Chapter 6

Sampling and sample disturbance

INTRODUCTION

Sampling is carried out in order that soil and rock description, and laboratory testing can be carried out. Laboratory tests (Chapter 8) typically consist of:

1. index tests (for example, unconfined compressive strength tests on rock);
2. classification tests (for example, Atterberg limit tests on clays); and
3. tests to determine engineering design parameters (for example strength, compressibility, and permeability).

Samples obtained either for description or testing should be representative of the ground from which they are taken. They should be large enough to contain representative particle sizes, fabric, and fissuring and fracturing. They should be taken in such a way that they have not lost fractions of the *in situ* soil (for example, coarse or fine particles) and, where strength and compressibility tests are planned, they should be subject to as little disturbance as possible.

Generally, samples of two types are specified — undisturbed and disturbed samples. Undisturbed samples are generally taken by cutting blocks of soil or rock, or by pushing or driving tubes into the ground. Disturbed samples are taken from cuttings produced by the drilling process. A large number of samplers and sampling methods are available, but before a suitable technique can be selected it is always necessary to consider whether the sample size will be adequate, and whether the most suitable method of sampling has been selected, to ensure that sample disturbance is sufficiently small.

SAMPLE SIZES

The size selected must be large enough to ensure that the sample contains a representative distribution of the particle sizes that are in the ground, and is large enough to ensure that:

1. samples with representative fabric can be tested, to give a realistic picture of consolidation behaviour;
2. samples contain sufficient fissuring or jointing to give strengths and compressibilities representative of the soil or rock mass; and
3. enough material will be available for the tests that are envisaged.

Representative particle sizes

It is necessary to take sufficiently large samples to ensure that any particle size distribution tests carried out are representative of the ground from which the sample has been taken, and to ensure that other testing will give representative results.

It is normally considered adequate to take samples which have a minimum dimension of the order of 5—10 times the maximum particle size of the soil. In practice this means that very different sizes of specimen are required for fine and coarse soils, Table 6.1.

Table 6.1 Sample size necessary for particle size distribution tests

Soil type	Maximum soil particle size (mm)	Minimum sample dimension (mm)	Minimum sample mass
Silt/clay	0.06	0.3—0.6	<0.lg
Sand	2	10—20	2—15g
Fine gravel	6	30—60	50—400g
Medium gravel	20	100—200	2— 16 kg
Coarse gravel	60	300—600	50—400 kg
Cobbles	200	1000—2000	2—15t

It is clearly unrealistic to expect a geotechnical laboratory to be able to handle and test the very small quantities of soil required for clays, silts and sands, so in these cases it is the minimum quantities required for various test procedures which are the controlling factor. These are discussed below. In very coarse soils it is clearly unlikely that any investigation can provide sufficient material for a full characterization. Trial pits can be used to make visual estimates of grading, but boreholes and samples from boreholes, will not give reliable results. BS 5930: 1981 states that 'none of the sampling methods ... is suitable for this type of ground. Disturbed samples are only class 5 (grading incomplete) because the fine fraction has been washed out and the coarse fraction may have been broken up by the use of a chisel'. The authors' experience of pile construction in cobbly soil suggests that site investigation data may often be unreliable.

In practice, as will be seen later in this chapter, and in Chapter 8 (Laboratory testing), most routine strength and compressibility testing is carried out using test specimens of standard sizes. Here, then, the question is not what size sample must be taken, but whether the results of tests carried out on standard-sized specimens will give reliable and representative results. In some cases, where the economic gains are sufficiently large, it may be possible to use larger-than-standard specimen sizes. This will not generally be possible.

Rate of consolidation

Rowe (1968a, b) has considered the effects of fabric on the results of laboratory tests. In assessing the need for sand drain installations he considered coefficient of consolidation (c_v) values obtained from conventional 76mm dia. x 19mm high oedometer tests, from 250mm dia. x 125 mm high consolidation tests, from *in situ* permeability tests and from field records. A selection of these results is given in Table 6.2 which illustrates just one aspect of sample size effect.

Rowe (1968b) made the following conclusions.

1. 76 mm oedometer tests could give completely false coefficient of consolidation values, except in uniform clays. Such materials are rather unusual.
2. 250mm dia. by 125 mm thick specimens are large enough to represent most fabrics, provided the laboratory test direction is relevant to the field case.
3. Because the coefficient of compressibility (m_v) is not very sample size dependent for most softer soil deposits, c_v may be derived with reasonable accuracy from small laboratory tests (for m_v) combined with permeability values from constant head *in situ* tests, using the equation:

$$c_v = \frac{k}{m_v \gamma_w} \qquad (6.1)$$

Table 6.2 Effect of fabric and test size on coefficient of consolidation values

Site and soil type	Coefficient of consolidation, c_v (m^2/year) Oed. (76mm)	Rowe cell (254 mm)	Piezo. records	*In -situ* perm.	$\frac{c_v \text{ in situ}}{c_v \text{ oed.}}$
Staunton Harold			Very high	6132	
Multi-fissured weathered shale	5.6h	2973	None	334	1100
Derwent	1.11 v				
Clay coarsely layered with silt and sand	?2.6h		None	836	321
Frodsham					
Estuarine clay (b2) with vertical rootlets	9.3v	185—1858	None	930	100
Selset					
Uniform boulder clay	1.6h		5.6		3.5

Undrained shear strength

The other very serious effect of sample size is to modify undrained shear strength, as measured in either the field or the laboratory. The effects of sample size on undrained shear strength have been reported by Bishop and Little (1967), Agarwal (1968) and Clapham (1978), all studies involving the London clay. Fissures in the London clay provide planes of weakness; larger samples are more likely to contain fissures in the preferred failure direction of a tested sample. The indications are that for a material with intact blocks estimated at 40mm the test specimen size should be about 300mm diameter. For other materials it is suggested that the minimum test specimen diameter should be six to eight times the intact block size found in the soil (see Table 6.3).

Table 6.3 Sample size effects: fissured London clay

Method of assessment of undrained shear strength	Ratios of undrained shear strengths: Bishop and Little (1967)	Agarwal (1968)	Clapham (1978)
Back-analysis of slope failure			0.63‡
Horizontal 600mm x 600mm shear box test	1.00*		100†
304.8mm dia. triaxial tests		1.00	1.04
152mm dia. triaxial tests		0.98	1.17
101.6mm dia. triaxial tests		1.01	1.29
38.1mm dia. triaxial tests	1.51*	1.52—2.21	2.25
Intact clay (c. 15mm dia. x 30mm high)	3.00*	4.68	7.54

* Values corrected for: (i) rate of testing; (ii) anisotropy and orientation.
†Value corrected for orientation of failure plane.
‡ Based on isotropy, with no correction for rate effects applied to test results.

In the UK, undrained shear strength is most commonly measured by 38mm or 102 mm diameter triaxial tests on 102 mm diameter tube samples. The data in Table 6.3 show that the average shear strength measured on 38mm specimens can be expected to be of the order of $1^{1/2}$—2 times that measured on 102mm diameter specimens. In addition, however, the scatter of results from 102mm diameter specimens will be greater, so that more must be tested before a reliable value of strength is obtained. An example of both the reduction of average undrained shear strength, and the reduction in the scatter of individual results, with size can be seen in Fig. 9.31. Since, both because of fissure fabric and because of tube sampling disturbance (*inter alia*), undrained shear strength is not a fundamental soil parameter, great care must be taken during geotechnical design to match the commonly used empirical design methods with the appropriate method of determining strength. The most important aspect of this for fissured clays is to use a specimen size which is similar to that used in the original design method. For some design methods 38mm specimens should be used (e.g. shaft adhesion on piles, and pressures on braced excavations, while for others the largest size possible is required (e.g. short-term slope stability).

Required volume of material for testing programme

A further consideration in fixing sample sizes is the standard test specimen sizes in use. In the UK specimen sizes commonly used are shown below.

Compressibility characteristics	
Oedometer	76mm dia. x 19mm high
Triaxial cell	102 mm dia. x 102 mm high
Hydraulic consolidation cell	up to 254mm dia. x 100—125mm high
Triaxial compression tests	
Small specimens	38mm dia. x 76mm high
Large specimens	102mm dia. x 204mm high
	or 152mm dia. x 305mm high
Direct shear tests	
Small specimens	60mm x 60mm in plan
Large specimens	305 mm x 305 mm in plan

Small triaxial specimens are normally tested in groups of three, all of which should be obtained from the same level in the sample in order that they are as similar as possible. Three 38mm dia. specimens can be obtained from a 102 mm dia. sample.

Soil testing equipment manufactured in the USA uses the following specimen sizes.

Compressibility characteristics	
Consolidometer	
(large specimen)	113 mm dia.
(standard size)	64mm dia.
Triaxial compression tests	
Small specimens	36mm dia. x 71mm high
Medium specimens	71 mm dia. x 142mm high
Large specimens	102mm dia. x 204mm high
	or 152mm dia. x 305 mm high
Direct shear tests	
Cylindrical specimens	63.5 mm dia.
Square specimens	51mm x 51 mm

Three 36mm dia. (1.4in. dia.) specimens can be obtained from either 89mm (3.5 in.) dia. samples or 102 mm (4 in.) dia. samples.

As noted above, when discussing the need for samples to contain representative particle sizes, in many cases it is the minimum quantity of soil required for a particular test procedure which will dictate the volume or mass that must be obtained. BS 5930: 1981 suggested sample sizes should be determined on the basis both of soil type and the purpose for which the sample was needed (Table 6.4).

Table 6.4 Mass of disturbed soil sample required for various tests from BS 5930: 1981

Testing	Soil type		
	Clay, silt or sand (kg)	Fine and medium gravel (kg)	Coarse gravel (kg)
Moisture content, Atterberg limits, sieve analysis, chemical tests	1	5	30
Compaction tests	25—60	25—60	25—60
Soil stabilization tests	100	130	160

Table 6.5 is based upon the more recent requirements of BS 1377:1990, and is relevant to disturbed and undisturbed samples required for index, classification, and compaction testing. The total mass of sample required should be obtained by adding together the masses for the tests envisaged. The total mass required should not be less than will ensure that the sample is representative (see earlier), and it should be borne in mind that the figures given in the table are maxima, and that once the precise type of test is defined it may be possible to use considerably less material. For example, the actual mass required for a compaction test will vary between 10 g and 80 g, depending upon the type of compaction test and the susceptibility to crushing of the soil.

Table 6.5 Mass of disturbed soil sample required for various tests based on BS 1377:1990. Note that, with the exception of plasticity tests and sieve analyses, the tests described in BS 1377 are unsuitable if >10% of the soil is retained on the 37.5 mm sieve

Testing envisaged	Soil type		
	Fine-grained†	Medium-grained†	Coarse-grained†
Moisture content	50 g	350 g	4kg
Atterberg limits	500 g	1 kg	2kg
Specific gravity	1.5 kg	2kg	4kg
Sieve analysis	150g	2.5kg	17kg
Sedimentation	250 g	250 g*	250 g*
Chemical tests	150g	600g	3.5 kg
Resistivity	12kg	15kg	20kg
Compaction tests	80 (50) kg	80 (50) kg	80 (50) kg

* Sufficient material to give the stated mass of fines.
†Fine-grained 90% passing 2mm: medium-grained 90% passing 20mm: coarse-grained 90% passing 37.mm.

In current UK practice, two sizes of disturbed samples are usually specified:

1. small disturbed samples (‘jars’) 0.5—1.0kg;
2. large disturbed samples (‘bulk bags’) 25—50 kg.

Both types of sample may be placed either in plastic bags, or rigid containers (such as glass jars or boxes). The soil should be packed in such a way that as little air is included as possible, and the containers should be sealed so as to be airtight.

These sizes allow only limited testing. Small disturbed samples can only be used for plasticity tests, particle size analyses, and the determination of the specific gravity and chemistry of fine-grained soils. Samples of coarse-grained soil of sufficient size to meet the requirements of compaction tests are rarely obtained during routine borehole investigations.

SOIL DISTURBANCE

The availability of good engineering parameters for geotechnical design depends on careful testing. Testing may be carried out in the laboratory or in the field, but in either case the most important factor controlling the quality of the end result is likely to be the avoidance of soil disturbance.

Soil disturbance can occur during drilling, during sampling, during transportation and storage, or during preparation for testing. Any sample of soil being taken from the ground, transferred to the laboratory, and prepared for testing will be subject to disturbance. The mechanisms associated with this disturbance can be classified as follows:

1. changes in stress conditions;
2. mechanical deformation;
3. changes in water content and voids ratio; and
4. chemical changes.

In their extreme, changes in stress conditions take the form of the reduction of the total horizontal and vertical stresses from their *in situ* value, to zero, on the laboratory bench. Mechanical deformations are shear distortions applied to the soil sample, for example by tube sampling. Changes in water content can occur as an overall swelling or consolidation of the soil sample, or a redistribution of moisture in response to pore- pressure gradients. Chemical changes may occur in the pore water or the soil, and may result from contact with drilling fluid or with sampling tubes.

These mechanisms can occur at different stages during the process of the investigation, and while some occur very quickly, others take considerable time. Some types of disturbance are unavoidable, but many can be minimized or even eliminated if the mechanisms of disturbance are understood and common sense is used to optimize the processes involved. The importance of a particular type of disturbance will depend not only upon the sampling processes being used, but also upon the type of soil being sampled. However, the unifying factors are that sampling disturbance affects the effective stress state of a soil sample, and in addition (and more seriously) can also affect its structural bonding.

Table 6.6 gives a list of the main causes of disturbance at various stages of a site investigation.

Table 6.6 Principal causes of soil disturbance

Before sampling	During sampling	After sampling
Stress relief	Stress relief	Stress relief
Swelling	Remoulding	Migration of water within the sample
Compaction	Displacement	Loss of moisture
Displacement	Shattering	Freezing
Base heave	Stones at the cutting shoe	Overheating
Piping	Mixing or segregation	Vibration
Caving	Failure to recover	Chemical changes Disturbance during extrusion

Stress relief

A reduction in the total stress applied to the soil being sampled is an inevitable product of the processes involved. Making a borehole reduces the total stresses at its base. Using sampling tubes with inside clearance reduces the lateral total stresses, and extrusion of the soil during specimen preparation will usually bring the total stresses in all directions to zero. In the ground, the total stresses in the horizontal and vertical directions will not normally be the same; that is there will be a deviatoric stress applied to the soil. The process of total stress relief may have two components:

1. the removal of the deviatoric stress (termed perfect sampling' by researchers); and
2. the reduction of the mean total stress to zero (termed 'block sampling' by researchers).

