doi: 10.1680/moge.57074.0667

Chapter 49

Sampling and laboratory testing

Chris S. Russell Russell Geotechnical Innovations Limited, Chobham, UK

The development of a good ground model relies upon a successful ground investigation with appropriate sampling techniques and laboratory testing. The involvement of all stakeholders in the process, open communication and supervision are essential for high quality parameter determination for the foundation or temporary works design. Various sampling techniques and their validity for parameter determination are discussed in a process-based fashion along with an insight into the various laboratory tests available. The list of laboratory tests is not exhaustive but lists many of the common test types (and some specialist considerations) associated with modern design requirements from low-rise buildings to advanced construction. The reader is given the main principles of sampling and sample disturbance with reference to the effects on laboratory testing and parameter determination which may be used as a basis for investigations worldwide.

CONTENTS

49.1 Introduction

From previous chapters and sections the engineer will have created and planned the site investigation and have a list of requirements needed to satisfy the construction program from the initial desk study phase through to the finished construction (see Chapter 4 *The geotechnical triangle*). These plans will have been laid out in a 'best practice' manner but should allow for some deviation especially when dealing with the 'unknowns of nature' in what lies beneath the surface. Plans that are too rigid may lead to problems later when dealing with the running of the project and certainly may cause problems in the ground investigation phase. The budget for ground investigations are always only a few percent of the overall project cost but it is here that major savings can be made in the overall design phase if thought and care is taken.

The design of the ground investigation will lead on from the desk study which will have identified the expected stratum and ground conditions. The ground investigation will verify these conditions and identify any deviations which may require further attention or categorisation leading to a ground model suitable for the engineering or design purposes intended. Laboratory tests are routinely used to calibrate ground models, however, *in situ* conditions (sometimes dominated by discontinuities or complex horizons) may require more expensive and time-consuming field tests for full categorisation (see Chapter 47 *Field geotechnical testing*). This sounds simple on paper but is in fact crucial for the design and construction stage. Failings here could have disasterous consequences. If in doubt seek advice. It should be noted that this chapter pays attention to the physical properties of the ground and so does not address testing for chemical properties or ground contamination (refer to Chapter 48 *Geo-environmental testing*). For further information with regard to the task/project in hand, you may wish (amongst others) to refer Chapter 13 *The ground profile and its genesis*; Section 3 *Problematic soils and their issues*; Chapters 43 *Preliminary studies* to 46 *Ground exploration*.

49.2 Construction design requirements for sampling and testing

The categorisation of the site should be comprehensive and provide the best possible parameters for the foundation design whether it be for simple load-bearing calculations for a strip footing through to finite element analysis which are often required for more complex or fragile construction. The required parameters drive the testing schedules for laboratory testing in order to gain accurate knowledge of the physical (and chemical – refer to Chapter 48 *Geo-environmental testing*) properties of the site (the ground model) to indicate uniformity/non-uniformity of the ground (both laterally and with depth). It is this coverage which is required for any form of foundation design or physical modelling. Soil tests should be identified to provide the parameters required for design.

Basic categorisation can be carried out on site through trial pitting, but drilling and sampling will be required for depth profiles to be identified. Each site or contract should be treated as unique and specifications must be reviewed for each contract in light of this. The complexity of the construction and the nature of the ground play a most important role in the parameters required. Interestingly the process of desk study followed by ground investigation and subsequent laboratory testing has required much thought for the layout and sequence of this chapter. It is the output in parameters required for the design that will dictate the sample types which need to be taken during the initial ground investigation phase, or a two-phase ground investigation may be required if the construction is complex or fragile. In this respect the order of description in this chapter has had to be reversed as the parameters for design will dictate the test types and therefore the sample quality which will finally control the sampling methods and preparation of the samples for the tests. In other words, think about what you need before you try and achieve it. The cost savings are in the planning of this and it can be expensive (apart from commercially embarrassing) to get this wrong. The parameters (and some associated tests) you may require can be summarised as follows.

49.3 The parameters and associated test types

In their simplest form, the results of laboratory testing may be used to categorise our area of interest and identify uniformity or non-uniformity of ground characteristics both laterally and vertically whilst more advanced parameters are based around the stability or reaction of the soil with regard to the loading (or unloading) of the construction, both in the short and long term. These values are related to soil strength and stiffness (possibly anisotropy), compressibility and permeability. For the short-term requirements and very low permeability materials we may be more interested in undrained scenarios whilst for the long term or high permeability materials we would be more interested in the fully drained conditions. These properties all vary with different stress states and again the ground investigation should be targeted to gain knowledge of these so that laboratory testing can be representative of the *in situ* conditions and be used with confidence in the foundation design stage. Test standards are listed in the appendix to this chapter for most common laboratory tests and some history/background of these tests and their basic methodology can be found in the *Manual of Soil Laboratory Testing* (Head, 1986).

49.4 Index tests

These are the most common and routine of all site characterisation techniques and are used as a profiling tool both vertically and laterally to build the ground model suitable for the project in hand. They may be carried out using both disturbed and undisturbed material and are relatively cheap and quick to perform. Index tests may be used to support 'expected' basic ground behaviour and identify where more expensive tests (or increased parameter interest) may be required for design. Such routine tests generally comprise moisture content determinations, particle-size distribution and Atterberg limits. When combined these provide very useful profiling tools which can also be used to calibrate ground models and verify the results of any further laboratory tests to be carried out. They provide information which can be combined with drilling logs to identify and corroborate with the height of the water table, variation of soil type and the expected soil behaviour, so giving clarity to the ground model.

49.4.1 Moisture content

This is the simplest and cheapest of the soil tests to be carried out in laboratories and can be carried out on both undisturbed and disturbed samples. The test consists of a small sample of soil being weighed before and after drying to determine the ratio of solid particles to water. Interestingly though, it is the moisture content of soils or weak rocks which often dictates or certainly can dominate their engineering behaviour. The 'engineering' of moisture content only improves the workability or placement of some materials (say for compaction in a landfill liner or dam core) but can cause catastrophic failure through loss of shear strength/cohesion if calculated or carried out incorrectly. Moisture content profiles of depth and lateral distance can be used to indicate zones of differing soil properties. Clays with 'high' moisture contents are often soft (or softened) compared to their drier counterparts (with identical mineral composition). They are also more likely to compress and collapse. The effect of desiccation and the possibility of ground heave can be identified through moisture content profiling to nearby vegetation or drawdown situations. Higher localised moisture contents can even be used to identify the location of broken drains and water pipes which may be the cause of undermined foundations (amongst other engineering problems).

'Natural' moisture content determinations require the material to be tested to be sealed in its natural state. This may appear obvious, but many people get it wrong. The sample should be taken from the ground at whatever depth without the influence of outside water/drill fluid/evaporation and should be immediately sealed in a fully airtight fashion. If the bag is not sealed immediately it will change its moisture content and be unrepresentative (if it is raining, water may enter the sample, and if it is warm and sunny then water may evaporate from the sample). Cohesive materials recovered from rotary cores or where drill flushes have been in contact with the material should be sub-sampled away from the periphery of the material (in contact with the drill flush). The natural moisture content in these material types will be preserved in the centre of the core for a while due to their low permeabilities. Conversely it is almost impossible to gain a natural moisture content of many non-cohesive materials (especially gravels) as the high permeability of such materials prevents retention of the water during sample extraction from the ground. Thought should also be given to the materials scheduled for these tests, especially if they are likely to contain hydrated minerals such as gypsum. In such instances the oven drying temperature used in the laboratory determination should be below the level at which such minerals 'dehydrate' or release water from their crystal matrix (if present). If the oven temperature is above 110°C (hence 105–110°C for standard BS 1377 determinations) other volatile fluids will be evaporated other than water leading to erroneous test values. For materials containing (or suspected to contain) gypsum the oven temperature should not be more than 80°C. The drying stage is complete when successive weighings are within 0.1% at four-hour intervals (BS 1377:Part 2:1990). Saline pore waters can also lead to incorrect determinations and in such instances other tests may be more appropriate.

49.4.2 Atterberg limits

These are used to classify fine-grained soils and commonly identify two of the original seven limits defined by Albert Atterberg. The limits are based upon the moisture content of the soil and can be carried out on both undisturbed and disturbed samples. The plastic limit is the moisture content at which the soil changes from a semi-solid to a plastic state whilst the liquid limit is the moisture content at which the soil changes from a plastic to a viscous state.

Liquid limit determinations are carried out either by measuring the penetration of a calibrated cone into a known volume of fully mixed material at four increasing moisture contents (four-point cone method) or by 'bumping' material in a calibrated Casagrande system using a grooving tool again at four different moisture contents. There are alternatives to the four-point systems described here, but they are not ideal. The plastic limit (PL) is determined by the point at which soil can be 'rolled' in a calibrated way to form a thread 3 mm in diameter which has shears both transversely and longitudinally. This part of the test (PL) may yield variable results due to differing operators and levels of experience. This is a basic test but not a simple one to carry out!

The difference between the plastic (PL) and liquid limit (LL) is known as the plasticity index (PI). The relationship allows approximate determinations of compressibility, permeability and strength and is therefore very useful for soil classification. The derived plasticity index (PI) can also be used to determine the amount of clay present. High PI values indicate significant clay contents whilst low PI values indicate the dominance of silt particles. A PI of zero indicates the absence of both clay and silt and is termed 'non-plastic'. Generally the higher the PI value the greater the soil's potential to change volume. High PI values would signify a large volume change when wetted and large shrinkage when dried, etc. This general rule, however, does not take into account the presence of particles larger than 425 µm (removed by sieving before the test commences) and so the modified plasticity index (I'_p) is often more appropriate, but only for overconsolidated clays. The calculation for the modified plasticity index was founded by the Building Research Establishment (BRE) and is:

Modified Plasticity Index (I'_p) = PI × (% <425 µm/100%).

The National House Building Council (NHBC) has used this same calculation but has modified the percentages which identify high, medium and low volume change/shrinkage potentials. Values should be recorded as percentages as the general terminology of high, medium and low I'_p values are slightly different between the BRE and HSBC references. It can be seen very quickly that such tests can yield very good information and confidence in the material properties of fine-grained materials, especially when profiled.

49.4.3 Particle size distribution (PSD) analysis

This test can be carried out on both undisturbed and disturbed samples. A PSD determination is the mass of particles within designated size ranges expressed as a percentage of the complete sample mass. The range of sample sizes split the soil into its component groups ranging from clay to silt, followed by sand and gravel upwards in size (through to cobbles and boulders). For a complete analysis two distinct test types are performed. For particles larger than 63 µm (for British Standard Tests, 75 µm for ASTM standards) the material is graded by passing through sieves of decreasing sizes. The definitive method for these 'coarse' grains is by wet sieving whilst a quantative test may be carried out by dry sieving (for soils containing insignificant quantities of silt and clay). For wet sieving the particles less than the smallest sieve size are washed from the material and retained for the second part of the test. Particles less than the smallest test sieve are then graded by settling from suspension (with time) in either the hydrometer test or by pipette methods.

From the visual description of soils we make estimations of the percentages of the various sediment sizes which make up our sample. The PSD determination scientifically derives the exact percentages of each soil size fraction within the sample. It is therefore possible that a visual description may be slightly different to that recorded from a PSD analysis, but they can be used to calibrate each other and expose inaccuracies in logging and drilling records. The particle size distribution of a soil will also indicate the permeability and, possibly, compressibility characteristics to be expected from other tests which may allow test specifications to be designed for the material types in question.

