict the magnitude of settlement. This is carried out for each load stage, and for a 100 kN/m2 load increment above the *in situ* vertical effective stress level at the sample depth. Coefficient of compressibility results are seriously affected by sample disturbance in soft or sensitive clays, and by sample size effects in hard clays and soft rocks.

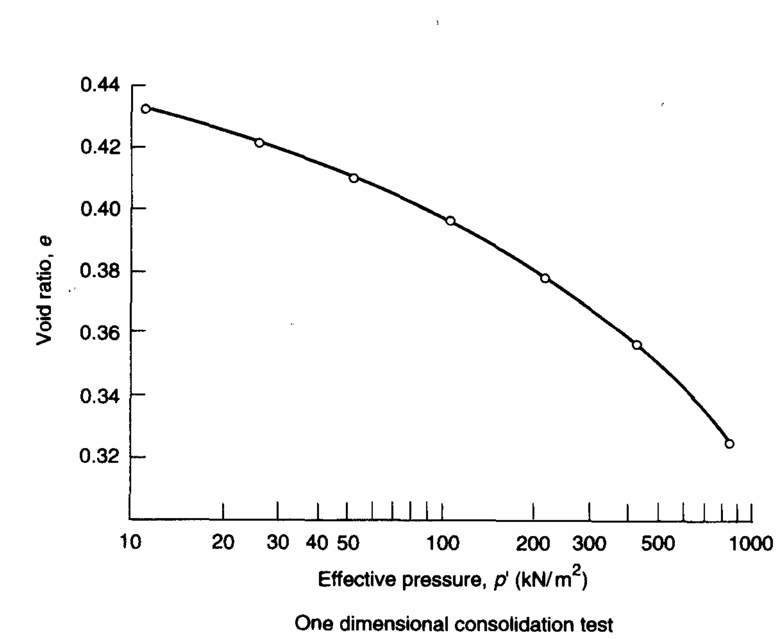


Fig. 8.20 Oedometer test result.

In soft clays, the effects of sample disturbance are to reduce the compressibility values measured in the oedometer, and to modify the voids ratio/log (pressure) curve so that the preconsolidation pressure in lightly overconsolidated clays is obscured (Schmertmann 1955). Schmertmann (1953) provides an empirical method for recovering the field' compressibility curve from the laboratory data, but it is better to obtain high quality large diameter piston samples. Work by Berre *et al.* (1969) and Bjerrum (1973) has amply demonstrated that poor sampling and storage techniques can modify the behaviour as a result of redistribution of moisture content between the periphery and centre of a sample, as a result of the imposition of small shear strains on sensitive soil structures, and because of chemical changes during storage. It does not seem possible that these effects can be reversed by the application of corrections'.

In hard clays and soft rocks the mass compressibility is much affected by the compressibility of the joints and bedding planes which traverse it. Compressibility measurements made on specimens which are unrepresentative because they do not contain these discontinuities will tend to under-estimate the settlements of structures to be placed on them, whilst in contrast the compressibility of relatively unweathered or fractured materials may be over-estimated because of bedding effects at the end caps, or as a result of disturbance during specimen preparation (Hobbs 1975).

In stiff overconsolidated clays it has been observed that for foundations of limited width compared with the depth of compressible soil the straightforward application of oedometer coefficient of compressibility to the expression:

$$\rho = \int_0^z m_v \Delta\sigma_z dz$$

where ρ = surface settlement, m_v = coefficient of compressibility in a soil layer of thickness dz, and $\Delta\sigma_z$ = stress increase due to the foundation at that level, yields overestimates of the consolidation settlement. Skempton and Bjerrurn (1957) proposed a model whereby the soil deforms in two stages. In the first, immediately after load application, a change of soil shape occurs without change of volume. As a result of undrained loading and shear stress application a pore pressure is set up which may be significantly less than the applied vertical stress, for overconsolidated soils. It is the change of effective stress due to the dissipation of this pore pressure which leads to long-term consolidation settlements, which will normally occur after the end of construction.

Simons and Som (1969) and Simons (1971) noted that the vertical strain of London clay is greatly influenced by the relative magnitudes of vertical and horizontal stress, and their increments, during consolidation. Since the effective stress path followed by soil in the oedometer test differs significantly from that taken by soil beneath a foundation in the field, oedometer tests cannot be directly applied to making accurate settlement predictions.