Skempton and Sowa (1963) examined the effects of perfect sampling in remoulded Weald clay specimens, in a series of experiments which attempted to follow a simple field total stress path for soil loaded by, for example, a foundation ('ground') and for soil subjected to total stress relief, isotropic stress increase, and monotonic deviatoric stress increase ('sample'). Figure 6.1 shows the stress paths for the two parts of the experiment. The undrained shear strength of the 'sample' was typically only 1.5% lower than that of the 'ground' although the stress paths were entirely different. Skempton and Sowa's experiments were conducted on a remoulded clay of medium plasticity (w_L = 46%), and as will be seen later, much of the effective stress applied under K_0 consolidation was maintained when total stresses were removed.

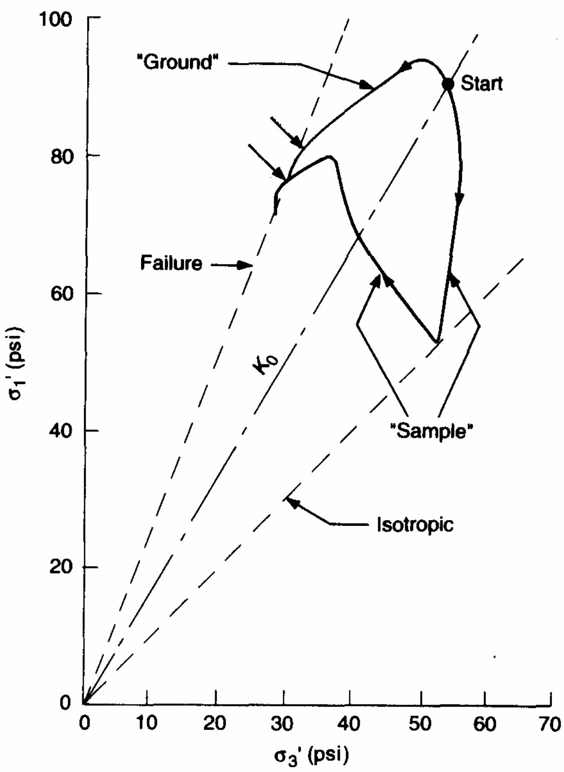


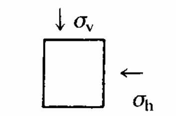
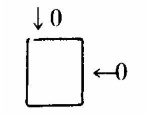
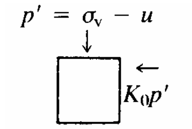
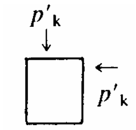
Fig. 6.1 Stress paths for 'ground' and 'sampling' after Skempton and Sowa (1963).

Swelling

Swelling occurs as a consequence of stress relief. In response to the reduction of applied total stresses, the pore water pressures in a soil will reduce and may normally be expected to become negative. If the soil is coarse-grained, it will have a high coefficient of permeability and a large average pore size, and water or air will rapidly penetrate it and dissipate the negative pore pressures. Thus, with total and effective stresses reduced to zero, a granular soil has little strength and is very difficult to sample or prepare for laboratory testing.

In a cohesive soil, a small average pore size normally precludes the penetration of air. The low permeability of clay means that a considerable period of time may be required for water to penetrate and dissipate the negative excess pore pressures set up in the mass of soil during drilling for sampling. Skempton and Sowa (1963) considered the stress changes occurring in a saturated clay as a result of stress relief. In summary these stresses might be as shown in Table 6.7.

Table 6.7 Stress changes occurring in a saturated clay

Stresses	Soil in ground	After sampling
Total stresses	 ↓ σ_v ← σ_h	 ↓ 0 ←0
Pore pressure	+ u_0	+ u_k
Effective stresses	$p' = \sigma_v - u$  ↓ ← $K_0 p'$	p'_k  ↓ ← p'_k = $-u_k$

Now the pore water pressure after stress relief (Uk) can be assessed using Skempton's pore pressure parameters (Skempton 1954, 1960a):

$$\Delta u = B[\Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3)] \tag{6.2}$$

For a normally consolidated clay (i.e. $\sigma_v > \sigma_h$):

$$\Delta\sigma_1 = -\sigma_v \text{ and } K_0 = \frac{(\sigma_h - u_0)}{(\sigma_v - u_0)} \tag{6.3}$$

$$\Delta\sigma_3 = -\sigma_h \tag{6.4}$$

For a saturated clay B=1, therefore:

$$\Delta u = u_k - u_0 = \Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3) \tag{6.5}$$

Under elastic soil conditions it can be shown that A =⅓ , and therefore the above equation can be rewritten in terms of the pore pressure expected from an 'elastic clay' and the difference of a real soil from this value, i.e.

$$\Delta u = \frac{1}{3}[(\Delta\sigma_1 + \Delta\sigma_3) + (A - \frac{1}{3})(\Delta\sigma_1 - \Delta\sigma_3)]$$

$$\Delta\sigma_1 - \Delta\sigma_3 = -p'K_0$$

therefore:

$$\Delta u = -p'\left[\frac{(1+2K_0)}{3} + u_0\right] - p'(A - \frac{1}{3})(1 - K_0) \tag{6.6}$$

Now:

$$-p'_k = -u_k = -(\Delta u + u_0) = p'\left[\frac{(1+2K_0)}{3} + \left(A - \frac{1}{3}\right)(1 - K_0)\right] \tag{6.7}$$

If, as is approximately the case for heavily overconsolidated clays, the material behaves elastically during unloading:

$$u_k = -p'\frac{(1+2K_0)}{3} \tag{6.8}$$

Skempton and Sowa (1963) carried out experiments on Weald clay to find the differences between predicted and observed effective stress levels after stress relief under laboratory conditions. The resulting values are shown in Table 6.8, where $\overline{p}'$ is the average effective stress on the soil before stress relief. Thus for this case u_k equalled $0.6\overline{p}'$ to $0.7\overline{p}'$.

Table 6.8 Differences between predicted and observed results for Weald clay

	Elastic prediction	Remoulded clay results
$\overline{p}'_k / \overline{p}'$	0.73	0.58
$p'_k / \overline{p}'$	1.00	0.80

Compaction

In granular soil, permeability is high, and therefore vibrations and compressive forces applied to the soil, whether in the ground or in the sampling tube, can lead to changes in density. These effects are most severe in loose granular material, where density will be increased. Compaction leads to changes in the effective strength and stiffness parameters of the soil.

Soil disturbance during drilling

Swelling can occur at the base of the borehole before insertion of a sampler tube, during the taking of a sample, and after sampling when the soil is inside the sampler tube. As examples, the ingress of water to material in the base of a borehole in London clay makes the recovery of soil using a claycutter more difficult, presumably because of the loss of shear strength as a result of swelling; in contrast, a waiting period after sample driving is sometimes used to improve the chances of recovery of London clay in an open-drive sampler with inside clearance. In the second case, swelling increases the diameter of the clay core inside the tube while increasing the effective stress level at the clay/cutting shoe contact.

The amount of swelling that can occur is proportional to the change of total stress occurring at the base of a borehole. Thus if the borehole is substantially empty of water there is likely to be more swelling than if the borehole is kept full of mud or water. Total vertical stress changes can effectively be halved by keeping boreholes full of water. The higher the water-table and the softer the soil, the greater is the benefit of a water filled borehole. Figure 6.2 shows the results of analyses (assuming elastic soil with K_0 equal to 1) to calculate the variation of pore pressure change caused by borehole stress relief with depth below the base of the hole. It can be seen that large negative pore pressures will be induced, and that these will vary with depth. The vertical extent of pore pressure decrease (and therefore swelling) will be about one borehole diameter.

The factors which complicate the control of swelling are time and water-table position. If drilling and sampling take place quickly, then little time will be available for water to penetrate the soil. Swelling will be limited. Above the water-table there may be relatively little water available in the borehole, and swelling may be slowed down. The recommendations of Hvorslev (1949) and Idel *et al.* (1969) with regard to the use of fluid filled boreholes are given in Table 6.9.

Table 6.9 Recommendations for the use of fluid filled boreholes

	Hvorslev (1949)	Idel *et al.* (1969)
Boring above groundwater level	Keep borehole dry, or use drilling fluid (mud). Water is not permitted.	Use water balance.
Boring below groundwater level	Fill borehole with water or mud, at least when in soft or cohesionless soils. In stiff soils, borehole may be kept dry, but it must then be completely dry.	Use water balance.

Hvorslev (1949) commented that 'swelling … may require considerable time for full development'. Ward (1967) however stated, with regard to the taking of block samples from the Ashford Common Shaft:

> I was never convinced that we ever appreciated what the London clay was like before the shaft was dug. We showed, even in the short time required to cut out samples, that the blocks which were integral with the base of the excavation were statistically wetter than the pieces which we trimmed off the surface of the excavation while preparing the block.

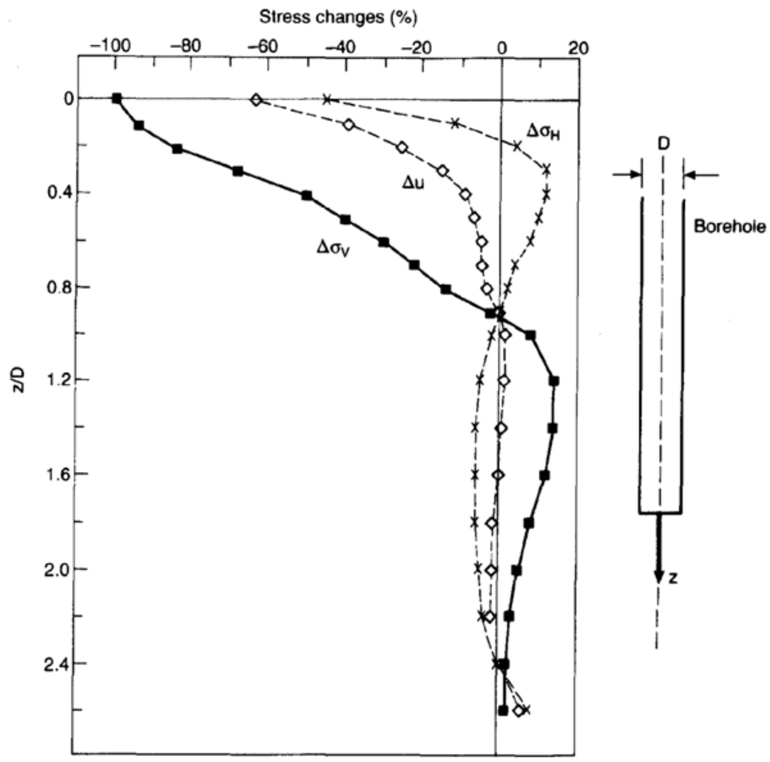


Fig. 6.2 Stress changes below the bottom of a borehole (modified from Galle and Wilhoit (1962) by Hopper (1992)).

If, as is current practice in the UK, water or mud balance is not used in stiff over- consolidated clays, then drilling must occur quickly immediately above each proposed sample location and sampling must take place as soon as that drilling is completed.

Compaction, *remoulding and displacement* of soil beneath or around casing or sampler tubes driven ahead of an open borehole can be minimized if care is taken. Soil displacement can occur as a deliberate method of advancing a borehole; many well-boring rigs operate on the percussion drilling principle, where a heavy drilling bit (referred to as a churn bit) is alternatively raised and dropped by a 'spudding' mechanism. This type of displacement drilling leads to significant remoulding and compression of the soil around and ahead of the bit. The depth affected can be up to three times the hole diameter.

Similar effects can be unwittingly caused during the more common types of site investigation drilling, principally when using augers or light percussion drilling in soft soil. Most rigs using continuous flight augers are capable of providing considerable downwards thrust; the Acker AD II can give up to about l6000lb (7.2t) while the Mobile B53 can give 19000 lb (8.5t). Over-eager drilling can lead to displacement of soil ahead of the auger before the flights have a chance to remove the soil. In very soft clays the soil may block flights and fail to travel up to the ground surface. Soil displacement then becomes inevitable.

Light percussion boring can induce the same sort of problems if casing is advanced below the bottom of the open hole. A plug of soil will form inside the base of the casing and lead to compaction, compression and bearing capacity failure immediately below the bottom of the casing (Fig. 6.3). Casing should never be allowed to go below the bottom of the borehole at any time during drilling; in this case samples taken through the bottom of the casing will probably be highly remoulded if clays, or compacted if sands or gravels.

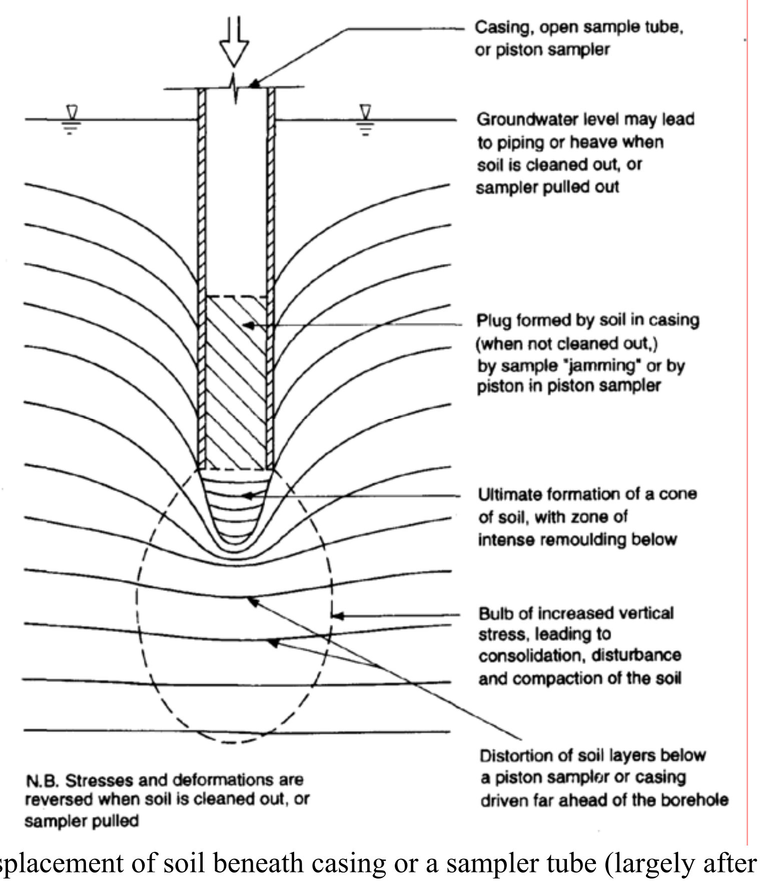


Fig. 6.3 Displacement of soil beneath casing or a sampler tube (largely after Hvorslev 1949).