Beware: it is not unknown for the sampling of some materials to be carried out badly, especially when retrieving granular material from depth. It is very easy to wash out the fines in the drill fluid or allow the sample tube to drain out water (carrying away fines in suspension) which will lead to an inaccurate PSD analysis. The author has even seen junior lab technicians pour the coarse material from the sample container/bag to be tested and leave the fine sediment in the bottom to be discarded. Both these instances would lead to a very inaccurate PSD analysis and an erroneous judgement of material properties.

Many other index tests exist and may be used in combination or with the main three tests listed above in order to complement soil characterisation. It is most important to remember that any test result is only as good as the representative sample taken and delivered to the laboratory and should be representative of the stratum from which it was taken. Coarse material should be taken in large quantities in order to be representative (see BS 1377:Part 2:1990 for required sample sizes for PSD and other analyses).

49.4.4 Compaction-related tests

These are a series of tests which identify the relationship of density (often with changing moisture content), with a known compactive force. These tests are mainly carried out on disturbed material or material to be classified for engineered fill (which by their nature are 'disturbed'). Compaction itself is a process where the density of the soil is increased by packing the soil particles closer together and so reducing the volume of air (without significantly changing the moisture content). The addition or reduction of moisture content for each test stage simply alters the strength characteristics of the soil and its 'compactibility'. These tests are common for field design of engineered fills as they will provide optimum moisture contents for the fill material with regard to compactive effort available (see Chapter 75 *Earthworks material specification, compaction and control*). Common forms of these tests for varying engineering uses and soil types are as follows:

- determination of dry density/moisture content relationship (2.5 kg rammer);
- determination of dry density/moisture content relationship (4.5 kg rammer);
- determination of dry density/moisture content relationship (vibrating hammer);
- determination of maximum and minimum dry density;
- determination of moisture condition value (MCV);
- determination of California bearing ratio (CBR);
- determination of chalk crushing value (CCV).

Please refer to the relevant current standards for full descriptions of the above methods with regard to the soil types to be tested. Of the tests listed above it is only the CBR which provides an empirical strength criterion (CBR value), and is often associated with pavement construction (see Chapter 76 *Issues for pavement design*).

49.5 Strength

This is defined as the limiting shear stress that a material can sustain as it suffers large shear strains (Atkinson, 2007). The response of the soil in both strength and stiffness are related to the 'state' of the soil. This state is related to the density of the material and stress level which is linked to pore pressure, and therefore, 'effective stress'. It is interesting that so many parameters and stress states are closely linked but all ultimately controlled by the principles of effective stress. 'All measureable effects of a change in stress, such as compression, distortion, and change of shearing resistance, are due exclusively to changes of effective stress' (Atkinson, 2007). The following is a list of laboratory tests including triaxial and direct shear types (amongst others). Triaxial tests are a family of tests whose subtleties are controlled by varying boundary, drainage and loading conditions, but 'appear' to use similar equipment.

49.5.1 Triaxial test types

For basic boundary, drainage and loading conditions see **Figure 49.1**.

- *Unconfined compressive strength* (UCS). This is a total stress test (no pore pressure measurement) and is carried out without radial confinement. It may also be termed 'unconfirmed' compressive strength, and unfortunately has the same acronym associated with the uniaxial compressive strength (UCS test) carried out on rocks. Although the applied stresses are all the same in these tests the standards, methods and test equipment used for soils and their rock equivalents are distinctly different.
- *Unconsolidated undrained triaxial test* (UU). Again this is a total stress test as pore water pressure is not measured. For this test a radial (confining) pressure (σ_3) is applied to the sample and is of a magnitude which relates to the depth of sample origin. Shearing rates are standardised for these tests and reference should be made to the relevant regional standards for further information. This test should not be confused with a UUP test (see below) which often goes under the same name.
- *Unconsolidated undrained triaxial test with pore pressure measurement* (UUP). An effective stress test which gives the undrained shear strength for the material. Be aware that the effective stress measured for such tests may not be representative of the mean effective stress of the material *in situ* due to the effects of sample disturbance. For more advanced testing the pore pressure may be measured both at the base and mid-height of the sample.
- *Isotropically consolidated undrained triaxial test* (CIU). As above but the sample is isotropically consolidated to a mean effective stress relevant to the *in situ* depth of the sample or a particular stress/depth condition which is to be modelled.
- *Isotropically consolidated drained triaxial test* (CID). This is for the 'drained' or long-term condition of the above test. For clay materials this may be a test of very long duration due to the very low permeability associated with such particle sizes and mineralogies, but for sands and free-draining materials it is usual to carry out drained triaxial tests rather than undrained shearing because the short- and long-term conditions should approximate (due to the high permeabilities). Undrained conditions are unusual for sands (unless in a fully confined state) within the ground unless one is trying to model a very specific ground or construction condition. Undrained shear tests in non-cohesive materials cause immediate dilation of the material which is often unrepresentative of field conditions.
- *Anisotropically consolidated triaxial tests* (CAUC, CAUE, CADC, CADE –see section 49.6).

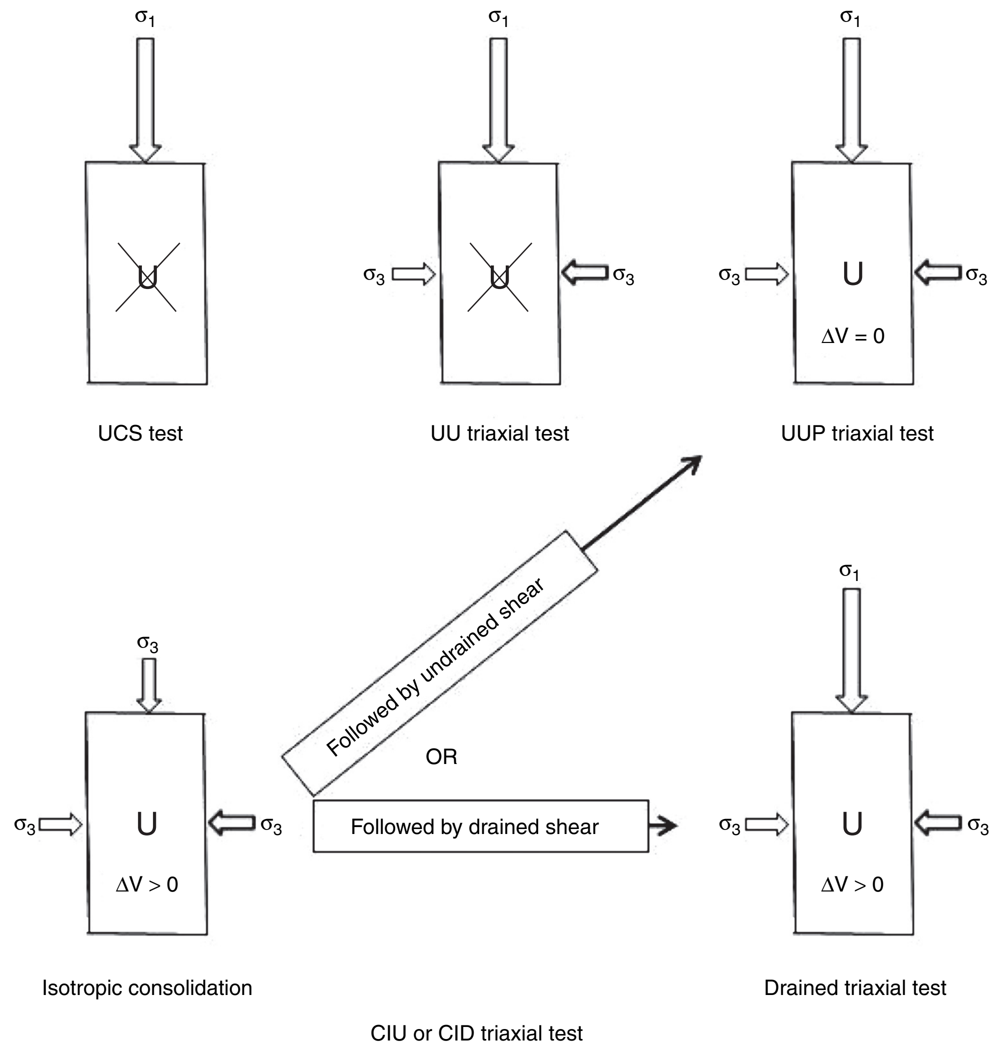

Figure 49.1 Triaxial test: boundary and loading conditions

Reference should be made to Chapter 17 *Strength and deformation behaviour of soils* for the interpretation of different strength parameters and 'ideal' triaxial tests. All of the triaxial tests listed above require undisturbed samples to be taken. Information and guidance is given later in this chapter as to the sampling types and methodologies which should be planned from the outset of the ground investigation. It can also be seen from the list of tests above that rates of testing are mentioned. For effective stress tests pore water equalisation is required in order to measure strength and stiffness correctly (see section 49.6).

Figure 49.2 shows a typical triaxial loading frame with a modern cell pressure and backpressure control system on the left and the transducer logging system on the right. Historically, pressures were applied manually and the measuring instruments were read manually by either reading dial gauges or writing down the specific outputs from digital readout units. Modern technology has advanced triaxial testing so that stress control and instantaneous measurements of transducers are becoming increasingly the norm. Load measurement can also be carried out in several different ways in laboratories and **Figure 49.3** shows the types of load measurement commonly available.

The original external load measuring devices commonly used were in the form of a load ring whose calibrated deflection could be read manually by an operator. These were superseded by load rings with integral digital readouts which can be manually read or their output logged by computers and read remotely. Electronic load measurement devices have become common now, but as with all previous versions the drawback of all of these is that they are generally not waterproof and therefore need to be mounted externally to the cell. This leads to the load transducer measuring the friction of the ram as it passes into the cell leading to errors in load measurement. Calibrations of ram friction can be made to minimise this error; however, slight non-concentric loading can lead to the ram ‘sticking’ and giving ‘false’ load readings. It is possible to utilise rotating bushes where the ram passes into the cell but these are not without problems themselves. The ultimate load measuring device is the submersible load cell which is mounted on the end of the ram and remains in direct contact with the sample within the cell whilst any loading occurs. In essence any load which is applied to the sample will also be seen by the submersible load cell without any external effects and so is the ultimate load measuring instrument. The cost of such devices is often prohibitive in many institutions and so external load measurement is most likely to be the norm.