In very soft or sensitive clays the accurate assessment of pre-consolidation pressure is important if settlements are to be reasonably predicted, because of the significant increase in compressibility at higher stress levels. Crawford (1964) notes that the rate of compressive strain in the laboratory may be as much as several million times greater than that in the field, and that test procedure has a very large effect on the estimated preconsolidation pressure. He suggests that pressure—compressibility characteristics should be investigated using soil strain rates more compatible with those observed in the field. Constant rate of loading or constant rate of strain tests have been widely reported (Smith and Wahls 1969; Aboshi *et al.* 1970; Wissa *et al.* 1971), but rarely used in the UK where very soft sensitive clays are not common.

Triaxial dissipation test

The measurement of consolidation characteristics can be carried out in the triaxial dissipation test. The most common size of specimen is 102mm high x 102mm dia., and the test is carried out in a triaxial chamber such as might be used for a consolidated undrained triaxial compression test with pore pressure measurement. The specimen is compressed under the isotropic effective stress produced by the difference between the cell pressure and the back pressure, and volume change is recorded as a function of time, as in the consolidation stage of an effective strength triaxial compression test, but in addition pore pressure is measured at the base of the specimen. Drainage occurs upwards in the vertical direction but soil compression is three-dimensional, and for this reason the results of this test are not strictly comparable with those of an oedometer test. The compressibility determined from volume changes during the triaxial dissipation is greater than that measured under conditions of zero lateral strain, and the difference is most pronounced for overconsolidated clays and compacted soils.

When testing compacted soils the initial stages of the test involve undrained application of cell pressure, to allow an assessment of the pore pressure parameter B. Most natural soils will be saturated *in situ*, and these are normally subjected to resaturation by the application of back pressure. Consolidation is started by either increasing the cell pressure or decreasing the back pressure to give the required effective stress, and then applying the back pressure to the top of the specimen. Volume changes and pore pressure measurements are plotted as a function of the logarithm of elapsed time, as shown in Fig. 8.21. Owing to the expansibility of leads in the system, and the compressibility of air in the specimen, the initial volume gauge reading must be corrected by assuming that volume changes in the first few minutes of the test are proportional to the square root of time.

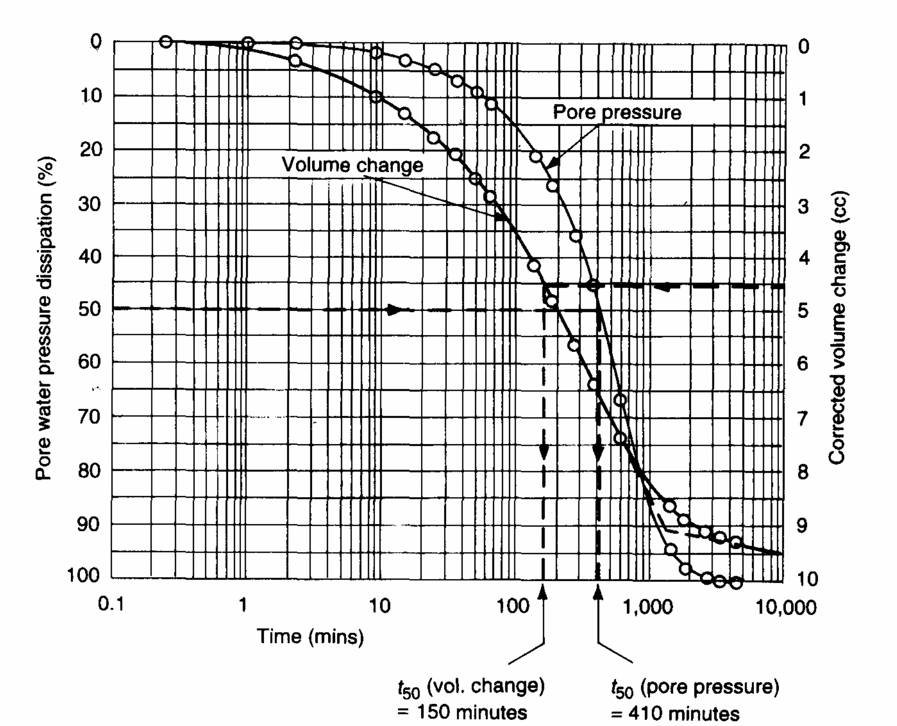


Fig. 8.21 Typical triaxial dissipation test result.