The problems discussed above occur equally during the taking of samples. Soil must be displaced to allow the penetration of the sampler tube, and if sufficient shear force is generated between the inside of the sample tube and the soil entering it then the sample may 'jam' in the tube.

Base heave, piping and caving are all severe effects of stress relief. Base heave can be thought of as foundation failure under decreased vertical stress, and the effects are broadly the reverse of those produced by displacement drilling. When the total stress relief at the base of a borehole is very great compared with its undrained shear strength, plastic flow of soil may take place upwards into the borehole. This effect may be encouraged when pulling sampler tubes out of the soil at the bottom of a borehole.

Once flow of soil occurs into the base of a borehole, disturbance may then take place for depths in excess of three borehole diameters ahead of the bottom of the hole and its casing, the actual depth being dependent on the volume of soil allowed to enter the hole. Since base heave is a problem in very soft soils, where the water-table will normally be high, the use of either water or mud balance is recommended.

The problem of base and wall instability of boreholes is similar to that of undrained bearing capacity of foundations and the base heave of excavations, which have been the result of considerable research (for example Skempton 1951; Bjerrum and Eide 1956; Britto and Kusakabe 1984). On the basis of this work, Hight and Burland (1990) have concluded that:

1. for an unsupported hole there will be failure of the borehole wall if the undrained shear strength (c_u) is less than $\gamma D/10$, where D is the depth of the hole and y is the average bulk density of the soil above the base of the hole;
2. mud support is helpful in all situations, whereas casing must be continuous, from the top to the bottom of the hole, to be effective;
3. base failure is inevitable in normally consolidated clays; and
4. in lightly overconsolidated clays the factor of safety against base failure will be so low that significant strains will be imposed on the soil immediately beneath the bottom of the borehole.

'*Piping*' is a term used to describe the behaviour of granular soil when its effective confining pressures, and hence strength, are removed as a result of high upward seepage pressures. Under these conditions the individual soil particles are free to move and finer soil particles are carried upwards with the water. The material appears to 'boil'. When a borehole is inducing total stress relief, and water balance is insufficient to prevent high seepage pressure gradients in the soil at the base of the hole, large volumes of fine granular soil may move up into the casing. Soil below the bottom of the casing will be brought to a very loose state.

Piping often occurs when a 'shell' is used without water balance, in conjunction with light percussion drilling. It is particularly troublesome if the soil is already loose, the groundwater table high, and the borehole diameter large. The effects of piping on the quality of soil samples taken from granular soil will not normally be too large while light percussion drilling, because loss of fines would be expected when using a shell. Thus, although bulk and jar samples taken from this type of borehole would normally be considered to be quality class 4 (Rowe 1972) in reality they will often be 'Nonrepresentative' (Hvorslev 1949).

The most serious effects of piping occur because it is normal to use *in situ* tests to determine the design parameters, such as allowable bearing pressure, for a granular soil. In a borehole, the most common test would be the 'Standard Penetration Test' (see Chapter 9), where a 50mm dia. tube is driven into the soil at the bottom of the borehole by repeated blows of a standard weight falling through a fixed distance. The number of blows necessary to drive the tube approximately 300mm is known as the SPT N value, and is used empirically to obtain various soil properties. Piping reduces the density of the soil at the base of the hole, and can therefore give completely false N values; for example, N values have been observed to decrease from 25 blows/300mm to 8 blows/300 mm in sand, which might lead to an unnecessary reduction in allowable bearing pressure for footings from about 250 kN/m^2 to 80kN/m^2. Sutherland (1963) observed the results shown in Fig. 6.4, where piping appears to have reduced N values by a factor of three or four.

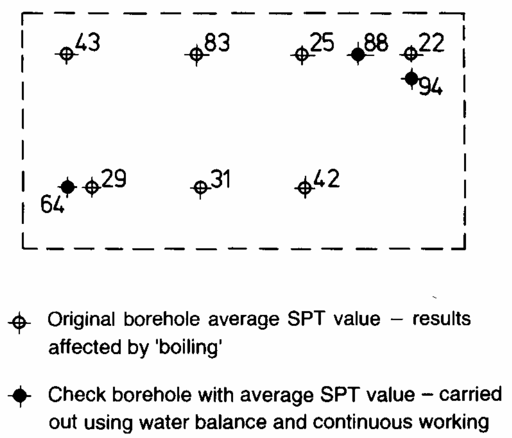


Fig. 6.4 Effects of boiling on SPT 'N' values in fine to medium sand (Sutherland 1963).

Piping can be prevented by giving some thought to its causes. The shell or bailer so often used to make progress in granular soils when drilling with light percussion rigs acts by creating suction on the bottom of the borehole. If the shell is a tight fit in the casing then suction will be large, progress will be fast, and disturbance will be enormous. The International Society for Soil Mechanics and Foundation Engineering has prepared a standard for penetration testing in Europe (1977) which specifies the use of a shell with a maximum diameter not greater than 90% of the inside diameter of the borehole casing. This will considerably reduce suction at the base of the hole, but it will not prevent piping if the natural groundwater level is high.

When the soil is loose and the groundwater table is high, the borehole should be kept full of water in order to ensure that seepage in the soil at the base of the hole occurs in a downward direction. Under this condition piping cannot occur, provided artesian groundwater is not present. When artesian conditions occur, casing will have to be extended above ground level and drilling may have to take place from a raised platform if piping is to be prevented.

If piping is not prevented then the depth of soil affected is a function of the casing or borehole diameter. Fletcher (1965) has discussed the development of the SPT, which was originally used by Colonel Charles R. Gow to provide information on the density of soil formations for the purpose of correlation with experience of bored and driven pile design and installation. In the USA, this test initially used a 52mm dia. SPT tool on size 'A' drill rods, in either a 64mm or 102mm casing. The hole was advanced by washboring. British practice currently adopts a minimum hole size of 152 mm; most commonly 204 mm internal diameter casing is used when drilling near to the ground surface in loose deposits. Clearly, British practice is most undesirable because the entire SPT test section can be loosened either by piping, or by stress relief. British SPT N values should be expected, on average, to be lower than values obtained by the American method.

Caving typically occurs when boreholes are advanced into soft, loose or fissured soils. Material from the sides of the borehole collapses into the bottom of the hole and must be cleaned out before sampling can take place. Progress is slowed because more material must be removed from the borehole.

Stabilization of the sides of boreholes is essential in soils which may collapse or slough. It may be carried out by a variety of methods, the most common of which use water, mud, or casing. Water stabilization is the least effective method, and works by reducing the stress level decreases on the sides of the hole. Further benefits come from the elimination of groundwater flow into the sides of the borehole. Water stabilization may work well in soft cohesive alluvial deposits, but it is not successful in a wide range of ground conditions. In partially saturated soils the loss of strength may encourage collapse, and in stiff fissured cohesive soils above the water-table the rate of swelling will be increased.

Drilling mud may be made by mixing bentonite and water in a grout mixer, typically in proportions of about 1:20 by weight. Mud has several advantages over water. It has a higher density and therefore replaces a greater proportion of the stresses originally on the soil. It forms a 'cake' over any surface into which it attempts to seep; this cake is relatively impervious, thus reducing the rate and amount of swelling that can occur. The main disadvantage of mud is its high cost, and there are also problems with its disposal. For these reasons its use is normally restricted to rotary drilling.

The most common method of stabilizing the sides of a borehole is the use of steel casing. Casing has the major advantages of being durable and providing a certain way of preventing collapse.

Two types of casing coupling are in common use; the outside coupling and the flush coupling. The flush coupling is to be preferred because it minimizes the decrease in diameter of hole between adjacent casing strings and also because it suffers less from friction with the soil, thus allowing greater ease of extraction at the end of drilling a hole.

Casing is usually advanced by driving with a heavy weight, such as a sinker bar or hammer. The use of casing can lead to certain types of soil disturbance, such as displacement, compaction, local over-stressing and piping. Alternative effects of caving or collapse of the sides of the borehole however, can be equally severe and difficult to control without casing. Material which falls to the bottom of the hole shortly before an open drive sampler tube is lowered may be sampled and erroneously thought to be representative of soil conditions at that level. Immediately before any sampling is attempted, the depth of the base of the hole should be checked with a weighted tape to ensure that no debris has collected. If the depth of the hole is not equal to the last depth of the drilling tool, the borehole should be cleaned out and its depth checked once more, before a sampler is lowered. Small amounts of debris should be expected at the bottom of a borehole, but its depth should preferably never exceed 100 mm.

Casing is normally fitted with a sharpened edge, or 'shoe' at its base. To minimize disturbance to surrounding soil this shoe should be kept sharp and should have an outside cutting edge. This will ensure that the soil displaced by the casing will be pushed into the borehole, from where it can be removed (Hvorslev 1940).

Soil disturbance during sampling

Each type of sampling will impose a different degree and form of sampling disturbance, but in principle sampling processes can be divided into three broad groups.

1. *Disturbed sampling*. Here there is no attempt to retain the physical integrity of the soil. These types of sample are suitable for classification tests.
2. *Tube sampling*. The soil sample is obtained by pushing or hammering a tube into the ground. Soil is displaced and distorted, to a greater or lesser degree, as the tube enters the ground. There will be stress relief during boring, and during sampling when inside clearance is used. The design of the tube has an important effect on the disturbance of the soil. Tube sampling has, for the past 50 years, been the routine method of obtaining 'undisturbed' samples.
3. *Block sampling*. The sample is cut from the ground, either from the base or side of a trial pit, or as part of a rotary drilling process. Traditionally block samples have been obtained from pits. Carefully controlled rotary drilling, or the use of the Sherbrooke sampler, aims to achieve a similar result. Block samples undergo stress relief, and swelling, but should not be subjected to shear distortions.

This section considers only block sampling and tube sampling.

Block sampling

Block sampling has traditionally involved the careful hand excavation of soil around the sample position, and the trimming of a regular-shaped block. This block is then sealed with layers of muslin, wax and clingfilm, before being encased in a rigid container, and cut from the ground. The process is illustrated in Fig. 6.5. A similar process can be carried out in shafts and large-diameter auger holes.

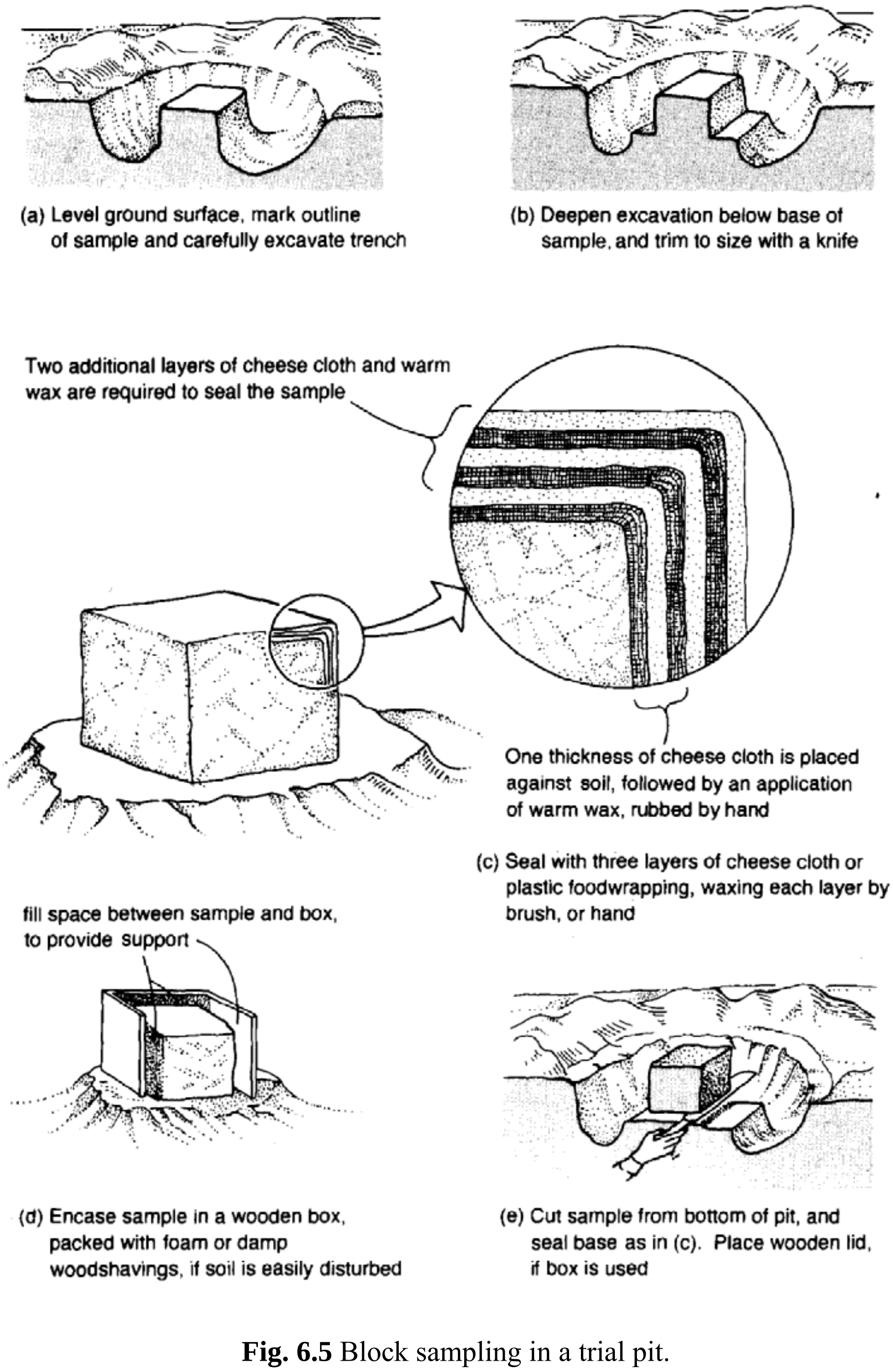


Fig. 6.5 Block sampling in a trial pit.