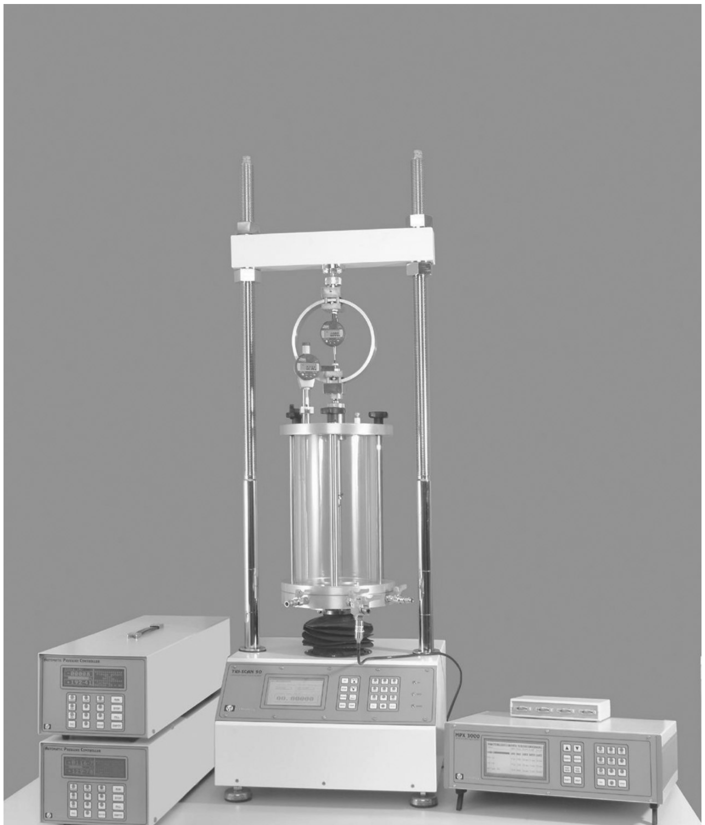

Figure 49.2 Triaxial test equipment
Courtesy of VJ Tech Ltd

It should be noted that testing of organic soils, especially peat, can produce rather ‘unexpected’ test results compared to those associated with sands and clays. This is due to the type, fabric, percentage and orientation of organic material which may be present in such samples. Undrained tests do not take into account the high compressibility of such materials and would yield low shear strengths whilst drained shearing stages will display very high strains and again possibly unrealistic shear strengths due to the complex nature of the material and the boundary conditions which exist in triaxial samples. These tests are certainly possible but care should be taken in the design and expectations of shear strength tests on such materials. More will be said about such materials as we move on.

Rock strength is dominated by its mineralogy and cementation along with the presence and orientation of discontinuities. Reference should be made to Chapter 18 *Rock behaviour* for further understanding and categorisation of the material. Commonly strength tests are carried out as uniaxial compressive strength tests (unconfined), but may also be carried out as confined or even effective stress tests using specialist high pressure/stress equipment. Reference is made to the main standards for such common test types in the appendix to this chapter. It should be noted that rock tests require very different equipment for preparation and testing than soil tests; however, there is a grey area where we might classify a soil as a weak rock and vice versa. In this instance, experience will prevail over which tests and equipment to adopt. As with all testing, these will require specialised personnel with significant skill

Figure 49.3 Triaxial load measuring equipment (Left: Load ring with manual dial gauge. Middle: Load ring with digital dial gauge. Right: Submersible electronic load cell)
Courtesy of VJ Tech Ltd

and experience. The 'art' of these tests is in the sample preparation and the specific equipment used (along with sample orientation with regard to discontinuities and preferred fabric). Check with your nominated testing laboratory that they have the correct equipment and expertise to carry out such tests.

The cutting and facing of the sample if carried out incorrectly can reduce the strength of the test specimen by up to two thirds by the introduction of point loads and non-parallel faces. Cutting equipment should have very thin diamond blades and work by the rock core being moved (whilst rigidly supported) across the cutting blade, not vice versa as in concrete cutting equipment. All cutting marks must then be removed by facing. This involves the polishing of the sample surface until both ends are completely flat and parallel (tolerances can be found within the standards listed in the appendices at the end of this chapter). The samples must then be mounted on specially hardened platens (which are calibrated for flatness) which have a diameter either the same or no greater than 2 mm larger than the specimen diameter, one of which will have a spherical seat and be placed on the top of the specimen. The loading surfaces of the compression machine itself will be rigid, parallel and unable to rotate. One can appreciate that rock testing is a highly specialist form of material testing and there are few laboratories which can carry out this form of testing correctly.

49.5.2 Direct shear tests

Alternative strength tests such as the shearbox test are also common where phi angles and cohesion intercepts are required for design purposes. Such tests can be carried out on both undisturbed and disturbed samples (depending upon the desired engineering use for the material). The historical test is the shearbox which can yield values for phi and cohesion (see **Figure 49.4** for test mechanism).

Originally designed for testing sand, the equipment is now used commonly on non-cohesive and cohesive materials alike. The equipment comes in a range of sizes which provide testing for a range of particles from fine through to coarse. The shearbox consists of a 'hollow box' which is split horizontally and into which a sample is placed. The sample is consolidated to a desired normal stress and then sheared horizontally. The bottom half of the box is displaced whilst the top half of the box reacts against a load measuring device measuring the resistance to shear. These tests are usually carried out as a set of three tests where the first test is consolidated at half the calculated effective stress for the material, the second is at the calculated effective stress and the third is at double the calculated effective stress. The values from these tests should be used in total stress calculations only (although there is some argument). The only point at which we know the effective stress of the material (during the test) is at the end of the consolidation stage. As pore pressures are not measured throughout the following shear stage we are unable to verify the true effective stress of the material at failure and certainly along the plane of shear.

Consolidated peak strengths are obtainable from this equipment along with residual strengths.

Residual strengths in clays in particular are often in error from this equipment as it is not possible to form a perfectly flat shear plane. Residual values are reached only at very high strains compared to peak characteristics and the shearbox has limited travel. Some attempts to overcome this are in the test standards by reversing the direction of shearing until a consistent 'apparent' residual state is measured. For non-cohesive materials and silts this method may work, but due to the 'platey' nature of the clay particles, the perfect (flat and polished) shear plane can only develop by continued shearing in a singular direction (no reversals as per the standard shear box). This problem was overcome with the invention of the Ringshear apparatus (E. Bromhead). The normal loading and relative displacements induced during the test can be seen in **Figure 49.5** whilst a commercially produced ringshear apparatus is shown in **Figure 49.6**.

49.5.3 Ringshear test

Rather than linear movement of a block of soil, the ringshear rotates constantly and so the linear displacement (in a single direction) is limitless (or at least until all the soil has been 'squeezed' from the test annulus). Note that two opposing load measuring devices are used on this equipment. This is most necessary as the cell rotates and it is actually torsion which is measured. The use of two load measurements (being of matched stiffness) balance the top cap under rotation and are designed to prevent friction being measured from the central locating pin for the top cap.

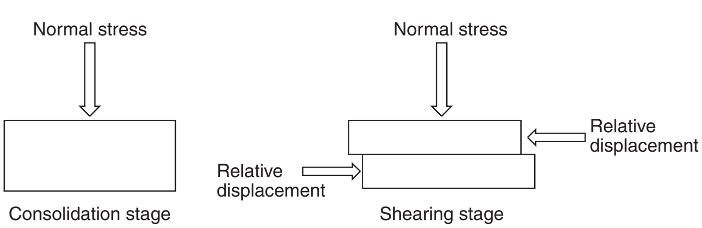

Figure 49.4 Loading and shear displacement in shearbox test

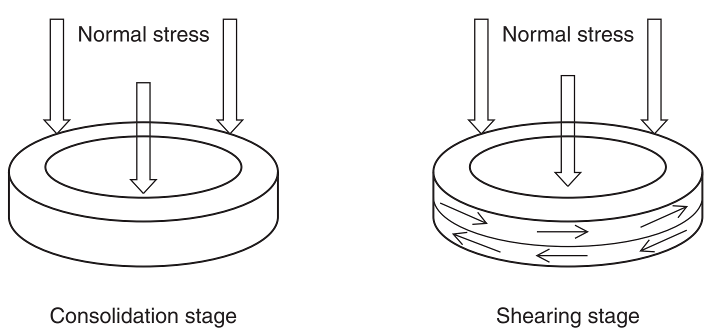

Figure 49.5 Loading and shear displacement in ringshear test

If we are interested in residual values (say for slope stability calculations) in clays, this is the test to perform. Peak strengths are not available from the 'Bromhead' or 'small' ringshear due to the fact that the sample must be remoulded as part of the test preparation. The material needs to be pressed into a tight annulus and benefits from having any structure or bonding destroyed. This allows for accelerated reorientation of the clay particles (shortening test times) and more repeatable test results through the homogeneity of the sample. This equipment is suitable for pure clays only. The presence of coarse particles may roll along the shear surface during the test and will destroy it by reorientating the clay particles. This will lead to higher (non-repeatable) residual values being obtained which may be disastrous where the true residual angle is actually lower still. For clays containing coarser material and non-cohesive samples the engineer should revert to the standard shearbox test (along with its known and well-documented limitations).

Figure 49.6 Ringshear apparatus
Courtesy of VJ Tech Ltd

49.6 Stiffness

There are many technical definitions of stiffness, but they all relate to the gradient of the line of stress plotted against strain and it is most important to consider that soils display a nonlinear stress–strain response (outside the highest levels of research). It should be remembered that the strength of the soil dictates its ultimate load-bearing capacity with large strains whilst stiffness identifies the compressibility (or strains) in the material at working loads (see **Figure 49.7**).

In general these parameters are not derived from the routine tests listed previously and require the highest quality undisturbed samples, specialist capabilities, instrumentation (**Figure 49.7**) and high levels of knowledge and experience. For stiffness of rock material reference should be made to Chapter 18 *Rock behaviour* for additional information. It is possible to fully instrument the following triaxial tests for the determinations of Young's modulus: UUP, CIU and CID. Shearing of samples may take place in either compression or

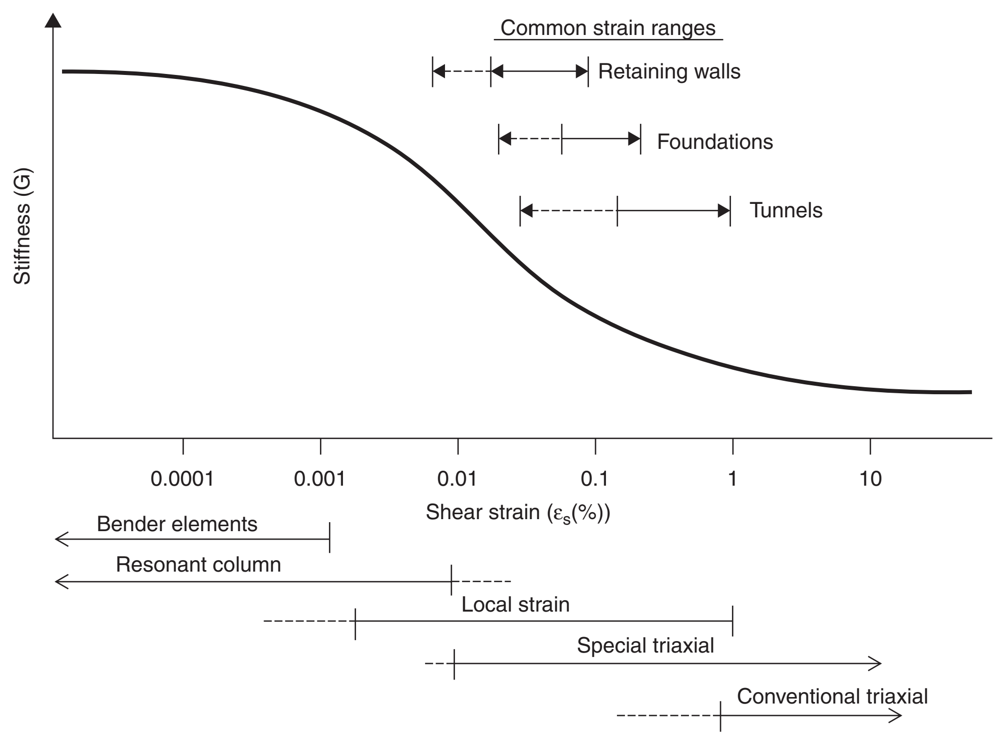

Figure 49.7 Idealised variation of stiffness with strain

extension and so a suffix may be added to the above abbreviations in the form of a C (compression) or E (extension).