The coefficient of consolidation (c_v) can be determined by matching the theoretical relationships between pore pressure dissipation, volume change and the logarithm of time at the 50% point. Theoretically, for vertical drainage to one end of the specimen:

$$t_{50} = \frac{0.197H^2}{c_v} \text{ at 50\% volume change} \tag{8.18}$$

$$t_{50} = \frac{0.38H^2}{c_v} \text{ at 50\% pore pressure dissipation} \tag{8.19}$$

where H = specimen height. The coefficient of compressibility (m_v) is calculated for each stage as in the oedometer test, as:

$$t_{50} = \frac{\Delta e}{(1+e_\circ)}\frac{1}{\Delta p} = \frac{\Delta V}{V_\circ}\frac{1}{\Delta p} \tag{8.20}$$

where Δe = void ratio change, e_0 = initial void ratio, Δp = pressure change causing ΔV volume change, and V_0 is the initial volume of the specimen.

The triaxial dissipation test is time consuming (no filter drains can be used), but relatively straightforward. Pore pressure measurement should be carried out by electrical pressure transducer and de-airing of the triaxial system should be thorough.

Because of the relatively long period of testing, even minor leaks will obscure the soil behaviour and may not become obvious until some time after the start of the test. Similarly, because the normal latex rubber membrane is slightly permeable to water and much more permeable to air, when tests of more than a few hours duration are to be carried out on unsaturated specimens a thin Butyl rubber sleeve should be placed around the specimen beneath the latex rubber membrane. Because small pressure and volume changes are significant in the later stages of this test, it is important that it is carried out in a temperature-controlled environment so that expansion of the measuring system and of the cell relative to the specimen do not obscure the consolidation process.

Hydraulic oedometer test

The consolidation of large specimens can be carried out in the hydraulic oedometer (Rowe and Barden 1966; Head 1986). This apparatus prevents lateral strain by confining the specimen in a bronze cast ring, and provides vertical stress through a pleated bellows-like rubber membrane (the rubber 'jack'), and is restrained at the top and bottom by thick metal plates bolted to the bronze ring (Fig. 8.22). Hydraulic oedometers are available for specimens of 76mm, 152mm and 254mm dia. A 254mm dia. specimen may have a thickness of 75—100mm.

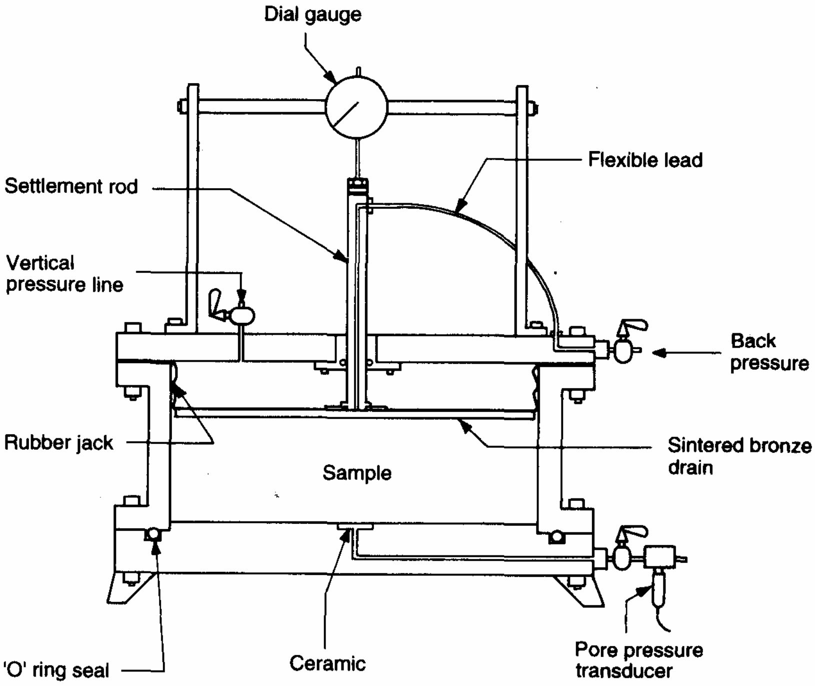


Fig. 8.22 The hydraulic oedometer (Rowe and Barden 1966).

The hydraulic oedometer test has most of the advantages of the triaxial consolidation test in that pore water pressure can be controlled by a constant pressure source through the top drain, and pore water pressure measurements can be made at small ceramics flush mounted in the base plate and connected to pressure transducers.

In addition, volume change measurements may be made by monitoring the movements of the settlement rod which brings the back pressure line through the top plate, or by measuring the movement of the water in the back pressure line with a volume gauge. Because of the use of a rubber jack, high vertical stress levels can be applied to the specimen without the need for a loading frame.