Trial pits are normally only dug to shallow depths, and shafts and large-diameter auger holes tend to be expensive. Therefore block samples have not traditionally been available for testing from deep deposits of clay. In the past decade, however, there has been an increasing use of rotary coring methods to obtain such samples. When carried out carefully, without displacing the soil, rotary coring is capable of producing very good quality samples. When the blocks are cut by hand then obviously the pit will be air-filled, but when carried out in a borehole it will typically be full of drilling mud.

During the sampling process there is stress relief. At one stage or another the block of soil will normally experience zero total stress. This will lead to a large reduction in the pore pressures in the block. The soil forming the block will attempt to suck in water from its surroundings, during sampling, either from the soil to which it is attached, or from any fluid in the pit or borehole. This will result in a reduction in the effective stress in the block.

In addition, where block sampling occurs in air, negative pore pressures may lead to cavitation in any silt or sand layers which are in the sample. Cavitation in silt and sand layers releases water to be imbibed by the surrounding clay, and the effect will be a reduction in the average effective stress of the block.

Block sampling is an excellent method of ensuring that the soil remains unaffected by shear distortions during sampling, but samples obtained in this way may not (as a result of swelling) have effective stresses that are the same as those in the ground. Therefore the strength and compressibility of the soil may be changed. This should be allowed for either by using appropriate reconsolidation procedures, or by normalizing strength and stiffness, where appropriate, with effective stress.

Tube sampling

Tube sampling is used in almost all routine ground investigations. It is carried out by pushing a tube into the ground, without rotation, thus displacing soil. This displacement introduces shear distortions into the ground, and these can have two effects:

1. the effective stress of the soil is changed; and
2. bonding between soil particles (termed 'structuring') is broken.

These effects are in addition to those induced by stress relief and swelling, described above for block samples, which occur in tube samples as a result of borehole disturbance and the design of the sampler.

Baligh (1985), Chin (1986), Baligh *et al.* (1987), Siddique (1990), Hajj (1990) and Hopper (1992) have studied the penetration of samplers as a continuous flow problem. Early work by Baligh and his co-workers showed (Fig. 6.6) that the strains imposed on the centreline of a soil sample as it travels into a sample tube are initially compressive, and then extensive. The magnitude of the strains for the simple tube geometry that they simulated (the so-called 'simple sampler') depended on the thickness to widthratio (B/it) of the sampler. La Rochelle *et al.* (1987) believed that the idealization of tube geometry used in these early, pioneering works is not realistic, and stated that there is strong evidence that the detailed geometry of the cutting shoe of a tube sampler has a very large influence on the quality of sample obtained. Subsequent work by Siddique (1990) and Hopper (1992) has shown this to be true. Flat-ended samplers (Siddique 1990) and the simple sampler (Baligh *et al.* 1987) represent the extremes of poor design, and good cutting shoe design can very greatly reduce tube sampling disturbance, by reducing the magnitude of shear strains applied to the soil.

Baligh *et al.* (1987) and Siddique (1990) applied the undrained strain paths deduced from Baligh's strain path method to reconstituted normally consolidated unaged laboratory specimens, and observed the resulting stress paths. Hajj (1990) carried out tests on normally consolidated and overconsolidated reconstituted kaolin. Hopper (1992) carried out similar work on reconstituted overconsolidated unaged clay (OCR = 3.7), and in addition tested high-quality (Sherbrooke and Laval) samples of intact lightly overconsolidated estuarine clay. These tests have shown that for normally and lightly overconsolidated soils the stress paths during tube sampling are of the form shown in Fig. 6.7. Only very small strains are necessary to cause severe disturbance to unaged reconstituted normally consolidated clays. Lightly overconsolidated (OCR = 1.5) structured natural clays appear to be able to withstand axial strain excursions of up to ±0.5% without significant loss of structure. Both normally and lightly overconsolidated soils suffer very large decreases in mean effective stress during tube sampling. More heavily overconsolidated clays appear to suffer little in the way either of destructuring or of effective stress change.

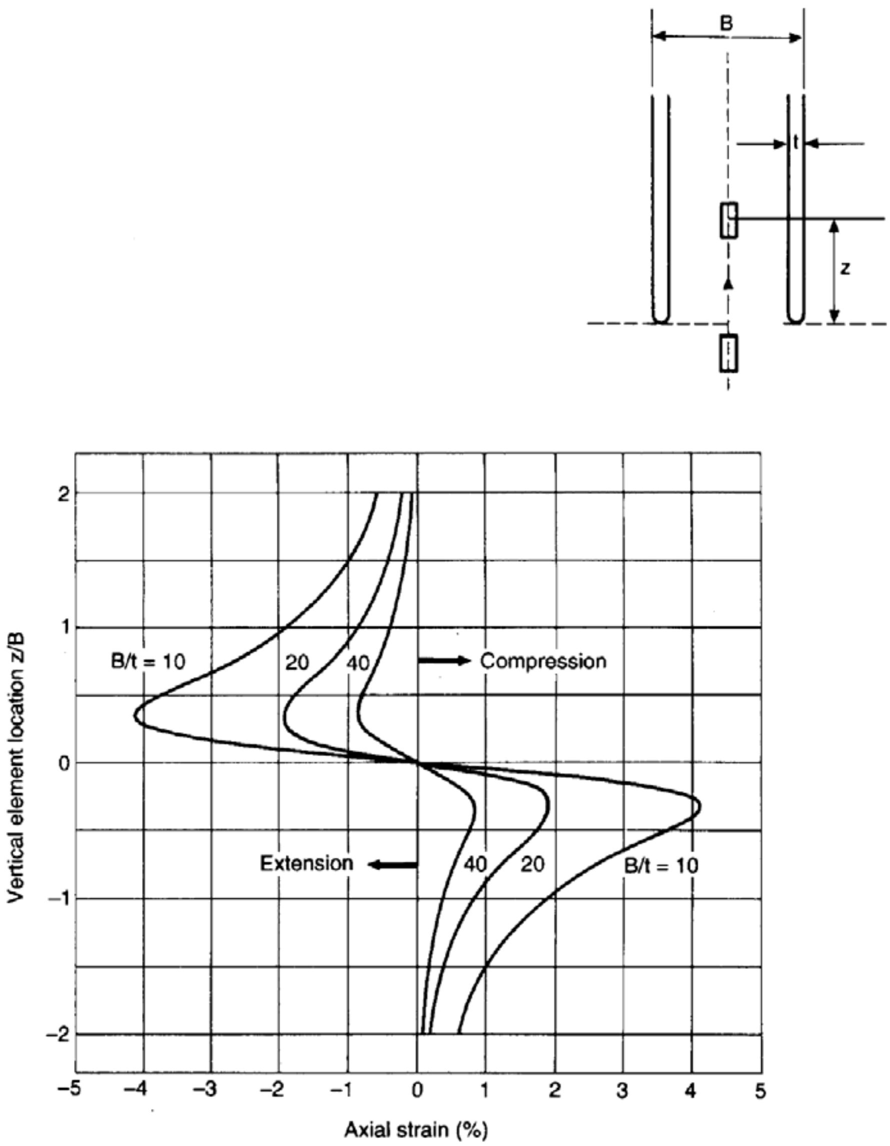


Fig. 6.6 Axial strain history at the centre-line of a simple sampler (after Baligh 1985).

On the other hand, tube sampling of heavily overconsolidated clays will often induce distortions of the type shown in Fig. 6.8. At the periphery of the sample the strains are similar to those imposed during simple shear testing; this cannot be modelled in the triaxial apparatus, where limited rupture zones occur at failure. This type of large-scale shear distortion results in a decrease in pore pressures, and an increase in the effective stress, in the periphery of the sample, which undergoes most shear distortion (Apted 1977; Hight 1986). If water is not available during the sampling process, either because the borehole is dry or because sampling takes places rapidly (and immediately after drilling to the required sampling depth), then pore pressure equilibration leads to a gradual increase in the mean effective stress in the centre of the sample, and a consequent increase in the undrained shear strength measured in laboratory triaxial tests.

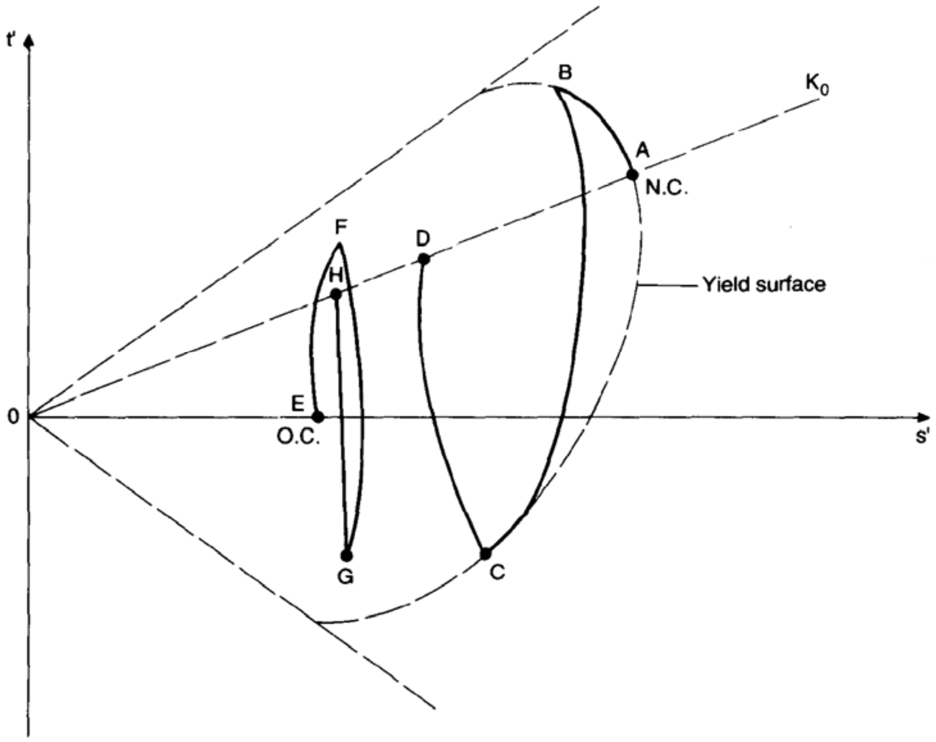


Fig. 6.7 Stress paths induced by tube sampling on normally consolidated, and lightly overconsolidated clays

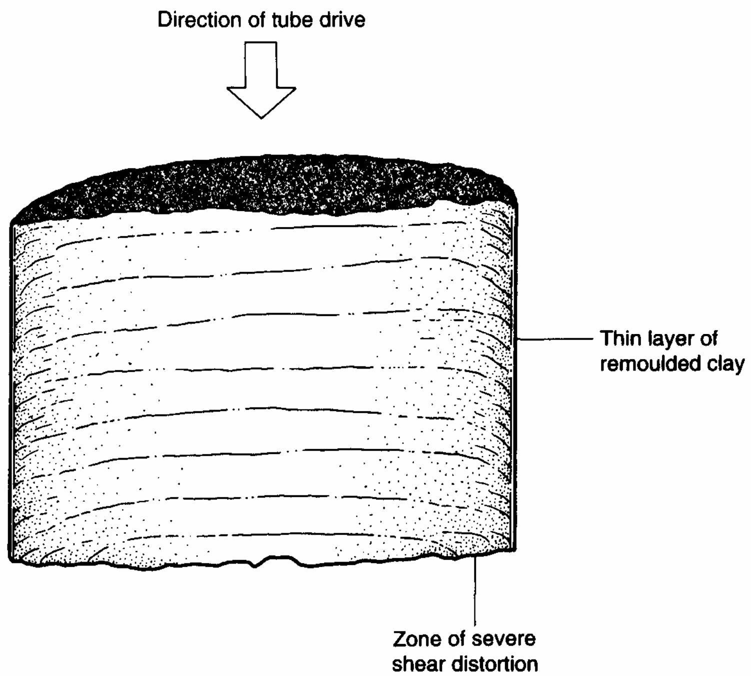


Fig. 6.8 Sketch of shear distortions induced in laminated heavily overconsolidated London clay by tube sampling.

Figure 6.9a shows estimates of the effects of tube sampling on the mean effective stress of London clay, shown by comparing the effective stresses in U100 tube samples (Fig. 6.11) with those in block samples (Fig. 6.5) from a similar depth in the London clay (about 22m below ground level) (Chandler *et al.* 1992). The mean effective stress is apparently almost twice as large in the tube samples as in the block samples. This implies that the stiffnesses and strengths of tube samples will also be much larger, and this is shown by Fig. 6.9b. Here Hight (1986) shows an estimated profile of the *in situ* undrained shear strength, back-calculated from undrained triaxial tests and the mean effective stress in the ground, and indicates the magnitude of correction that should be applied to the uncorrected undrained strength obtained from tube samples in order to allow for the increase in shear strength due to tube-sampling distortions. As might be expected from Chandler *et al.*'s findings, uncorrected strengths are about twice the estimated *in situ* values.