The highest level of triaxial effective stress test is known as a stress path test or anisotropically consolidated undrained (or drained) triaxial test (CAU or CAD) and again may have the C or E suffix depending upon the final shearing direction. High-resolution transducers are attached directly to the sample to measure axial strains for the determination of Young's modulus and when combined with a radial strain transducer can be used to determine Poisson's ratio, Shear modulus (G) for undrained shearing and Bulk modulus (K) for drained shearing. In addition it is possible to measure Gmax using bender elements which measure sample stiffness beyond the resolution of local small strain instrumentation (see **Figure 49.8** for typical advanced instrumentation).

These tests are designed to take the specimen through its recent stress history in order to minimise any effects of sample disturbance and return the specimen to its true *in situ* mean effective stresses or to a particular stress level required for modelling.

Figure 49.9 shows an advanced triaxial test with local axial and radial strain instrumentation, base/mid-plane pore water pressure measurement along with measurement of Gmax using bender elements in all three possible directions. This specialist

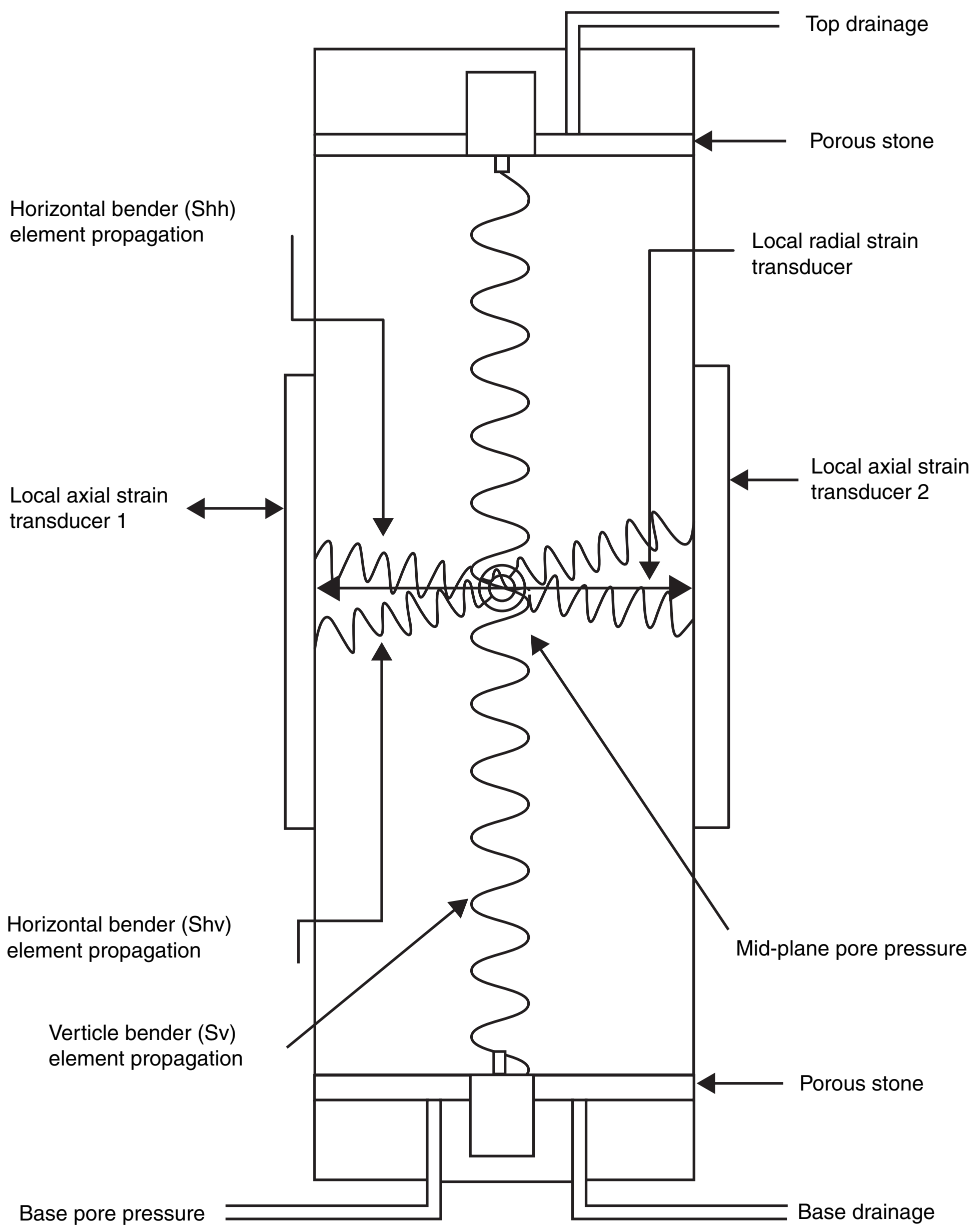

Figure 49.8 Typical advanced triaxial instrumentation for CAUC, CAUE, CADC and CADE test types

Figure 49.9 Triaxial stress (CAUC) tests with advanced instrumentation
Courtesy of Russell Geotechnical Innovations

instrumentation is used not only for the measurement of very small strain stiffness, but the utilisation of both base and mid-plane pore water pressure measurement allows verification of effective stress measurement.

The use of local axial and radial strain transducers allows the measurement of small strains directly on the sample and minimises the boundary effects (bedding and localised stress distributions) caused by the soil being in contact with the (much stiffer) end platens. Bender elements measure stiffness at even lower strains than local strain instrumentation and provide a completely non-destructive measurement of stiffness. They can be mounted in three orientations on a triaxial specimen and provide a very good indication of general sample condition and/or possible anisotropy.

During normal triaxial testing the pore water pressure is traditionally measured only at the base of the sample throughout the test. As the sample is in contact with far more rigid (usually metal) base pedestal and top cap, the areas at the specimen ends suffer boundary effects caused by the contact with the far stiffer material. For the calculation of shearing rates (time to failure) as per BS 1377:1990 we calculate that pore pressure dissipation is 95% only at the time of failure of the sample. The use of mid-plane pore pressure allows a second reference for pore pressure measurement and not only is unaffected by the metal pedestal (as it is in the central region of the sample), but can be used to display full pore water pressure equalisation throughout the sample (and not just at one end). This is ideal for drained shearing stages where the base and mid-plane pressures should remain the same if we are shearing at the correct rate. If the mid-plane pressure begins to deviate (increase) from the base measurement then excess pore pressures are being generated due to the sample being sheared too fast. For undrained shearing the pore pressures should react in unison and in the same direction (whether in compression or extension) again providing evidence that the correct shearing rates have been used. Interestingly, deviation between the base and mid-plane may occur later during sample rupture as pore pressures may be generated/dissipated in different ways depending upon the inclination and orientation of any shear planes which may form in the specimen (and continued shear along rupture surfaces).

For rocks we can measure stiffness by carrying out a more advanced version of the uniaxial compressive strength test which also has additional high-resolution instrumentation which measures axial and radial strains. Because rock specimens fail at strains (usually an order of magnitude) lower than those of soils, specialist strain transducers (unsuitable for soil testing due to their very small range) are bonded axially and radially to the central third of the rock specimen. These are logged throughout the loading test and used to measure Young's modulus and Poisson's ratio.

Again, high technical competence is required for these tests. An ideal example is that the bonding agent used to bond the strain transducers to the sample must be able to not only prevent the gauges from creeping on the sample by bonding the gauge fully and remaining so throughout the test, but be of a lower stiffness than the sample itself so that they measure the natural strains evolving in the sample under load rather than the artificial strains caused by a stiffer bonding agent which may have locally filled the voids within the sample (causing a localised 'stiffer' response).

49.7 Compressibility

This is a term often used in soil mechanics and largely describes the relationship between stress and strain. It is the stiffness of the ground which determines the strains and displacements with changing stress and so by combining the stress level and stiffness of the material its compressibility can be determined. Laboratory tests associated with these parameters are:

- *Oedometer consolidation test*. Useful for Cv and Mv. The coefficient of consolidation (Cv) has the units M^2/year which is the 'scaled up' time (for consolidation) from the laboratory test to the full-scale field material being modelled. The coefficient of volume compressibility (Mv) has the units M^2/kN and is the slope of the porosity against the applied effective stress curve resulting from the test. Since porosity and void ratio are related quantities it also follows that Mv can be calculated from the void ratio against effective stress curve. It should always be remembered though that the value of Mv is dependent on the stress level applied.
- *Hydraulic cell consolidation test*. This is the full effective stress version of the above and is able to impose various loading conditions and drainage paths on the specimen. In addition the hydraulic cell can be used to measure the permeability of the sample at each consolidation stress along the drainage path used for the test.

Typical test equipment for these two types of test can be seen in **Figure 49.10** and a simplified line drawing of the loading and boundary conditions is given in **Figure 49.11**.

Reference should be made to **Figure 49.12** for an idealised consolidation curve. The first stage (primary consolidation) is the result of reorientation and re-packing of the soil particles with the expulsion of water from the voids. Secondary consolidation is the actual compression of the soil particles themselves with a further expulsion of water (mainly from the particles as they themselves compress). It is most important to identify the presence of organic materials, especially peats, and understand their compressibility characteristics as settlements may be orders of magnitude higher than those associated with non-organic soils. Both the oedometer and hydraulic cell can also be used for secondary consolidation or 'creep' monitoring. Secondary consolidation is the continued compression of a soil after primary compression is complete and is caused by viscous behaviour of the soil grain–water system or the physical compression of organic matter. For quartz sands we expect

Figure 49.10 Oedometer (left) and hydraulic cell (right)
Courtesy of VJ Tech Ltd

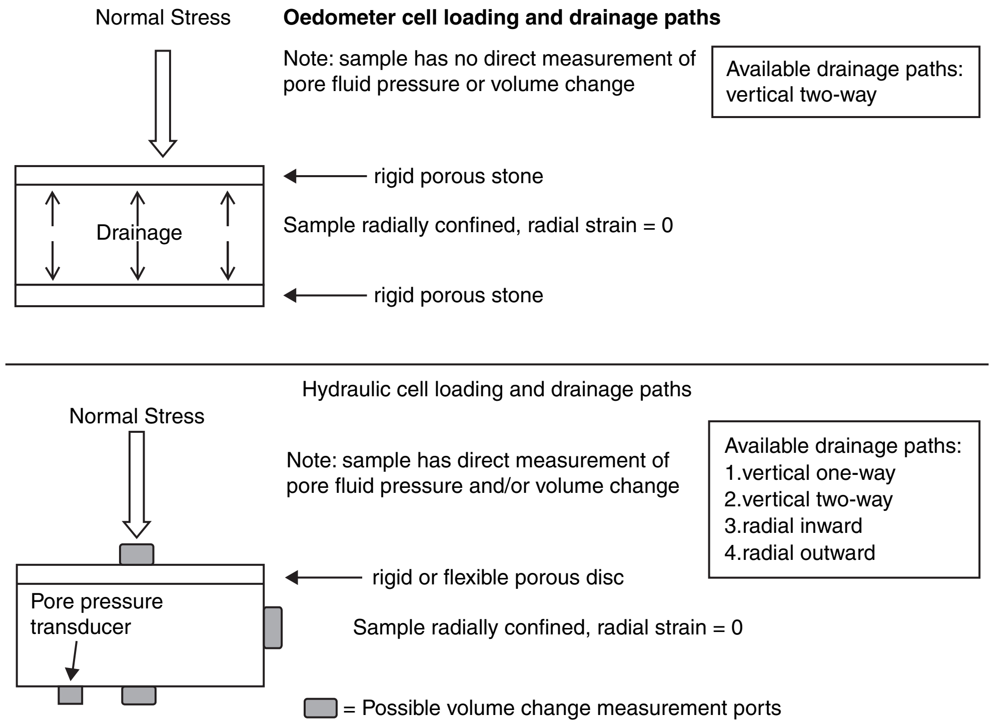

Figure 49.11 Oedometer (top) and hydraulic cell (bottom) loading and drainage paths