The hydraulic oedometer cell can be used with at least four types of specimen drainage: drainage may be vertical to a single porous sintered bronze plate beneath the rubber jack, or porous plates may be provided at the top and bottom of the specimen in which case mid-plane pore pressure dissipation cannot be measured. Horizontal drainage may be used either by augering a sand drain in the centre of the specimen (inward drainage) or by placing a 1.5mm thick porous plastic material at the periphery of the specimen. Because of the tendency of soils to be layered, their horizontal coefficients of permeability and consolidation will often be many times greater than in the vertical direction. The ability of the hydraulic oedometer to test with vertical compression and horizontal drainage (as might happen beneath an embankment on a limited depth of soft alluvium) is a major advantage.

Permeability tests

Laboratory determinations of the permeability of granular soils can be made using the constant head and falling head permeameter tests (for example, Akroyd 1964; Vickers 1978; Head 1982; BS 1377:part 5:1990, clause 5). For granular soils any values of permeability must be regarded as approximate, since several important factors affect the accuracy of these tests.

First, it is difficult, time-consuming and expensive to obtain even relatively undisturbed specimens of granular soil. Such specimens are rarely available, and as a result disturbed samples must be recompacted to form the test specimen. Differences in porosity, particle orientation, particle size arrangements and flow direction between the specimen and the field situation are inevitable. Further problems may arise because of air in tap water collecting and occluding pores of the soil, and because the testing system may restrict flow as much as the soil if the soil permeability is high. Finally, it should be noted that the viscosity of water is temperature dependent, and most laboratory determinations of coefficient of permeability will not be carried out at soil temperature.

Cohesive soils can be tested for coefficient of permeability in the laboratory, and indeed it was for this purpose that Terzaghi (1923) produced the one-dimensional consolidation theory. Terzaghi noted that smear on the specimen boundaries greatly affected the measured soil permeability in his permeameter tests, and used an oedometer test in order that all water flow would occur out of the sample. Thus the coefficient of permeability can be obtained from triaxial or hydraulic consolidation tests since:

$$k = c_v m_v \gamma_w \tag{8.21}$$

where k = coefficient of permeability, c_v = coefficient of consolidation, m_v = the coefficient of compressibility, and γ_w = density of water.

Where the coefficient of permeability is required with greater accuracy, determinations for clays can be made under constant head gradient either in the triaxial apparatus (Bishop and Henkel 1962), or in the hydraulic oedometer (Rowe and Barden 1966; Wilkinson 1968; Vickers 1978). The specimen can be subjected to a total stress level approximating to that in the ground, and the pore pressures applied at each end of the specimen can be arranged to give an average equal to the field pore pressure. In this situation the accuracy of the test is very much affected by the differences in effective stress across the specimen. The applied pressure difference should be kept to less than 10% of the average effective stress on the specimen.

Inevitably some changes of effective stress are introduced by these tests, because even if the pressure difference driving the water could be kept very small, the horizontal *in situ* stress on the specimen could not normally be accurately predicted. Changes of effective stress at the start of the test introduce consolidation or swelling, or both, and the test must therefore be run until steady flow is achieved. As with the *in situ* permeability tests described in a previous chapter, the rate of flow can be plotted as a function of the inverse of the square root of time elapsed since the start of the test.

Chemical tests

During site investigation it is often necessary to carry out laboratory tests to determine the effects of the sub-soil or groundwater on concrete to be placed as foundations. Chemical tests may also be used to check the soundness of aggregates for concrete or soil cement, to determine if elecrolytic corrosion of metals will take place, or simply to act as index tests.

The effects of aggressive ground are numerous. Details can be found in Neville (1977), BRE Digest 250 (1981), Tomlinson (1980) and BS 5930:1981. The available tests include those listed in Table 8.7.

Table 8.7 Available chemical tests

Test	Source
Organic matter content	BS 1377:part 3:1990, clause 3
Loss on ignition or ash content	BS 1377:part 3: 1990, clause 4
Sulphate content of soil and groundwater	BS 1377:part 3:1990, clause 5
Carbonate content	BS 1377:part 3:1990, clause 6
Chloride content	BS 1377:part 3: 1990, clause 7
Total dissolved solids	BS 1377:part 3:1990, clause 8
pH value	BS 1377:part 3: 1990, clause 9
Resistivity	BS 1377:part 3:1990, clause 10
Redox potential	BS 1377:part 3:1990, clause 11

In the UK, sulphates are widespread in such soils as the London clay, the Lias clay and the Oxford clay. It is therefore good practice to insist on the analysis of representative soil and groundwater samples whenever foundations are being considered. Aqueous solutions of sulphates will attack the hardened cement in concrete, leading to chemical changes which are associated with a large volume increase. This increase of volume causes cracking and spalling. If fresh sulphates can readily move to the concrete, the speed at which deterioration takes place will be accelerated. Insoluble sulphates in the ground are not a problem.