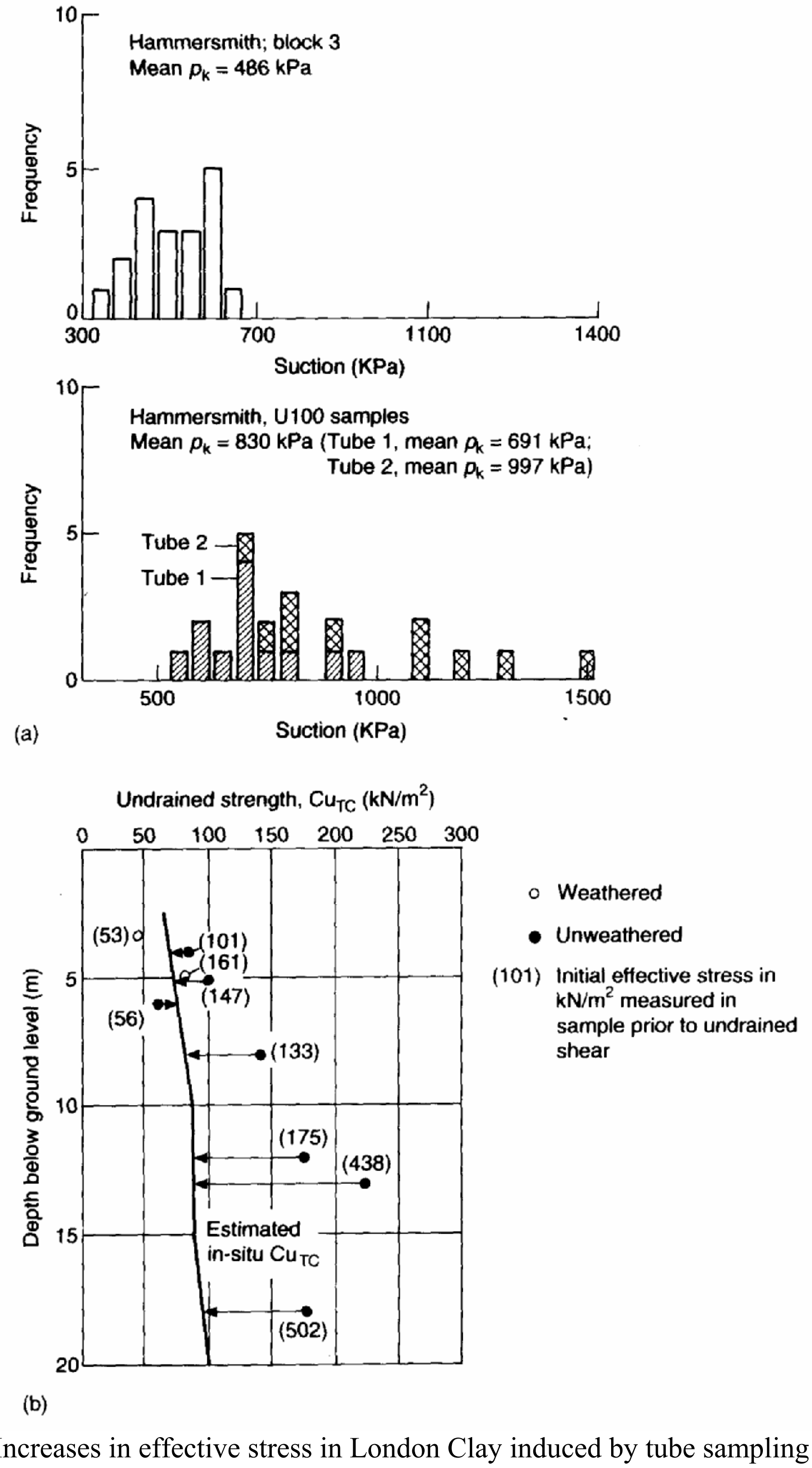


Fig. 6.9 Increases in effective stress in London Clay induced by tube sampling (Chandler, Harwood and Skinner 1992), and their effect on undrained shear strength (Hight 1986).

Whilst the data in Fig. 6.9 indicate that strength and stiffness values obtained for stiff clays from tube samples should be reduced significantly, this cannot always be relied upon. Some clays contain significant laminations of silt or fine sand, or silt- covered fissure planes. Here sampling in the presence of water will probably be accompanied by swelling, since rapid penetration is possible. In other cases known to the authors, drilling fluid has not been completely removed from the top of tube samples, and mean effective stresses have been very significantly reduced. Also, swelling resulting from drilling disturbance may affect soil samples, especially when the base of the hole is not cleaned out immediately before sampling.

As is evident from Fig. 6.8, significant shear stresses may be set up between sampling tube and the soil during driving. If the sampler is not properly designed these shear stresses can become sufficiently large that they prevent the entry of further soil into the sampler tube. This is termed ‘sample jamming’. Other pressures that may be applied to the soil during sampling include:

1. pressure on top of the sample, due to trapped borehole fluid, as the soil enters the tube; and
2. tension at the base of the sample, as the tube is withdrawn from the base of the borehole.

Over the past half century practical experience has led to the development of an empirical design basis for samplers. This has been based upon the following parameters:

- area ratio;
- cutting edge taper angle;
- lid ratio; and
- inside clearance.

In addition, it has long been known (for example, Hvorslev (1949)) that sample driving methods have a significant effect on the quality of the sample that is recovered.

AREA RATIO

Hvorslev (1949) defined one of the critical parameters affecting the disturbance of soil during sampling as the area ratio, defined by (see Fig. 6.10):

$$area\ ratio = \frac{D_e^{\,2} - D_i^{\,2}}{D_e^{\,2}} \qquad (6.9)$$

where D_e = external diameter of the sampler cutting edge and D_i = internal diameter of the sampler cutting edge.

BS Code of Practice 2001:1957 specified that the maximum area ratio for the British Standard open-drive sampler should be 25%. The revised Code of Practice on Site Investigations (BS 5930) specifies a typical open-drive sampler as having an area ratio of ‘about 30%’. In view of the fact that Hvorslev (1949) noted that the incremental ratio of sample length recovered to length of drive was 1.25 (i.e. a greater length of sample was being obtained than the distance the tube was driven) for area ratios of 40—45%, this change seems retrogressive, since cutting edge taper angle is not specified.

CUTTING EDGE TAPER ANGLE

Increasing area (or kerf) ratio gives increased soil disturbance and remoulding, increased penetration resistance and the possibility of the entrance of excess soil from the area immediately beneath the cutting edge during the initial part of the sampler penetration. the permissible area ratio will depend on the soil type, its strength and sensitivity, and the purpose of sampling.

The use of very small area ratios leads to very fragile sampler tubes which may bend or buckle during driving the sampler into the soil. The practical need for a large ratio can be compensated for by the use of a small cutting edge taper angle, as proposed by the International Society for Soil Mechanics and Foundation Engineering's Subcommittee on Problems and Practices of Soil Sampling (1965). For samplers of about 75 mm dia. they suggested the combinations of area ratio and cutting edge taper given in Table 6.10.

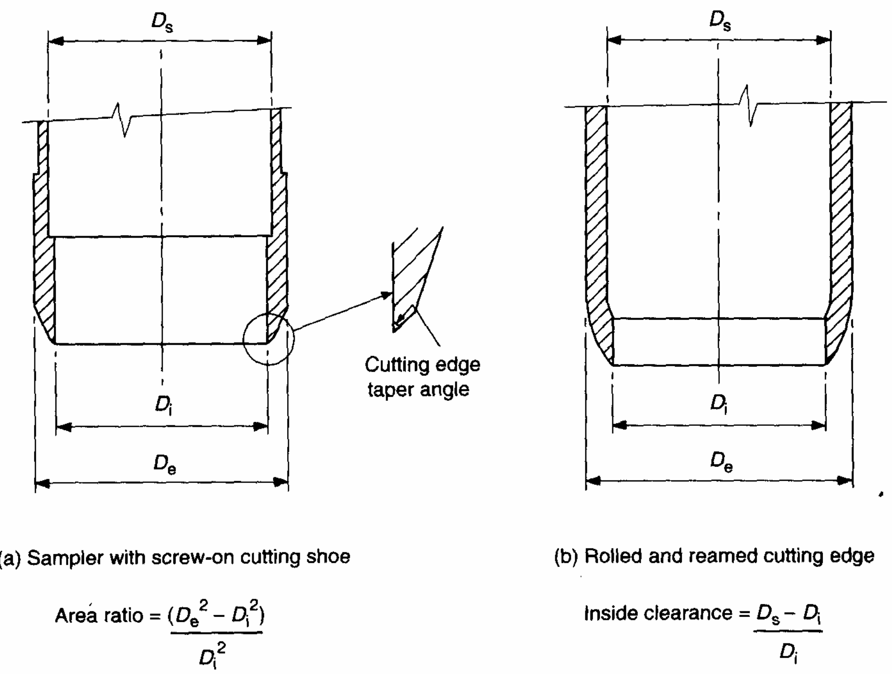


Fig. 6.10 Definition of area ratio and inside clearance.

It was also suggested that for clays the extreme edge of the cutting shoe could be given a 60° taper, until a 0.3 mm thickness was reached. In granular soils this thickness was suggested as the 10% grain size of the soil. When cutting edge taper angles are small, Scandinavian experience has shown that area ratios are largely irrelevant (Kallstenius 1958; Swedish Committee on Piston Sampling 1961).

Table 6.10 Combinations of area ratios and cutting edge taper

Area ratio (%)	Cutting edge taper (deg.)
5	15
10	12
20	9
40	5
80	4

INSIDE CLEARANCE AND L/D RATIO

With even moderate lengths of sample the adhesion or friction of the soil on the inside of the sampler tube may be sufficient to prevent further soil entering the tube. When wall friction is low it may produce slight compaction or compression of the soil, together with a down-dragging of soil layers at the edge of the samples. Severe distortion produces parabolic shapes in soil layers which are difficult to distinguish from plastic flow into the base of the borehole because of stress relief in soft soils.

Severe effects of wall friction are transmitted to the soil lying beneath the bottom of the sampler.

Ultimately, when the friction is large enough to prevent further entry of the soil into the sampler tube, bearing capacity failure of the soil beneath the bottom of the tube will take place. The soil will be severely remoulded and any material which enters the sampler will be useless even for visual examination. If samples are being taken continuously the top of the next sample will be worthless.

One of the major factors controlling sample jamming is the length to diameter ratio of the sampler. The adhesion between a cohesive soil and the inside of a sampler barrel will be:

$$A = \pi D L \alpha c_u \tag{6.10}$$

where D = inside diameter of the sampler, L = length of the tube, and α= reduction factor applied to the shear strength, c_u, to give the adhesion between the soil and the tube.

The bearing capacity of the soil beneath the tube is:

$$q_f = N_c c_u + p_0 \tag{6.11}$$

where N_c is the bearing capacity factor (5—9), and p_0 is the over-burden pressure and may approach zero if the borehole is large relative to the sampler.

Equating the adhesive force to the bearing resistance of the soil, and taking p0 equal to zero, leads to:

$$\frac{L}{D} = \frac{1}{4}\frac{N_c}{\alpha} \tag{6.12}$$

In an extreme condition, taking α= 0.5 and N_c = 5, it appears that a maximum permissible length to diameter ratio of 2.5 should be considered.

Three methods exist to reduce or eliminate wall friction between soil and the sampler; inside smoothness, inside clearance and sliding liners. The inside of all sampler tubes should be kept clean and smooth, and preferably polished. Oil may have to be used on old steel tubes, but this is not desirable.

Inside clearance (see Fig. 6.10) is defined as the ratio:

$$\textit{inside clearance} = \frac{D_s - D_i}{D_i} \tag{6.13}$$

where D_s = inside diameter of the sampler tube, and D_i = inside diameter of the cutting shoe.

Inside clearance gives the soil sample room for some swelling and lateral strain Jwing to horizontal stress reduction. Although neither of these types of behaviour is desirable, they are less undesirable than the consequences of adhesion between the soil and the inside of the sample tube. Inside clearance is usually less than 4%, because it should be large enough to allow partial swelling and lateral stress reduction but it should not allow excessive soil swelling or the loss of the sample when withdrawing the sample tube. Hvorslev (1949) suggests 0.75—1.5% inside clearance for long samplers, and up to 1.5% for very short samplers: he suggests an inside clearance of between 0.75 and 1.5% under average conditions. Where inside clearance of this magnitude is provided, Hvorslev recommends that for ‘properly designed and operated’ drive samplers of 50—75mm inside diameter the maximum length to diameter ratios can be increased as follows: loose to dense cohesionless soils L/D>|5—10, and very soft to stiff cohesive soils L/D>|10—20.

The ISSMEFE Report of the Subcommittee on Problems and Practices in Soil Sampling (1965) suggested that if the inside surfaces of the sampler tube are smooth and clean and the coefficient of friction is low, an inside clearance of 0.5—1.0% is suitable for sampling to depths of 20m in 'non-swelling' soils. With this inside clearance the permissible length to diameter ratio should depend on the soil type as shown in Table 6.11.

Table 6.11 Dependence of permissible length to diameter ratio on soil type

Type of soil	Greatest length to diameter ratio
Clay (sensitivity> 30)	20
Clay (sensitivity 5—30)	12
Clay (sensitivity < 5)	10
Loose frictional soil	12
Medium loose (?) frictional soil	6

The Subcommittee commented that large inside clearances (>1—3%) cause deformations of the samples, opening of fissures, and swelling of soils containing gases: 'A need for excessive inside clearances may indicate bad sampler design or sampling technique'.

Inside clearance has always been regarded as a necessary evil, and recently some samplers have been designed which deliberately do not make use of it. These samplers (for example, the Laval sampler — see Chapter 7) are intended for use in normally and lightly overconsolidated and sensitive clays, where disturbance at the sample periphery will produce a very low-strength clay, that is to some extent self-lubricating. They have low length to diameter ratios. The use of zero inside clearance to sample heavily overconsolidated clays cannot be recommended unless the length to diameter ratio of the sample can be less than 2.

The use of sliding liners inside sampler tubes would appear to be preferable to the use of inside clearance. Samplers described by Kjellman *et al.* (1950) and Begemann (1961) use foil and stockinette respectively, and can give near continuous samples of great length. The disadvantages of these types of sampler lie in their great cost.

SAMPLE DRIVING METHODS

Sample driving methods can have a severely damaging effect on soil. The effects of trying to drive a thick walled open-drive sampler into hard soil by repeated blows of a hammer are obvious; the soil is usually heavily fractured and if any material is recovered it often has the appearance of an angular gravel.

The method of driving a sampler is often crucial, not only to the disturbance of the soil, but in consequence to the ability of a sampler to recover it. Hvorslev (1949) rates drive methods as shown in Table 6.12.

Table 6.12 Driving methods (Hvorslev 1949)

Method	Motion	Sample quality
Hammering: repeated blows of a drop hammer	Intermittent fast motion	
Jacking: levers or short commercial jacks	Intermittent slow motion	Worst
Pushing: steady force — no interruptions	Continuous uniform motion	↓
Single blow: blow of a heavy drop hammer	Continuous fast motion	
Shooting: force supplied by explosives	Continuous very fast motion	Best

Hammering is a method commonly used to advance open-drive samplers into the ground, particularly in conjunction with light percussion drilling. The hammering action may take place down the hole, or at the top of the hole. In the former method (Fig. 6.11) the sampler tube is separated from a weight (the sinker bar) by a jarring link. The sampler tube is advanced into the soil by repeatedly lifting the sinker bar and allowing it to fall on the drive head. The use of a relatively thin and sometimes worn jarring link at the base of the borehole allows the sampler tube to rock from side to side; this can lead to breaks in the sample. Similarly severe effects can be produced if the sinker bar is lifted too high during driving, when the sampler tube will be pulled upwards and tension applied to what will be the middle of the sample.