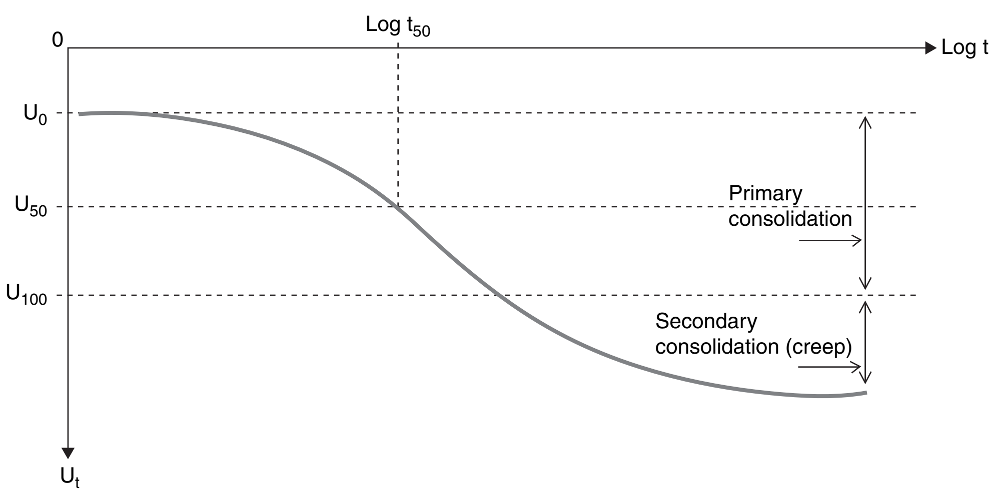

Figure 49.12 Log time consolidation curve

that creep (secondary compression) would be negligible as the sand grains are generally non-compressible. Other sand types (such as calcareous or carbonate sands) may show different behaviour or 'sudden collapse' if the normal stress exceeds the ultimate strength of the grains (and their asparites), leading to failure of the grains themselves. For peats and organic soils the soil 'solids' are themselves highly compressible and so a two-phase consolidation process is often seen with organic materials. Secondary consolidation also occurs in clays and can play important roles especially when dealing with very soft clays. When scheduling tests with the requirement for secondary consolidation parameters always bear in mind that these tests may take days, sometimes weeks per loading stage and have not only time but cost implications to match over the 'standard' tests where the interest is in primary consolidation only. It should also be noted that due to the fabric/structure of peaty materials they may show highly anisotropic behaviour especially with regard to drainage directions due to the orientation of the long axis of the vegetative material causing preferred drainage paths. In instances where these preferred paths have been identified it may well be preferable to schedule the use of the hydraulic cell and schedule a suitable drainage path/direction.

For rock materials we often assess compressibility from the intact strength of the rock material along with its discontinuity spacing and aperture (rock quality designation – RQD). It is possible to test rock samples for compressibility in high stress equipment; however, remember that you are only testing the intact material. *In situ* the bulk strength and compressibility of rock materials are usually dominated by their discontinuity spacing/orientation, aperture spacing/orientation and aperture contact areas. Other means (preferably field-based) should be used to assess these engineering characteristics.

It can be seen from the parameters gained from such tests that the soil structure and fabric (which controls drainage paths) are the controlling factors (material properties) that are being measured. Undisturbed samples of known orientation are the basic prerequisite for such tests. Re-moulded samples may be used if the construction requires design parameters to be used for such 'engineered' materials. Attention to sample quality and test preparation is paramount, especially as such small samples are tested and then the results scaled-up to the field model. Small errors magnify!

49.8 Permeability

For design purposes or interest in seepage problems we need to think about drained and undrained conditions or short- and long-term behaviour of the ground. This is dominated by the particle size, orientation and packing of the soil grains and whether they are cemented (in the case of hard pans or calcareous zones), cohesive or non-cohesive. Careful thought should be given to the sampling of such materials as the size, orientation and packing of the particles can lead to strong anisotropy *in situ* leading to high variations in permeability with flow direction. Sampling and the correct orientation of samples selected for laboratory testing is of paramount importance if true ground conditions are to be modelled representatively. Laboratory tests associated with the determination of permeability are:

- *Constant head permeameter*. For non-cohesive materials. Sample is radially confined, therefore lateral strain is zero.
- *Constant head permeability determination in a triaxial cell*. Normally for cohesive materials and where desired mean effective stresses are required. The sample is isotropically consolidated and a pressure differential applied (driving head) to the separate top and base drainage lines causing flow to occur.
- *Permeability determination in a hydraulic cell*. This equipment, mentioned previously, can be used to measure permeability along different vertical or horizontal drainage paths and can be carried out as part of a test which also gives consolidation parameters. For the possible flow directions available see **Figure 49.10**.

For rock samples, intact specimens can be tested (usually constant head), but it should be remembered that the permeability of the *in situ* material may be dominated by the presence of discontinuities and their aperture/orientation, infill, spacing and persistence.

49.9 Non-standard and dynamic tests

By mentioning the broad range of basic (and some more advanced) parameters which can be gained from the test types outlined previously, it would also be prudent to write a little about the more 'advanced' types of test which are also available in a few specialist laboratories. These tests are certainly not routine and require the use of highly specialist equipment and highly experienced staff.

Due to modern construction requirements and the need for design with dynamic loadings (such as wind turbine monopiles, etc.), geotechnical engineers are increasingly asked for the dynamic parameters more associated with those for foundation design in earthquake regions. Other tests are not necessarily 'dynamic' but are equally more towards the research end of testing. As with the anisotropic triaxial tests and their advanced instrumentation, these test types are often more appropriate to advanced numerical analysis designs and studies. They are certainly not routine and often come with a price tag to match; however, you do get what you pay for (as long as open and clear communication prevails throughout).

The following list is not exhaustive but is intended to highlight some of the more common research-level tests available and the apparatus associated with them.

49.9.1 Cyclic triaxial test

As it is named, this is a 'hybrid' triaxial testing frame which is built to 'cycle' the soil/weak rock sample by either stress or strain control around a mean level at rates commonly around 0.3 Hz with data capture of the transducers at many times a second. Cyclic strength depends upon many factors, including density, confining pressure, applied cyclic shear stress, stress history, grain structure, age of soil deposit, specimen preparation procedure, and the frequency, uniformity and shape of the cyclic wave-form (ASTM D5311). In addition it should be noted that non-uniform stress conditions are imposed by the specimen end platens which may cause a redistribution of void ratio within the specimen during the test. These tests are often carried out on non-cohesive soils and, since such materials are unable to withstand tension, the maximum cyclic shear stress that can be applied to the specimen is equal to one half of the initial total axial pressure (ASTM D5311). Care should obviously be taken in the design of such tests, and thought given to the fact that uneven pore pressure distributions throughout the sample may result depending upon the permeability of the soil and the rate at which it is cycled. Young's modulus and soil damping properties can also be evaluated for specialist design using this test type (ASTM D3999).

49.9.2 Simple shear

This can be carried out either as a monotonic test or as a dynamic cyclic test. The shear strength is measured under constant volume conditions that are equivalent to undrained conditions for a saturated specimen; hence, the test is applicable to field conditions where soils have fully consolidated under

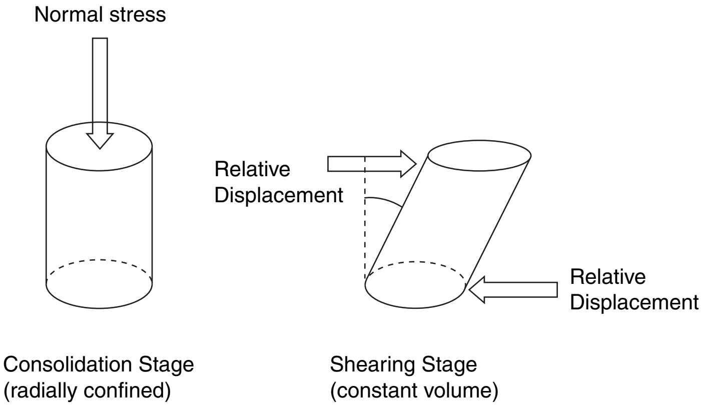

Figure 49.13 Simple shear mechanism

one set of stresses, and are then subjected to changes in stress without time for further drainage to occur (see **Figure 49.13**). The constant volume (undrained strength) is a function of stress conditions (plane strain) and the principal stresses continuously rotate due to the application of shear stress. This simple shear stress condition occurs in many field situations including zones below a long embankment and around axially loaded piles (ASTM D6528).

49.9.3 Resonant column

Figure 49.14 shows a resonant column with the cell top removed. The top part of the equipment is used to induce torsional movement to the top of the sample by an electromagnetic drive system which can run at a range of frequencies to determine the resonant frequency of a sample.

Such tests and test equipment are used to evaluate the shear moduli and damping characteristics of soil at very small strain amplitudes. Although there are two distinctly different equipment types for these tests, both apply torsion/rotation to the top of the sample in order to find the resonant frequency of the material at a controlled stress. These test methods are non-destructive if the strain amplitudes are less than 10^{-4} radians and many measurements may be made on the same sample and with various states of ambient stress (ASTM D4015).

49.9.4 Hollow cylinder test

Certainly the rarest of commercial tests, this equipment allows a rotational displacement to be imposed on a ‘hollow’ cylindrical specimen where independent control can be maintained for all three principal stresses (unlike triaxial tests which can only independently control two of the three principle stresses (where $\sigma_2 = \sigma_3$)). For this reason studies can be made of the intermediate principal stress (σ_2), sample anisotropy and the effects of principal stress rotation. These are ‘research level’ tests and the parameters derived are usually only used for the most advanced numerical analysis. Tests are available for both soil and rock and the hollow cylinder is most useful in the definition and determination of anisotropic material properties.