The rate at which sulphate attack can occur is a function of the type and concentration of the sulphates, the amount of groundwater movement, the permeability of the concrete, the type of cement and the type of structure in which it is to be used. Where groundwater will be encouraged to travel along the face of a structure (such as a basement) the concrete will be at a higher risk than where groundwater is static. The normal method of avoiding problems due to sulphates is to ensure that the concrete is dense and impermeable, with a sufficiently high cement content.

It should be noted that while *BRE Digest* 250 and Tomlinson (1980) give recommendations based on the results of tests on a 2:1 water:soil aqueous extract, BS 1377:1975 test 10 uses a 1:1 water:soil extract.

The effect of acid attack on concrete is discussed by Gutt and Harrison (1977). They point out that it is not reasonable to make recommendations for the type of cement or concrete based solely on the knowledge of the pH value of a particular soil. The risk of acid attack should be assessed from pH data, depth to water table, the likelihood of water movement, the thickness of concrete, and whether it is subject to any hydrostatic head. Examples of low and high risk conditions are given below.

1. *Low risk*. pH 5.5—7.0, stiff unfissured clay soil with water table below foundation level.
2. *High risk*. pH < 3.5, permeable soil with water table above foundation level and risk of groundwater movement.

In high risk conditions supersulphated cements or protective impermeable membranes are used.

Sulphate content, chloride content, and organic matter content in aggregate or materials intended for use as soil cement can seriously affect the behaviour of the finished product. A part from any damaging chemical effects these materials have very low strengths in their solid form. When dispersed throughout the mix, high organic contents in material used for soil cement can interfere with hydration, while chlorides may lead to unsightly efflorescence and in large quantities will attack the steel in reinforced concrete and cause rapid deterioration owing to the spalling of the cover. Sulphate contamination of aggregate will retard the set.

Organic contents are also of use in classifying organic soils such as peats. For most purposes the determination of 'loss on ignition' or ash content is sufficient, but it should be remembered that this method tends to yield organic contents which may be up to 15% too high because the oven-dried specimen is fired at about 800—900°C and clay minerals and carbonates are altered. Classification of chalk has been carried out partly on the basis of carbonate content, since this can similarly be used to determine the impurities it contains.

CP 1021 makes recommendations for cathodic protection. Metals exposed to soil and water may be subjected to corrosion as a result of the formation of elecytrolytic cells, either because of the presence of dissolved oxygen and different types of metals, or as a result of different air contents or densities in the soil. In addition, in near neutral soils sulphate-reducing bacteria may lead to cathodic attack which is particularly aggressive to iron and steel.

Soil conditions which may lead to corrosion may be detected on the basis of a low apparent resistivity. CP 1021 suggests that soils with an apparent resistivity of less than lOΩm will be highly corrosive. Alternatively pH value, redox potential, and the presence of soluble salts may be used as a guide. In most situations involving the installation of steel into disturbed soils (for example, piles at depth) electrolytic corrosion is not a problem because oxygen is not available. In shallow anaerobic situations however, if the soil is near neutral then sulphate reducing bacteria may attack. A low redox potential indicates the reducing conditions under which the bacteria will flourish; bacteriological analysis will be necessary when these conditions are encountered in recent organic deposits such as tidal mud flats or rubbish tips.

ACCURACY AND MEASURING SYSTEMS

When setting up or using a laboratory it should never be assumed that even the simplest equipment works as intended. In the past, the authors have encountered manufacturers' weights which were more than 8% different from their claimed values, ovens which could not maintain a constant temperature to better than a 30°C range, and pressure transducer/digital readout systems with a temperature fluctuation of 25 kN/m2 °C. Even proving rings, which have been calibrated by a manufacturer immediately prior to recalibration, have been found to be in error by up to 50%. The general requirements for apparatus, instrumentation, calibration and sample preparation are given in BS 1377:part 1:1990.

These types of fundamental inaccuracy must be guarded against by the frequent checking of equipment against standards. Balances should be frequently serviced, records should be made of the diurnal temperature fluctuations of both ovens and of the laboratory. If repeatable measurements of pressure, permeability and compressibility are to be made, then it is necessary to control laboratory room temperature to about 1°C.

As measuring systems in the more complex tests become more sophisticated, and use more electronics, the need for accurate sources of load and pressure in the laboratory has increased. The introduction of the electronic pressure transducer (Whitman *et al.* 1961) and the internal load cell have provided considerable potential improvements in accuracy, but unless calibration is frequent these improvements cannot be guaranteed. Fortunately, systems such as the Imperial College/Budenberg dead weight devices are readily available and give the high level of accuracy required.