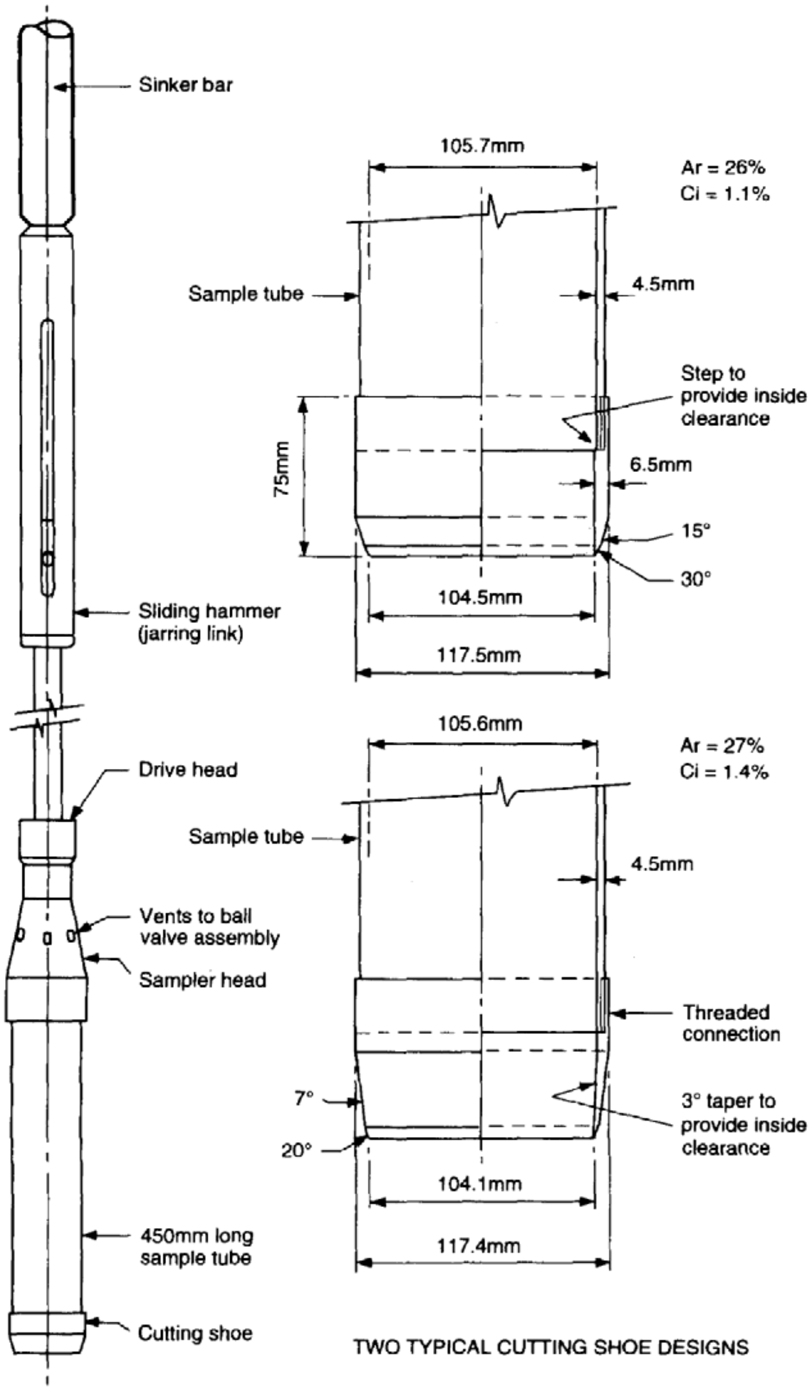


Fig. 6.11 U100 sampler assembly and details of cutting shoes.

If samples are to be hammered into the soil, then it is essential that the sampler should be rigidly connected to rods extending to ground level. If the borehole is deep and large compared with the rod size, spacers may be required to reduce rod buckling as the hammer energy travels to the base of the hole.

Hammering is cheap, but gives poor quality samples. At the other end of the scale, a single blow or the use of explosives will give a relatively high energy input which is difficult to control. One of the obvious dangers is that the sampler will be driven too far, leading to compaction of the material within it. The best practical method of sample driving is therefore pushing. Most modern auger rigs can readily supply a steady downwards force, with no interruptions, but a light percussion rig will need some adaptation. A typical arrangement for pushing a piston sampler into soft ground is shown in Fig. 6.12.

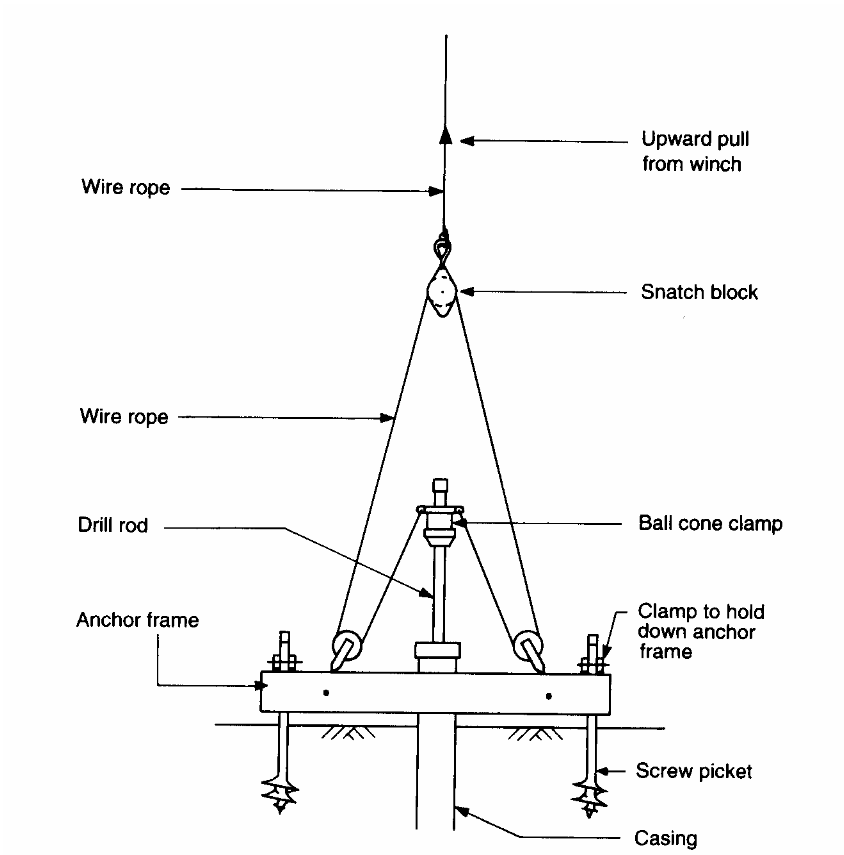


Fig. 6.12 Continuous push driving by means of winch and block and tackle.

When driving sample tubes into the ground, by whatever method, it is important to remember that water (or air) above the top of the sample or piston, contained inside the tube, must be able to escape without significant increases in pressure occurring. It is normal to provide vents in the top of the sampler, but their size must be limited for reasons of geometry and sampler strength. Therefore it is necessary to limit the speed of sampler penetration. For most samplers a speed of 25mm/s will be satisfactory.

Disturbance after sampling

Changes to the soil after sampling can be at least as severe as those occurring during boring and sampling. Five major types of change can be recognized:

1. moisture loss;
2. migration of moisture within samples;

3. the effects of inadvertent freezing;
4. the effects of vibration and shock; and
5. the effects of chemical reactions.

Moisture loss

Representative samples do not need to have their moisture content preserved, but it is often helpful to the engineer if considerable moisture loss is not allowed to occur. In order to restrict moisture loss and prevent loss of fine soil particles it is normal, therefore, to place the soil in heavy gauge polythene bags, boxes or glass jars.

Block samples and tube samples must not be allowed to lose moisture. Hvorslev (1949) reports the results of long-term experiments with different sealing methods which are shown plotted in Fig. 6.13. These results indicate that the best sealant for tube samples was battery sealing compound, with a water loss of only 0.1 g after sample storage in a tube with $^{3/4}$in. (19mm) sealing compound plugs. The stickiness of this asphaltic material, however, makes subsequent removal from soil and cleaning of tubes very difficult, and it is now rarely used.

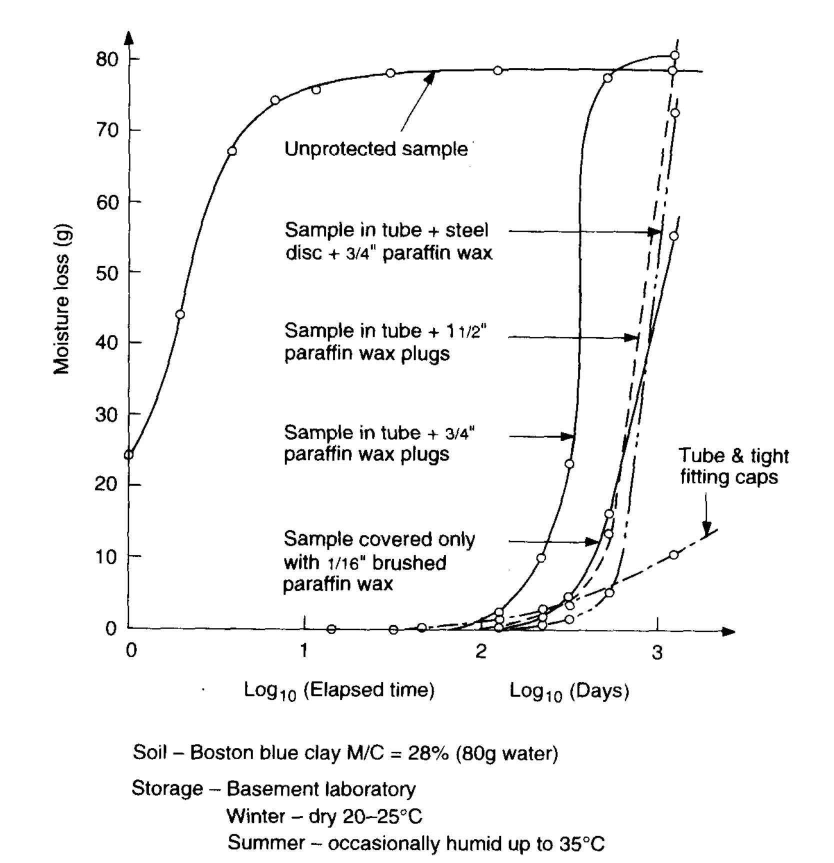


Fig. 6.13 Moisture loss with various sample sealing methods (data from Hvorslev 1949).

Block samples should be sealed by initially applying a brush coating of 2mm of paraffin wax, followed by wrapping in cheesecloth or clingfilm and subsequent dipping to increase the covering to at least 4mm. Large block samples which cannot be readily dipped can be placed in an oversize box and encased in paraffin wax poured around the soil to fill the void. Alternatively, after an initial brush coating of wax, the sample can be wrapped in aluminium or clingfilm, waxed again, and placed in a wooden box and encased in polyurethane foam.

Paraffin wax shrinks upon cooling, and small cracks will lead to rapid moisture loss. All paraffin wax should be applied as close to melting point as possible to reduce shrinkage. At this temperature it should be possible to dip a finger in the wax without being burnt. (Note that the melting point of paraffin wax is about 50°C, and therefore in very hot climates it will not be a suitable sealant.)

Tube samples are generally sealed with paraffin wax, but Hvorslev's experiments showed that even when properly applied this material will tend to undergo plastic deformations after about six months. Once defects appear in the seal, moisture loss is rapid. The best sealing method for tubes appears to be the use of tightly fitting plastic caps, but when these are applied to the sample the necessary escape of air may leave an unsealed channel which will allow moisture loss. These channels can often be closed by rotating the cap once in place, or by using plastic self-adhesive tape.

Alternatively, equally effective sealing methods are vented push-on or screw-on caps, combined with paraffin wax seals. With 0.4 mm paraffin wax and vented caps Hvorslev (1949) observed a 6.8g moisture loss in 1250 days. In this type of sealing it is important to fill the gap between the sample and the caps if the sample is short, since this will reduce the loss of moisture into the airspace and will also stop the sample sliding in the tube during handling.

Where caps are not available, the wax seals should be reinforced with metal discs, made of a material such as aluminium foil. This will reduce shrinkage and eliminate the formation of pinholes in the central section of the end of the sample.

If wax is used to seal samples of high permeability, or where voids or discontinuities are open in the soil, the penetration of wax poured directly on to the end of such a material may reduce its worth. These sorts of problem can be overcome by brush application of the first layer of wax, followed either by application and wax soaking of a layer of cheesecloth or by application of a layer of aluminium foil and further paraffin wax layers.

Even after the samples reach the laboratory. care must be taken. Storage conditions are important. As seen above, wax seals will often become ineffective after only a few months. Where sealing has been carried out to a high standard, the speed at which seals deteriorate is increased by a high storage temperature. Samples should therefore be stored in a cool room, with temperatures preferably not exceeding 30°C. Low temperatures and high humidity will help to reduce moisture losses once imperfections appear in the wax.

Migration of moisture within samples

Once samples are adequately sealed, migration of water within the sample may still lead to significant changes of properties such as undrained strength and compressibility. Two types of effect have been noted; in the first, water migrates from one type of soil to another (Kimball 1936; Rowe 1972); while in the second, differential residual pore pressures in the samples equalize with time (Casagrande 1936; Schjetne 1971). Consider a laminated soil, containing alternate layers of silt grading into fine sand and clay. *In situ* the clay might have a firm or stiff consistency, but once stress relief occurs the water in the granular layers will migrate to the clay and relieve the negative excess pore water pressures. Upon examination, the soil might appear to consist of very soft clay layers interbedded with relatively dry silty sands.