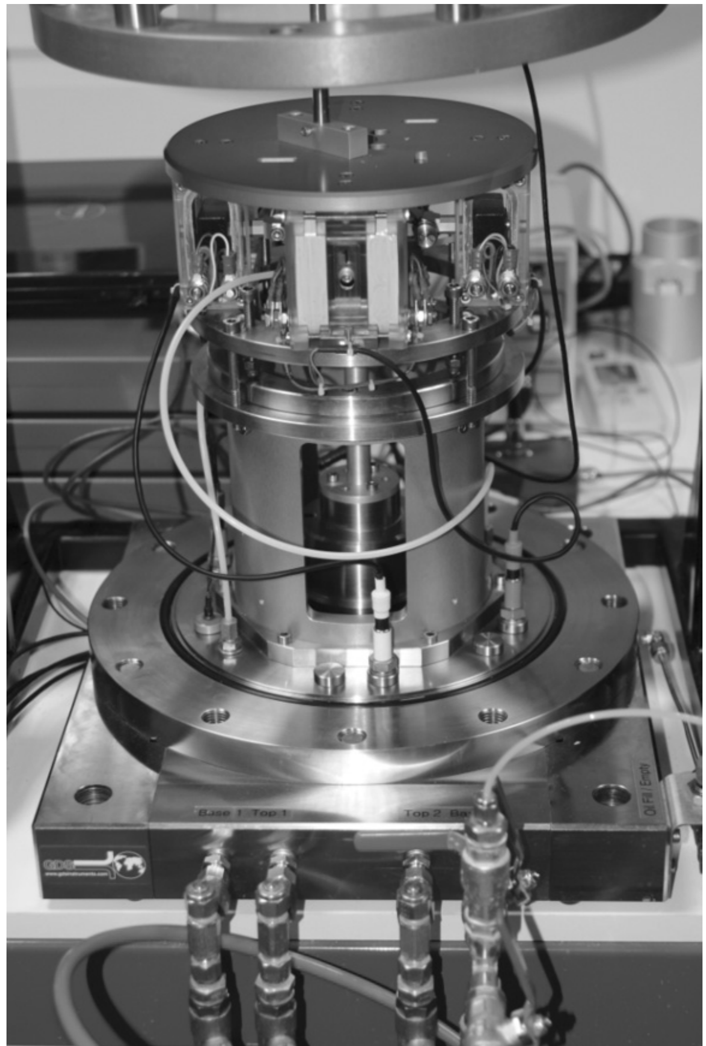

Figure 49.14 Resonant column apparatus
Courtesy of Russell Geotechnical Innovations

Simpler but equally non-routine tests can be used to model specific behaviour of soils and materials. The shearbox can be used to model the frictional behaviour which occurs at the interface between soil and a geotextile/steel/concrete surface. A ringshear can be used to model the soil behaviour at an interface between soil and a steel pile (Jardine *et al*., 2005). In addition tests can be carried out to determine the dispersion or erodibility of soils and rocks when abraded or exposed to persistent high moisture contents or flowing water. The tests listed in this section are certainly non-exhaustive and complete volumes could be written about laboratory testing and parameter determination. A good general series of volumes to read are those by Head (1986), *Manual of Soil Laboratory Testing*.

49.10 Test certificates and results

Test results are issued as certificates which identify the parameter requirements, data and graphs in accordance with the test methods and standards used. There should be no problem requesting further detail from the laboratories involved, to understand or verify any particular test conditions or methods used. It is here that open communication is important, and always remember that you are the paying customer. This line of communication should also allow the laboratory to freely communicate any observations or potential sample problems which may lead to unexpected results. It is this knowledge sharing that will improve the resultant parameter and overall design quality.

In order to supplement our ground model and design requirements we can see that careful planning at all stages of the ground investigation process is required and hence, why the natural process order of this chapter has been reversed somewhat. This process of investigation from sampling techniques through to sample storage, transport and laboratory testing requires careful planning and supervision. Any loss in integrity of the sample material properties at any stage in this sequence could have catastrophic effects on the parameters supplied from the testing house. These parameters are largely interlinked, especially moisture content and the sample physical integrity which dominate the effective stress characteristics. Geotechnical engineering is one of the only sciences which starts with a natural material being removed from a stable environment (the ground) and taken through a series of potentially damaging processes, through water addition/loss, exposure to atmosphere, physical handling (including jarring and vibration), temperature cycles and, finally, to a 'stable' environment where the material is tested for specific parameters which are to be representative of the ground from whence it came. How do we do it? This is the difference between a good site investigation (and all of the processes involved therein) and a not so good site investigation.

Fundamental in this way of thinking is the preservation of the sample moisture content. The determination of moisture content in the laboratory is only accurate if the sample retains the same moisture content that it had at that time in the ground. The determination of bulk density, strength, effective stress and stiffness, to name but a few, are all controlled by moisture content. If this is allowed to change between sampling (or altered by the sampling method) and the final test then the design will be based on erroneous values.

To realise this from the planning stage will help to identify possible problems and build them into the specification, and drive the ground investigation in the correct way from the start. If all stakeholders are involved in the initial stages and the project expectations and responsibilities are clear then the majority of these 'integrity loss' components can be minimised. The sample tested in the laboratory is only as good as the sample received there and this also assumes that the correct test was scheduled and that the laboratory was proficient in that particular method.

49.11 Sampling methods

This part of the project will have been conceived at an early stage as the ground investigation contractor will need the relevant equipment and correct experience for the project awarded. They will need to extract the samples from the ground and to deliver them to the laboratory for testing in the best possible condition. Here a chain of custody is formed and the 'smooth' operation of this will depend upon the sharing of information and open communication which will allow some flexibility to be built in for 'on-the-job' improvement. In an ideal world the laboratory will test a soil or rock which is in the same condition (and is therefore entirely representative) of the material *in situ*. This chain begins with the excavation of a trial pit or the drilling of a hole. A sample is then taken in various ways (to be explained later in more detail) and sealed in order to maintain its integrity. This sample is then either stored or immediately transported to a laboratory for testing, where again it may be stored (in a queue) whilst awaiting testing. Sampling for chemical and contamination testing is dealt with in Chapter 48 *Geo-environmental testing* of this volume and should be referenced as necessary. Here we are dealing specifically with sampling of the ground for the physical testing required for design parameters. There are many sampling methods available globally, but may be categorised simply as bulk samples, block samples, tube samples and rotary-cored samples. Each category has associated levels of disturbance and some indication of these is given along with the basic requirements for sample preservation. The following list is certainly not exhaustive but should indicate the main principles for 'good practice'. The reader must also adhere to the provisions of Eurocode 7 or other prevalent standards depending upon the geographical location of the investigation or agreed project requirements. Eurocode 7 is very prescriptive in terms of the sample types which may be used for various types of test as are many of the other standards generally used worldwide.

49.12 Bulk samples

This constitutes probably the simplest but the most disturbed sample type. Samples are often hand- or machine-excavated from a trial pit or spoil heap and placed in bags for logging purposes or index tests only. These samples should be of sufficient size to be representative of the horizon of interest, uncontaminated by material from other horizons, and of sufficient quantity for the testing required.

For bulk samples the material should be sealed in a bag with as much air evacuated as possible in order to prevent the sample from 'sweating' or the production of mould during storage. No samples, even if fully sealed, should be left in sunshine as this will not only cause the sample to 'sweat', but will also cause non-uniform heating leading to expansion/contraction of any fissures or textural fabric or aid the growth of mould, fungus or microbial organisms which again may alter the material properties. Any sample should be kept at a constant temperature and away from any localised heat sources. In the UK such temperatures should be no more than 20°C and no less than 5°C (under which the sample may begin to freeze). This is a range of temperature in which the sample may be kept, but it should not be cycled more than 3°C over a mean temperature if at all possible.

49.13 Block samples

Block samples are undisturbed hand-dug blocks which are usually some 0.5 metres square or diameter and at least 0.3 metres in depth. They are then sealed with an impermeable barrier followed by a rigid supporting container constructed around them. This is followed by careful paring from the substrata and removal for total sealing and support for storage and transport. Bearing in mind the preservation of the sample which is required, site- and environment-specific methods would be required for taking block samples and preserving the *in situ* characteristics of the medium sampled. Such samples are limited by access and depth of interest as space is required for personnel to cut the block safely and extract it from the horizon of study.

49.14 Tube samples

Tube samples are taken in a variety of ways depending upon equipment available and access.

The most common tube sampler used in the UK is the U100 and is often used in conjunction with light percussion drilling techniques (tripod-type rigs) where the tubes are driven into the ground. You will easily visualise the disturbance which the material may undergo by having a tube 'hammered' into it. This is part of the reason why U100 tube samples are not suitable for undisturbed testing parameters (but fine for index tests only and logging purposes). Tubes may also be 'pushed' into the substrate using piston-type (fixed piston) equipment which 'jacks' the tube into the bottom of the borehole. This method is superior and when used in conjunction with thin-wall sample tubes provides acceptable quality undisturbed samples. For thin-wall push samples, make sure that the end of the tube has been sharpened, has no burrs and that it is straight and true (at least before it is used). After sampling, also check that the tube has remained straight and true. If the tube has buckled or deformed during the sampling process then the soil within will have deformed (strained) as well, rendering it unsuitable for high quality parameter determination. For tube sampling the basic stages are outlined in **Figure 49.15** (Hight, 2000). From this we can determine the stresses to which the sample is exposed. Disturbance may be caused at any stage in the process but probably the most destructive is when the sample tube actually penetrates the bottom of the borehole (Hight, 2000).

For sampling to take place, the tube (and cutting shoe for U100s) is required to be pushed into the ground. Due to this additional material (sampler) being introduced into the natural soil the bulk density of the material into which they are inserted changes whilst the sampler is intruded into the bottom of the borehole. With this in mind we should in theory use a sampler which has as thin a wall and as sharp a cutting tip as possible in order to prevent local ground densification and undue strains or fabric disturbance. The sampler should be pushed in smoothly and without side movement or jarring (not percussive techniques). From this it can be understood that the area ratio (%) of a sampler plays an immense role in the disturbance of the sample taken. The area ratio is calculated as the volume of soil displaced by the sampler as a proportion of the sample volume (calculated by measuring the internal and external diameter of the cutting shoe or edge). In theory, the lower the value, the less disturbed the material within. Clayton and Siddique (1999) examined the different effects of tube geometries and used examples from the four main sample tube geometries used in the UK at that time plus a fifth experimental design. The geometries of these tubes are shown in **Figure 49.16**.

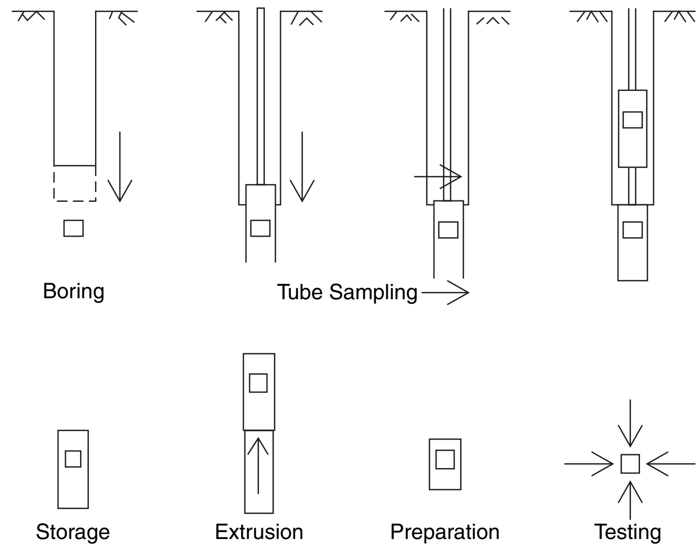

Figure 49.15 U100 Sampling stages
Reproduced from Hight (2000)

Sampler 1 is the geometry of the cutting shoe used on standard 'metal' U100 sample tubes and has an area ratio of 27%. Inside clearance is obtained by a step out from the cutting shoe where it screws onto the sample tube above it.

Sampler 2 is an upgraded version of sampler 1. It has a very similar area ratio, but with an inner step which is replaced with a slight taper and the cutting edge tapers that have been reduced (sharper).

Sampler 3 is the version of the UK cutting tube used for U100 samplers with plastic liners. The area ratio is considerable at 48% and again a small step inside produces the inside clearance between the shoe and the liner.

Sampler 4 is the 'thin-wall' push sampler which has been widely used in many circumstances to produce quite high quality samples. The original tube was described by Harrison (1991) and consists of a tube (normally stainless steel) with a 15° taper on the cutting edge. This tube is 'pushed' into the bottom of the borehole rather than 'hammered' like the previous samplers.

Sampler 5 is an experimental sampler (Hight, 2000) which is similar to sampler 4 but is sharper (5° taper) and has a 0.1 mm flat at the cutting tip.

This is not intended to be an academic publication but it is more than prudent to give some background to the sampler types with regard to sampling and disturbance and link them to the parameters required from the samples. From research

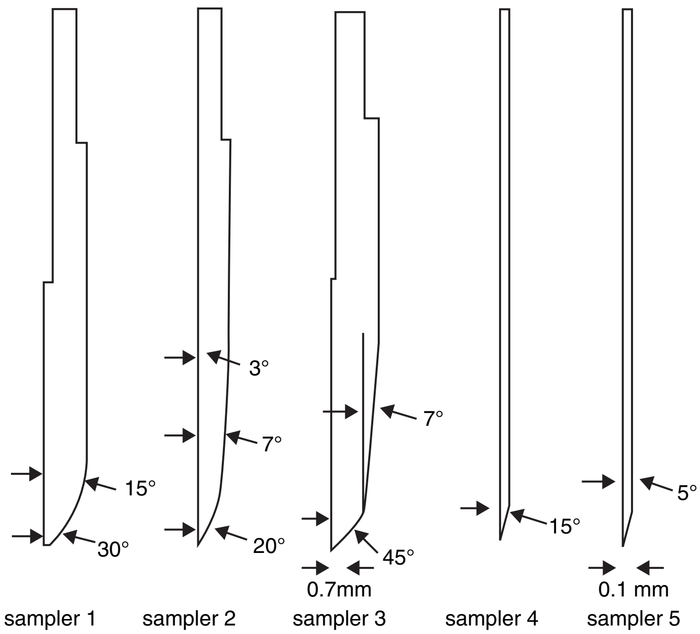

Figure 49.16 Tube sampler geometries
Reproduced from Clayton and Siddique (1999)

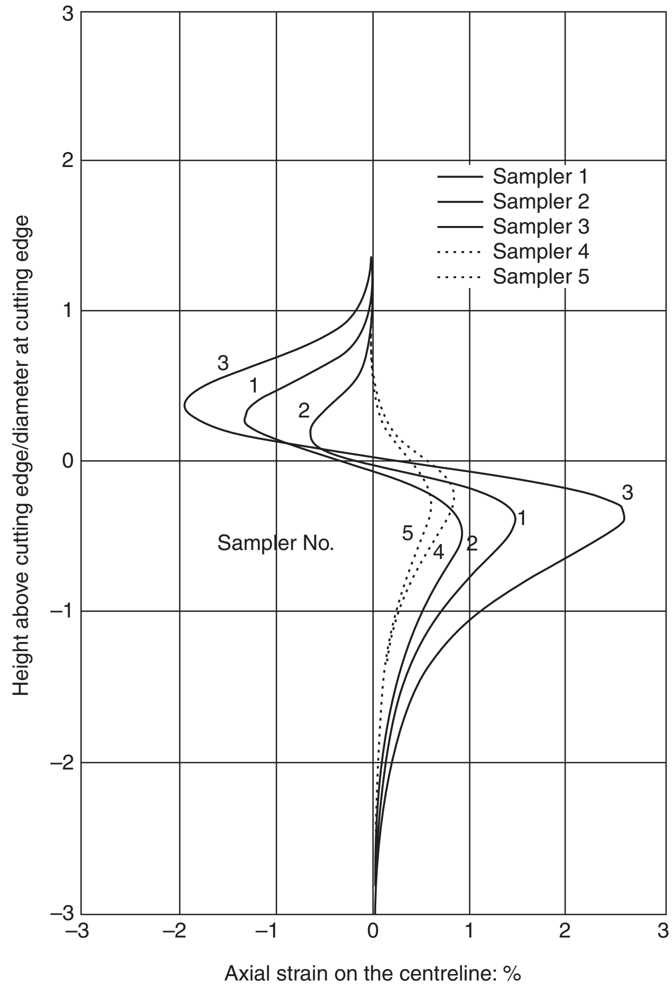

Figure 49.17 Sampler predicted axial strains
Reproduced from Clayton and Siddique (1999)

testing and some very high grade commercial tests it is known that on natural stiff clays such as London Clay failure occurs at axial strains in the region of 0.75–2.0%. Clayton and Siddique (1999) studied the sampler geometries and made strain predictions along the centre line of the sample as it would be taken using the different geometries (**Figure 49.17**).

From this we can see that samplers 1 and 3 are likely to fail the natural stiff clays during sampling due to the strains imposed on the specimen with sampler 3 being by far the worst offender. The thin-wall push samplers outperform all geometries modelled and sampler 4 should be used as a minimum if stiffness parameters are required. For normally consolidated and lightly overconsolidated clays Hight (2000) notes that the strains at the periphery of the sample, during sampling, causes a zone of re-moulded soil which combines with shear-induced pore water pressures which increase across the sample and are at their highest at the periphery. This leads to an overall reduction in mean effective stress caused by an increased water content in the centre of the sample as the sample re-equilibrates (due to the highly disturbed periphery). The same outcomes are true for overconsolidated clays along with damaged material structure and fabric.

Due to the high permeability of sands, tube sampling will be 'drained' and both volumetric and shear strains will occur (Hight, 2000). Levels of sample disturbance will vary with the *in situ* density of the material and destructuring/density changes will result due to the yield strain at the particle contacts being very low.

For all tube sampling methods it is imperative that the base of the borehole is cleared of debris before the sample is taken. If this does not occur, the tube will be filled with varying amounts of highly disturbed material which may have very high moisture contents leading to incorrect borehole logs and unrepresentative soil parameters.

On extraction from the borehole the sample tubes should be cleaned and any excess water immediately removed. The intact samples recovered should have their ends painted with low melting point wax and an identification label (indelible ink) placed in the top of the tube and marked as the 'top'. In cases where full recovery has not occurred, an inert non-compressible material should be used to fill the void left in the tube before the sealing caps are put in place. This is carried out to prevent the intact sample sliding around in the tube. The tubes should be stored and transported upright with the top of the sample uppermost. This is most important where soft samples have been taken which may try and 'flow' down the tube if knocked or vibrated during handling or transportation. Even the more competent samples may suffer from fissures opening up if stored on their side when moved or transported. The end sealing caps should be clearly marked with all the samples' details and way up, again with indelible ink. The tubes should be preferably marked both ends just in case the identification from one end is unreadable or removed (markings on the side of the tube are often rubbed off during handling and transport). A waterproof identification label should also be placed within the top of each tube as a failsafe.

49.15 Rotary core samples

This sampling method is probably the most common high quality method used extensively in the UK and in many circumstances is superior to thin-wall push samples. The advantage here is that the material is removed from the cutting face of the drill bit and so the sampler does not densify the soil as it is inserted but simply 'reams' a stick of intact material from the ground. The drawback is that some lubrication in the form of drill flush is required which means the possible addition of moisture to the sample; however, with sufficient expertise and thought this can largely be overcome.

Rotary drill rigs come in various sizes depending upon access and depth of hole required. A lorry-mounted rotary rig can be seen in **Figure 49.18** whilst a smaller tracked rotary rig (**Figure 49.19**) can be used for slopes or where there is limited access.

These rigs can use a variety of drill bits and drill flush agents depending upon the types of material encountered. They are far too numerous to list here, but if in doubt, drilling trials can be written into the ground investigation plan in order to verify sample quality and recovery before the main exploration phase begins. Typically triple barrel core tubes are used in which the innermost barrel is a semi-rigid plastic liner which enables the sample to be removed without undue stress. Using such equipment correctly and safely requires specialist contractors with knowledge and experience and care should be taken when choosing such contractors. With this method it is possible to sample not only rock, but firm through stiff to hard clays and compact sands. With the correct drill equipment, cutting heads and flush recovery should be very good.

Following is an overview of the field sampling procedure in order to identify the main processes. Variations are allowable depending upon particular requirements and conditions but these must be agreed by or scheduled by the client.

1. Gain a suitable sample from the ground.
2. Clean the sample in such a way that will preserve its strength and fabric retaining the properties of the same material *in situ*.
3. Sub-sample and/or trim the specimen to a size (normally H:D = 3:1) that is suitable for a laboratory test (triaxial or other).
4. Preserve the specimen so that it may be stored until testing is required.
5. Protect the sample from the effects of time and any changes to environmental conditions which may alter the properties or integrity of the sample such as storage temperatures, cycling ambient temperatures, transport shocks and vibrations, ultraviolet light.

The full specification for the preparation and storage of samples from drilling a rotary core ready to be tested in the laboratory may take the form of (but not be limited to the following):

- Remove the core from its liner immediately on extraction from the ground in order to remove drill fluids used and protect the natural moisture content and physical integrity of the core. The core liner should be split diametrically in two halves by using some form of counter-rotating opposing blades set so that they cut the plastic tube (liner) without cutting into or marking the sample within. The use of sharp knives for this operation should not only be prohibited for health and safety reasons but also due to the force required to penetrate and then pull along the tube. This may not only damage the sample within but there is a high risk of operator/spectator injury when slips occur.
- Clean off by wiping with an absorbent cloth any drill fluid or flush/water from the outside of the core.

Figure 49.18 Lorry-mounted rotary drill rig
Courtesy of Soil Engineering (formerly Norwest Holst Soil Engineering Ltd)

Figure 49.19 Track-mounted rotary drill rig
Courtesy of Soil Engineering (formerly Norwest Holst Soil Engineering Ltd)

- Log the core and sub-sample by carefully cutting for laboratory testing. Samples intended for triaxial testing should have an approximate length to diameter ratio of 3:1 (which will allow for later trimming in the laboratory).
- The sub-samples should then have their extremities prepared for sealing. Rotary-cored samples with very high permeabilities – compact sands for example should have any drill flush removed from their extremity and immediately be sealed. Clays should have the outer 5 mm of the sub-sample carefully trimmed off in a soil lathe in order to expose 'fresh' material which has not been contaminated with drill fluids. This process should be carried out very quickly for both material types but for slightly different reasons. For sandy materials it is important to prevent moisture loss (due to the relatively high void ratio and permeability) and for clay-type samples to prevent moisture ingress softening the sample and reducing its effective stress. Care should be taken that this process is carried out swiftly and in an environment suitable to reduce evaporation and localised heating of the sample.
- The sub-sample should then be sealed in such a way to trap minimal air, not only to maintain its natural moisture content but also its physical integrity for storage and transport. Often samples are wrapped in a layer of aluminium foil for the first layer, which is fine for most sample types as it can be lightly moulded in order to expel air and maintain contact with the sample surface as an impermeable barrier. Some thought should be taken though as salt water and some alkali pore water along with alkali minerals (gypsum) can react with the aluminium causing loss of sealing and reaction with the sample itself. In these instances non-permeable plastic film should be used as the first layer. Traditionally (and because it is commonly available from the local shop) many core sealing operations are carried out using plastic food wrap. The only problem with this is that this stretchy plastic 'food wrap' is by its nature osmotic or semi-permeable, and therefore not ideal. It has been found that the plastic film used to wrap pallets is not only stretchy, strong and seals against itself but is non-osmotic and impermeable. The sample should then have a layer of such material wrapped round two to three times to completely encase it. A label should be enclosed providing the sample identity and orientation. The complete sample should then be coated with low melting-point wax (which is usually a mixture of 50% petrolatum and 50% paraffin wax) which is quite soft and 'sticky'. *Not pure candle wax* (as its melting point is too high). This wax should be heated only to the temperature required for melting and not to the point of boiling. This is not only a health and safety issue but the idea is that as soon as the wax contacts the cooler specimen it will set whilst transferring minimal heat to the sample. The sample should not be dipped in the hot wax pot but should be painted with a brush dipped in the warm wax. These alternating coatings/layers can be repeated in order to protect the sample further, but the sample identity should always be clearly visible. Strong tape such as carpet tape should then be wrapped around the ends of the sample to protect the wax and layered coatings from damage. Finally the sample may be placed in split core liner which is then taped in order to give support to the sample. This core liner (or suitably rigid material) should be cut to the length of the sample so that the sample is retained rigidly and is unable to move about within its support. Caps should be placed over the ends in order to complete the encapsulation. The sample should be relabelled in an indelible fashion with full identification and orientation (top/bottom).
- In essence the sample is to be preserved as near as possible to the condition as it was *in situ*, but isotropically de-stressed. This preservation must be able to withstand handling transportation and storage which in some instances may be for some appreciable time.