As an example of the second type of moisture migration, consider the effects of sampling on a very stiff clay of high plasticity, such as the London clay. After sampling the bulk, the soil (its inner portion) will be expected, as a result of stress relief, to have strongly negative pore pressures, whilst retaining effective stresses similar to those that it had in the ground. As a result of the type of shear distortions induced by tube sampling (Fig. 6.8) the outer part of the sample will have lower pore pressures, but if rotary coring has been used, or the sample tube was left in a water- filled borehole, then there may be have also been some swelling, and a consequent increase in the pore pressures around the outside of the sample. With time, as pore pressures equalize in the sample, there will be a change in the average effective stress in the sample, and therefore a change in the strength and compressibility that will be measured in the laboratory. Hight's results (Fig. 6.9) and those of Apted

(1977) suggest that generally tube sampling produces an increase in measured unconsolidated undrained strength (c_u). Rotary coring leads to a decrease in measured undrained strength. The change in strength and stiffness that is measured will, to some extent, be time dependent, because water must flow between the outside and the centre of the sample in order that pore pressures may equalize. Shear strengths measured immediately after sampling will be different from those measured, for example, after the samples have been transported to a laboratory and stored for some time.

In soft clays, Casagrande (1936) noted that the outer layer of a soil sample would have higher pore pressures than the centre immediately after sampling, as a result of the higher tube sampling strains that are experienced. This has been confirmed by a number of researchers (Schjetne 1971; Bjerrum 1973; Siddique 1990; Hopper 1992). Schjetne (1971) has measured the effects of sampling soft clays of different sensitivities with a Norwegian Geotechnical Institute piston sampler on pore pressures within the sample during sampling and after extrusion. His observations confirm Casagrande’s mechanism. Bjerrum (1973) has shown that owing to remoulding and moisture migration, the outer 5mm of extruded Drammen clay specimens typically have a moisture content about 3—4% lower than at the centre. To avoid this Casagrande recommended that the outer disturbed layer of the soil samples should be shaved off as soon as the samples are removed from the borehole.

Freezing

Probably the most serious effects of poor storage will occur if clay or silt samples are allowed to freeze (Kallstenius 1958). Ice lenses form initially in fissures, and the soil is gradually broken up by a wedging action as water is attracted from the rest of the sample to these lenses. Frozen samples are highly disturbed samples, and therefore a sample store should never be allowed to drop to temperatures below 4°C.

Vibration, shock, and mechanical disturbance

Vibrations caused during the transportation of some soils to the laboratory may cause a loss of strength and remoulding (Kallstenius 1963) particularly on very soft silty or sandy clays stored in horizontally positioned tubes. Compaction effects may cause soil distortion, the liberation of pore water and the movement or break-up of the wax seals at the ends of the specimen. Preventing these effects is often rather difficult, but they can be reduced by supporting the samples vertically on a compressible base such as a foam mattress.

Because they have no supporting tube, block samples of very soft, soft and sensitive clays are often at considerable risk from sudden shocks during handling and transporting to the laboratory. It is therefore advisable to place each sample in a separate rigid container, surrounding it with packing material to prevent it from moving.

Loose granular samples are likely to undergo density changes even when very carefully handled.

In the last stage of the life of a sample it will be extruded. The rules for good extrusion should be based on the same factors as control sampling. The soil should be pushed out at a steady speed. To avoid disturbance and distortion of the soil layers a plunger of almost the same diameter as the inside sampler diameter should be applied to the bottom of the sample, so that the same relative movement between soil and tube is continued. This means that the top of the sample must be marked in the field, and the first soil to emerge will be disturbed and should be discarded.

Various methods are available to provide force to an extruder plunger; these include direct fluid pressure, hydraulically operated pistons and mechanical devices. Air or water will often penetrate past a plunger, and can cause considerable disturbance to the sample. The most reliable systems use either a continuously screw-threaded shaft or an hydraulic piston to advance the plunger. Of those two methods, the hydraulic system is the most convenient, provided that it can provide a stroke of sufficient length.

Reactions between soil and tube during storage

Since it may be necessary to store samples for some time before laboratory testing can be carried out, there may be a considerable opportunity for chemical reaction between the soil and the sampler tube. Acid and alkali soils will attack sampler tubes, as will soil specimens with saline pore water. Further problems may occur if the tube and end cap are made of different types of metal. Changes in the pore water chemistry can have serious effects on soil behaviour, for example decreasing sensitivity. Electrolytic action may cause a change in soil plasticity, compressibility and shear strength.

Disturbance in the soil-testing laboratory

Even when the utmost care is taken to avoid the serious effects that have been described above, it is still possible for soil testing to be carried out on disturbed materials, as a result of further disturbance induced once the sample enters the laboratory. The principal causes of disturbance are:

1. poor extrusion practice, either due to high extrusion pressures being applied to unsaturated soil, or due to lack of proper support of low-strength clays during extrusion;
2. use of poorly designed tubes to take small-diameter specimens from larger diameter samples; and
3. damage to soil ‘structure’ as a result of poor saturation or reconsolidation procedures.

Effects of sample disturbance

The most obvious effect of sample disturbance can be seen when attempting to tube sample very soft, sensitive clays with a poorly designed sampler. The soil around the edge of the sample undergoes a very large decrease in strength, such that when the tube is withdrawn from the soil there is no recovery. But, as has been noted above, sample disturbance occurs in all sampling processes and, if sampling is carried out well, the effects of disturbance will hopefully be more subtle. Whatever its magnitude, sampling disturbance normally affects both undrained strength and compressibility. In addition, chemical effects may cause changes in the plasticity and sensitivity of the soil sample.

Failure to recover

Failure to recover is the most serious result of sample disturbance and can be caused by a number of factors such as:

1. Remoulding adjacent to the sampler walls. Adhesion or friction is required to support the soil when the sampler is being lifted out of the ground. Many soils exhibit sensitivity (i.e. a loss of shear strength during remoulding), and the remoulding of soil adjacent to the sampler barrel therefore reduces the chances of recovery. In soft or very soft soils a low area ratio or cutting edge taper angle is essential.
2. Pressure over the top of the soil sample can be created if no vent is built in to allow air or water to escape as the soil enters the lower end of the sample tube, or if the vent is too small and the velocity of soil entry is large. When pulling the soil samples from the soil, pressure over the sample will help to push it out of the tube. Most samplers are provided with one or more vents in the head. It is essential that they be kept clean.
3. Suction beneath the sample will occur as the sample tube is pulled from the soil, since a void must be created at the level of the base of the sample. This effect can be reduced or eliminated by either fixing lugs to the sides of the tube which will give an outside clearance if the sample

tube is rotated (Harper 1931) or by providing pipes down or in the sides of the sampler tube to allow injection of air or water at the base of the sample (Mohr 1943). Alternatively, suction may be opposed by using a piston sampler; if the soil tries to slide out of the base of the tube then a suction force will also be set up at the top of the sample. The use of suction at the top of a sample is apparently incorporated into many open-drive sampler tubes by the use of a 'ball valve' in the head (see Chapter 7). Even when perfectly clean however, the ball will not normally seat perfectly to provide an efficient seal and prevent re-entry of air into the top of the tube if the soil should start to fall out.

4. The tensile strength of the soil at the base of the sampler must be overcome. If the sampler is simply pulled vertically then the combination of disturbance, vacuum and tensile strength will often be sufficient to cause loss of recovery. To overcome the tensile strength the sampler may be rotated two or three times before being gently pulled upwards. Rotation of the sampler will induce torsional soil failure at the base of the cutting shoe. More sophisticated and less practical methods have been used involving snare wires (Buchanon 1936, 1938; Hvorslev 1940) or pushed curved springs which cut the sample free. These are unnecessary for routine work. When soil samples are lost a number of simple techniques can be tried to improve recovery. These include the following:
   i. A rest period after driving the sampler and before extracting it will allow the soil to swell inside the sample tube, improving the adhesion of fatty overconsolidated clays to the side of the tube.
   ii. Slight over-driving, which also increases soil disturbance, will help the retention of both cohesive and non-cohesive soils since it will splay them against the side of the tube and improve friction or adhesion.
   iii. Core retainers (core catchers, catcher boxes) can be incorporated in the cutting shoes of open-drive samplers to improve recovery. The most common designs are the 'basket', a series of curved springs mounted in or immediately above the cutting shoe, and the use of hinged flaps mounted in the upper part of the cutting shoe. Core retainers often cause severe disturbance around the edge of the sample, and the sampler area ratio will need to be large to accommodate them.

Strength

Although it has been noted above that tube sampling disturbance has the greatest effect, in terms of reductions in mean effective stress, on reconstituted clays its effect on the undrained shear strength of such material is, perhaps surprisingly, small. Laboratory experiments by a number of workers have shown that the stress paths during undrained shearing converge on the critical state and, because the soil is initially reconstituted, the state boundary surface is not disrupted by tube sampling. Typically, it has been found that the undrained strength is reduced by less than 10%, even when the material is not reconsolidated back to its initial stress state (for example, Siddique (1990)).

Tube sampling does, however, have a significant effect on real soils, most of which are either bonded ('structured'), and/or more heavily overconsolidated. Shearing of bonded soils during tube sampling can have the effect of progressively destructuring them. Clayton *et al.* (1992) show comparisons of the stress paths taken by soil specimens tube sampled in different ways. Figure 6.14 shows how tube sampling a lightly overconsolidated natural, structured clay with a standard piston sampler leads subsequently to much higher pore pressure generation during undrained shear, with the consequence that undrained strength is reduced. Clayton *et al.* (1992) found that provided tube sampling strain excursions were limited to ± 2% and that appropriate stress paths were used to reconsolidate the material back to its *in situ* stress state, the undrained strength of the Bothkennar clay would be within ± 10% of its undisturbed value. It is to be expected, however, that much greater effects will occur when sensitive clays are sampled.

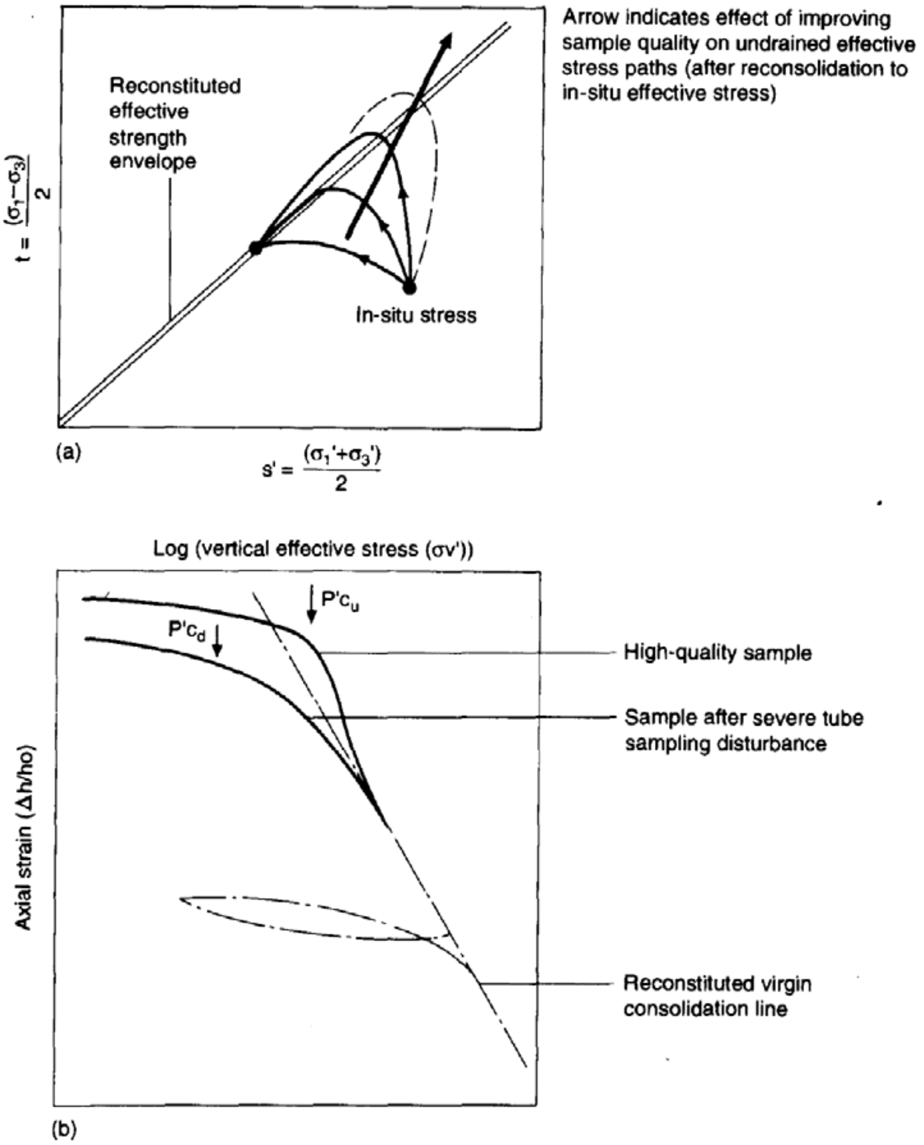


Fig. 6.14 Effects of tube sampling disturbance of lightly overconsolidated natural ('structured') clay on: (a) stress path and strength during undrained triaxial compression (b) one-dimensional compressibility during oedometer testing.

Heavily overconsolidated clays often display almost vertical stress paths under undrained shear. An increase in the mean effective stress level as a result of tube sampling will result in approximately proportional increase in intact strength. Unfortunately, however, this is not the only effect at work. Hammering of tubes into stiff clays can cause fracturing, and loosening along fissures, and this may lead to a marked reduction in measured undrained strength. In a simple study of the influence of different methods of sampling, Seko and Tobe (1977) measured the unconfined compressive strength as a function of depth obtained from samples taken using different sampling devices. The very wide variation in the strength of stiff Tokyo clay can be seen in Fig. 6.15, which shows that thin-walled open-drive hammered tube sampling gave much lower strengths than double-tube rotary coring methods with mud flush — the opposite of what might be expected from simple considerations of effective stress change alone. Single-tube rotary coring with a tungsten bit produced the lowest strengths.