49.16 Transport

This is a simple process, but very often overlooked. The samples should be transported in such a way that they are not shocked, dropped or vibrated. If sample integrity is an issue then the samples should be protected accordingly. Personal delivery and transport within the chain is often the only way to maintain high integrity. External transport contractors often do not appreciate the care required when handling a piece of 'soil' and so this should be avoided. Many couriers see such parcels as just 'heavy' rather than highly fragile scientific material. The samples should be transported 'upright' and in a way that they are supported laterally (padded boxes) to prevent toppling, rattling and vibration. Even when transported personally, the samples should not be placed in footwells where they are near heating/cooling vents which may locally heat/cool the samples. This completes the chain of custody to the testing laboratory who will then test the samples for the parameters required.

49.17 The testing laboratory

There are many testing houses available throughout the world, and offer testing to many levels and standards. Care should be taken in choosing the testing laboratory as it is relatively simple for them to buy the equipment to carry out tests but this does not mean that they are proficient in that particular test. Make sure that you are clear about the parameters you require from your tests and the identity of the samples to be tested. Bear in mind that you will need sufficient soil sample of the correct quality in order for the tests to be representative. It is often a good idea to visit prospective laboratories, if possible, and again this promotes the sharing of information and keeping up-to-date with the latest testing and contractual developments. You will also be able to assess the level of expertise, the equipment and processes involved with your proposed testing. Some tests may be beyond the scope of external accreditation bodies (and the accreditors!) and in this instance the reputation and experience of the staff carrying out the tests will prevail.

Over the years the writer has seen some 'very interesting' practices carried out in a range of laboratories; some with high levels of external accreditation. These accreditations are not a guarantee of high quality but proof of a level of proficiency and management system on the day of the audit. Procedures, systems and tests are audited (often by external auditors) so that a certificate can be issued providing evidence of compliance and repeatability. However, this does not necessarily mean that the result issued will be correct. It is very possible to follow an incorrect method repeatedly and gain the incorrect results repeatedly and still be accredited for this! There are many good laboratories out there. Find them and use them.

The new Eurocodes are addressing the standardisation of modern drilling and sampling practices and their relative merits with regard to sample quality for laboratory testing. Hopefully the demise of the U100 plastic liner sampling system has arrived as samples from such tubes are 'highly disturbed' and are therefore of little value for laboratory testing apart from index properties. Strength, compressibility and permeability determinations will be in varying degrees of error to the material *in situ*. Where the intention is to study the variation of these parameters with depth, U100 tube samples are not good and often contribute to the 'scatter' we see in plots with depth. This, along with other 'mishaps' and lapses in attention or detail in the chain from sampling to testing, all adds to the error band or 'scatter'. Much of this can be avoided very simply through care and attention to detail. Unfortunately this is sometimes lost in the pressure of 'getting the job done'. The parameters we require from these samples for our design should be representative of the material *in situ* and not to the environments which the material has been exposed to on its journey from the ground to the laboratory (and subsequent test methods).

This completes the physical sampling/testing loop of ground investigation. You should have spare material if possible for alternative or repeat tests should you find the need for additional testing. It is very expensive to re-drill or have further boreholes sampled at a later date. All data from testing should be available if required for further analysis even down to the weights for moisture contents or the cone values and moisture contents for Atterberg determinations.

It is understood that in some instances and in some areas around the world some deviations from these guidelines will be required; however, the same principles and ideals should be followed. Notes should be kept as to the methods used and the environment in which the samples were taken and sealed including dates, times and personnel involved. Everything that we do as professionals should be able to withstand scrutiny but also provide sufficient data to repeat or improve our activities in the interests of a forward-looking science. The attention to detail in the complete chain of custody will pay dividends in the quality of the subsequent tests and parameters derived.

49.18 References

American Society for Testing and Materials (ASTM) (2000). ASTM D6528-07. *Standard Test Method for Consolidated Undrained Direct Simple Shear Testing of Cohesive Soils*. West Conshohocken, PA: ASTM.

American Society for Testing and Materials (ASTM) (2000). ASTM S4015-92. *Standard Test Methods for Modulus and Damping of Soils by the Resonant Column Test.* West Conshohocken, PA: ASTM.

American Society for Testing and Materials (ASTM) (2003). ASTM D3999-91. *Standard Test Methods for the Determination of the Modulus and Damping Properties of Soils Using the Cyclic Triaxial Apparatus*. West Conshohocken, PA: ASTM.

American Society for Testing and Materials (ASTM) (2004). ASTM D5311-92. *Standard Methods for Load Controlled Cyclic Triaxial Strength of Soil*. West Conshohocken, PA: ASTM.

Atkinson, J. H. (2007). *An Introduction to the Mechanics of Soils and Foundations*. London: Routledge.

Clayton, C. R. I. and Siddique, A. (1999). Tube sampling disturbance – forgotten truths and new perspectives. *Proceedings of the Institution of Civil Engineers Geotechnical Engineering*, **137** (July), 127–135.

Harrison, I. R. (1991). A pushed thinwall sampling system for stiff clays. *Ground Engineering*, April, 30–34.

Hight, D. W. (2000). Sampling methods: evaluation of disturbance and new practical techniques for high quality sampling in soils. Keynote lecture. In *Proceedings of the 7th National Congress of the Portuguese Geotechnical Society*, Porto, Portugal.

Jardine, R., Chow, F., Overy, R. and Standing, J. (2005). *ICP Methods for Driven Piles in Sands and Clays*. London: Thomas Telford.

49.18.1 Further reading

British Standards Institution (1990). *Methods for Soil Testing*. London: BSI, BS1377: Parts 1 to 8.

British Standards Institution (2006). *Geotechnical Investigation and Testing – Sampling Methods and Groundwater Measurements*. London: BSI, BS EN ISO 22475-1:2006.

British Standards Institution (2007). *Eurocode 7: Geotechnical Design – Part 2: Ground Investigation and Testing*. London: BSI, BS EN1997-2:2007.

Clayton, C. R. I., Simons, N. E. and Matthews, M. C. (1982). *Site Investigation*. London: Granada.

Head, K. H. (1986). *Manual of Soil Laboratory Testing*, 3 vols. London: Pentech Press.

Simons, N. E., Menzies, B. and Matthews, M. C. (2002). *A Short Course in Geotechnical Site Investigation*. London: Thomas Telford.

49.18.2 Useful websites

ASTM International (formally known as the American Society for Testing and Materials), contains many internationally recognised references for soil and rock testing; www.astm.org

Home of the British Geotechnical Association, contains information for updates of many relevant technical Standards and links to many other sites of interest; http://bga.city.ac.uk

British Standards Institution, references for UK and European Standards including training and accreditation; www.bsigroup.com

Engineering Group of the Geological Society (EGGS), many useful references for rock behaviour and categorisation; www.geolsoc.org.uk

International Society for Rock Mechanics, contains the European suggested methods for various rock tests (the 'Blue Book'); www.isrm.net

It is recommended this chapter is read in conjunction with

- Chapter 17 *Strength and deformation behaviour of soils*
- Chapter 18 *Rock behaviour*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles.* A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation.*

Appendix A
Standard soil and rock tests

The standards named here are generally applicable to the UK and should be used in association with the present guidelines required to apply to *Eurocode 7: Geotechnical Design – Part 2: Ground Investigation and Testing* (BS EN1997-2: 2007). Eurocode 7 is applicable to European construction and may be accepted in other parts of the world. It should be noted that different standards may apply depending upon the various locations around the world where the construction is to occur. The other major standards applied worldwide are the ASTM standards and these should be applied or used where applicable.

The list here is not exhaustive and is only an indication of some of the more common soil and rock tests available. It should also be noted here that the European standards for the identification and classification of soils and rocks (BS EN ISO 154688-1 (2002), BS EN ISO 154688-2 (2004) and BS EN ISO 14689-1 (2003) implemented into UK practice in 2007 have all been incorporated into BS 5930:1999 Amendment 1 which incorporates a revised section 6 (published in 2007). Earlier versions of BS 5930:1990 do not meet the requirements of the new Eurocodes and would not comply with the recent code changes.

Soil test	Reference
CLASSIFICATION/INDEX TESTS	
Determination of moisture content (MC)	BS1377:Part 2:1990, **3**
Determination of Atterberg limits (liquid and plastic limit, usually four-point cone method)	BS1377:Part 2:1990, **4, 5**
Determination of density	BS1377:Part 2:1990, **7**
Determination of particle density	BS1377:Part 2:1990, **8**
Determination of particle size distribution (PSD)	BS1377:Part 2:1990, **9**
COMPACTION-RELATED TESTS	
Determination of dry density/moisture content relationship (compaction test)	BS1377:Part 4:1990, **3**
Determination of maximum and minimum dry densities for granular soils	BS1377:Part 4:1990, **4**
COMPRESSIBILITY TESTS	
Determination of one-dimensional consolidation properties using a hydraulic cell	BS1377:Part 5:1990, **3**
CONSOLIDATION AND PERMEABILITY EFFECTIVE STRESS TESTS	
Determination of permeability in a hydraulic cell	BS1377:Part 6:1990, **4**
Determination of isotropic consolidation in a triaxial cell	BS1377:Part 6:1990, **5**
Determination of permeability in a triaxial cell	BS1377:Part 6:1990, **6**
SHEAR STRENGTH TESTS (TOTAL STRESS)	
Determination of shear strength by direct shearbox	BS1377:Part 7:1990, **4, 5**
Determination of residual strength using the small ringshear apparatus	BS1377:Part 7:1990, **6**
Determination of undrained shear strength in a triaxial specimen WITHOUT measurement of pore pressure (QUU)	BS1377:Part 7:1990, **8**
Determination of undrained shear strength in a triaxial specimen with multi-stage loading and WITHOUT measurement of pore pressure (QUU multi)	BS1377:Part 7:1990, **9**
SHEAR STRENGTH TESTS (EFFECTIVE STRESS)	
Consolidated-undrained triaxial compression test with measurement of pore pressure (CIU)	BS1377:Part 8:1990, **7**
Consolidated-undrained triaxial compression test with measurement of pore pressure (CID)	BS1377:Part 8:1990, **8**

Rock test	Reference
Preparation of rock core specimens and determination of dimensional and shape tolerances	ASTM D4543-08 or ISRM suggested methods (2007)
Determination of water content	ASTM D2216-10 or ISRM suggested methods (2007)
Determination of porosity/density using buoyancy technique (for both regular and irregular shapes)	ISRM suggested method (2007)
Determination of slake durability index	ASTM D4644-08 or ISRM suggested methods (2007)
Determination of point load strength for diametral and axial tests	ASTM D5731-08
Determination of splitting (Brazillian) tensile strength of intact rock core specimens	ASTM D2936-08
Determination of compressive strength and elastic moduli of intact rock core specimens under varying states of stress and temperatures	ASTM D7012-10