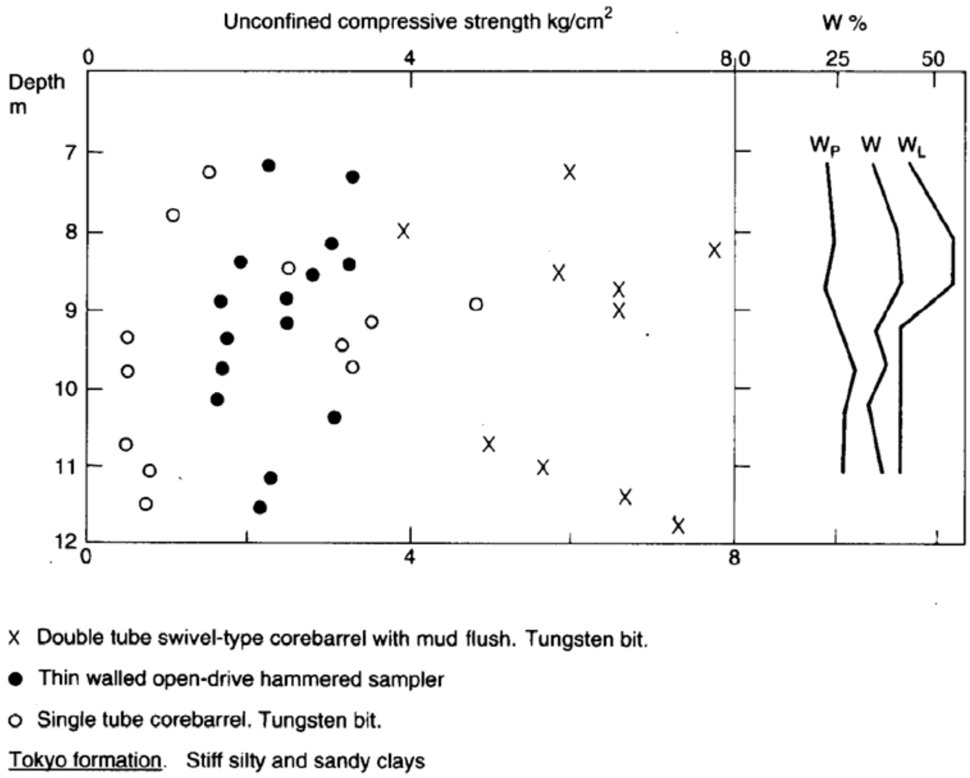


Fig. 6.15 Variation in unconfined compressive strength of Tokyo clay caused by different sampling methods (Seko and Tobe 1977).

Compressibility and stiffness

The effects of sampling on compressibility (as measured in the oedometer, for example) are difficult to assess because of bedding effects, particularly in heavily overconsolidated clays. The use of local axial strain measurement on triaxial specimens during the past decade (see Chapter 8) has produced new and more reliable stiffness data than can normally be expected from routine one-dimensional consolidation tests, It is now known that the measured small-strain stiffnesses of clays, most relevant to many geotechnical engineering problems, is for a given clay approximately linearly proportional to the mean effective stress at the time of measurement. ,This means that changes in effective stress as a result of disturbance are directly translated into proportional changes in measured soil stiffness.

Because of the growing appreciation of the influence of bedding and effective stress changes on measured stiffness, it has become common practice in the UK to adopt laboratory methods which will avoid these problems. In heavily overconsolidated clays, small-strain stiffness is often normalized with respect to the mean effective stress at the start of shear ($p'_o=(\sigma'_1+\sigma'_2+\sigma'_3)/3$). Alternatively, the stiffness of bonded soils is perhaps more appropriately normalized with respect to undrained shear strength, although it may be difficult to determine the true *in situ* value of this. *In situ* stiffness can then be recovered if $p'_{o(in\ situ)}$ or $c_{u(in\ situ)}$ be estimated. In lightly overconsolidated natural clay Clayton *et al.* (1992) have shown, however, that even the careful reestablishment of *in situ* effective stress levels before shearing cannot fully recover the undisturbed stiffness behaviour of the soil. A 60% reduction in $E_u/\ p'_o$ (measured locally, and after re-establishment of *in situ* stresses) was obtained for the Bothkennar clay following tube sampling strain excursions of ±2%, for example.

The results of a literature survey by Hopper (1992) are shown in Fig. 6.16. Here the very severe effects of tube sampling (including the effects of borehole disturbance, and obtained by comparing test results from tube samples with those on block samples in the same soil type) can be seen.

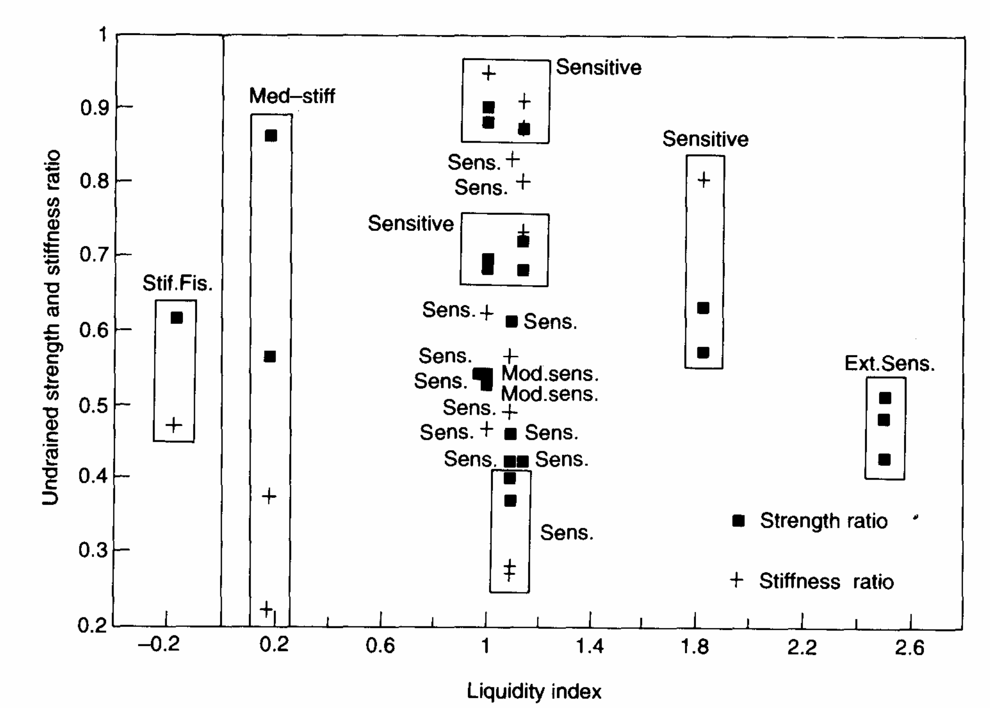


Fig. 6.16 Influence of tube sampling disturbance on undrained strength and stiffness (from a survey by Hopper 1992).

Siddique (1990) carried out an analytical study of typical sampler cutting shoe geometries, and found that:

1. increased area ratio, as a result of increasing the thickness of the sampler tube (and therefore decreasing the B/t ratio (Fig. 6.6)) causes a significant increase in the peak compressive strain occurring ahead of the sampler, but has only a limited effect on the peak extensive strain;
2. increasing inside clearance as a result of increasing the inside diameter of the sampler tube causes a significant effect on the peak extensive strain, and a slight decrease in the peak compressive strain; and
3. outside cutting edge taper angle has a marked effect on the peak axial compressive strains experienced by a sample.

In order to restrict the peak axial compressive strains (both in extension and in compression) to less than 1%, he recommends the following design for tube sampler cutting shoes:

- area ratio >|10%;
- inside clearance ratio>|0.5%;
- inside cutting edge taper angle 1 to 1.5°;
- outside cutting edge taper angle >| 5°.

In addition, Hvorslev's work indicates that tube samples should be pushed smoothly into the soil, in a single smooth action. Even given a maximum strain of 1%, normally consolidated reconstituted clays show considerable signs of disturbance. Compared with an 'undisturbed' specimen of reconstituted London clay, Siddique (1990) found the following reductions in effective stress, strength and stiffness:

p'_0	26%
E_{50}	65%
$(E_u)_{0.01\%}/p'_0$	78%
C_u	6%

Strain path tests on very high quality (Laval and Sherbrooke) undisturbed samples of natural clay by

Hopper (1992) has confirmed that for normally and lightly over- consolidated clays, stiffness is greatly affected by tube sampling, but that undrained strength reductions are less significant and can, in any case, be recovered by good reconsolidation procedures.

CLASSIFICATION OF SOIL SAMPLES

Hvorslev 's classification

Despite the more recent, and more sophisticated classifications which have been produced subsequently (see below), it is Hvorslev's (1949) classification of soil samples which remains widely used in British ground investigation. It is simple, and in view of the fact that we must now recognize that all soil will undergo some disturbance before reaching the laboratory test apparatus, there is arguably no need to further subdivide his categories. Hvorslev considers only three classes of sample.

1. *Non-representative samples* are samples containing mixes of soil or rock from different layers, or soils where certain fractions have been removed or exchanged by washing or sedimentation. This type of sample is now not normally considered as useful in site investigation, particularly since considerable skill may be required even to obtain a preliminary classification of the sub-soil. This type of sample is typically produced by the following.
    i. Washboring — where progress is made by jetting, and tests are made on open drive samples, fine granular soils may be washed away, and coarse granular particles may collect at the base of the hole, giving false particle size distributions in samples.
    ii. Bailing — the use of a 'shell', 'bailer' or 'sand pump' while percussion drilling forces the soil at the base of a borehole into suspension in the water. The coarse fraction of the soil will tend to sediment quickly, while silt- or clay- size material will remain in suspension in the water, and will often either be left in the borehole or tipped away before samples are taken.
    iii. Rotary open-holing — which uses a similar technique to washboring to advance the hole. Gravel-size particles will not be lifted up the hole, except by unacceptably high up-hole flush velocities which will lead to excessive borehole erosion.
2. *Representative samples* are samples of soil from a particular stratum which have not been contaminated by minerals or particles from other levels in the borehole, and have not been chemically altered, but may have been remoulded and have had their moisture contents changed. These samples may be obtained from samplers which are unsuitable for the soil conditions, or where samples are taken from the cutting shoe of samplers before they are sealed. In addition representative disturbed samples may be obtained from material obtained from relatively uniform soils by claycutter, or where clay materials are removed from the sampler shortly after sampling and placed in containers which allow them to alter their moisture content with time.
   Hvorslev's classification differs from Rowe's in that Rowe terms the ideal sample as representative (i.e. moisture content, material content, fabric and structure and stress state all remain unaltered). Hvorslev's 'representative samples' correspond to the British 'disturbed samples' which are sometimes specified as 'to be truly representative of the composition of the *in situ* soil'.

3. Undisturbed samples are samples in which the soil is subjected to little enough disturbance to allow laboratory experiments to determine the approximate physical characteristics of the soil, such as strength, compressibility and permeability. Hvorslev's 'undisturbed samples' correspond to Rowe's quality class 1 and 2 because although quality class 3 utilizes driven or pushed thin- or thick-walled samplers, these may not be suitable for the soil conditions and may lead to sampling disturbance.

Rowe's classification

The problem facing the engineer is to obtain adequate samples for the purposes envisaged. Rowe (1972) has defined five qualities of soil sample, based on the German work of Idel *et al.* (1969) (Table 6.13). This classification places heavy emphasis on the use of water balance. This means that where artesian conditions are encountered soil samples intended to be quality 1 or 2 must be taken from a rig mounted on a platform with casing extending above ground level, or by using drilling mud.

Table 6.13 Sample quality classes after Idel *et al.* (1969) as modified by Rowe (1972)

Quality class	Required soil properties	Purpose	Typical sampling procedure
1	Remoulded properties Fabric Water content Density and porosity Compressibility Effective strength parameters Total strength parameters Permeability* Coefficient of consolidation*	Laboratory data on *in situ* soils	Piston thin-walled sampler with water balance
2	Remoulded properties Fabric Water content Density and porosity Compressibility* Effective strength parameters* Total strength parameters*	Laboratory data on *in situ* insensitive soils	Pressed or driven thin- or thick-walled sampler with water balance
3	Remoulded properties Fabric A* 100% recovery Continuous B* 90% recovery Consecutive	Fabric examination and laboratory data on remoulded soils	Pressed or driven thin- or thick-walled samplers. Water balance in highly permeable soils
4	Remoulded properties	Laboratory data on remoulded soils. Sequence of strata	Bulk and jar samples
5	None	Approximate sequence of strata only	Washings

*Items changed from German classification.

BRITISH PRACTICE, AND THE BS 5930 CLASSIFICATION

British site investigation practice at present commonly divides samples into the following categories.

1. Disturbed samples:
   i. small disturbed samples ('jars'); and
   ii. large disturbed samples ('bulk bags').
2. Undisturbed samples:
   i. block samples;
   ii. open-drive samples;
   iii. piston-drive samples; and
   iv. rotary core samples (such as from the corebarrel).

All of these samples are intended to be representative of the composition of the *in-situ* soil; non-representative samples are, of course, never intentionally taken.

It is important to recognize the distinction between sampling in cohesive and noncohesive soils. In cohesive or cemented soils it is usually possible to obtain what Hvorslev termed ‘practical undisturbed’ samples, and there is a wide variety of sampling equipment and laboratory equipment to obtain and test such samples.

In non-cohesive soils Rowe (1972) has stated it is doubtful whether Quality 1 samples have even been obtained’. The problems of obtaining and testing non- cohesive samples can be briefly summarized as follows.

1. Volume changes during driving or subsequent handling of sampler tubes, due to vibrations.
2. The soil may collapse if unsupported. Special precautions must be taken to get the soil into the test apparatus without releasing compressive stresses.
3. High friction, developed as the sample enters the tube, may remould or alter the stress levels on the soil.
4. Inevitably some modification of the stress levels on the sample will take place. The strength and compressibility of non-cohesive materials are highly stress-dependent.

In special circumstances, samples have been obtained using freezing (Fahlquist 1941) and chemical injection (Van Bruggen 1936; Karol 1970). Both these techniques alter the soil, the first by volume change and the second by contamination. Rowe claims that Quality 2 samples may be obtained using mud or water-filled boreholes and thin-walled pushed piston samplers have been successfully used in medium dense sands, but in most cases it will be sufficient only to consider obtaining Quality 3 samples to allow fabric examination for the planning of *in situ* tests. Quality 3 samples may often be obtained using relatively common sampling techniques, such as thin-wall piston sampling. One device specifically designed for sand sampling has been described by Bishop (1948).