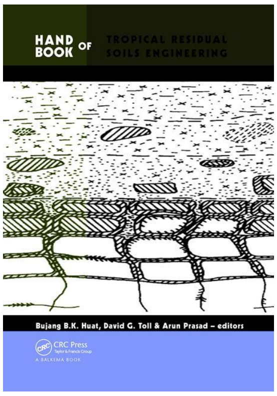


## Handbook of Tropical Residual Soils Engineering

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# Sampling and testing of tropical residual soils

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# 3

## Sampling and testing of tropical residual soils

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## 3.1 INTRODUCTION

The characterisation and assessment of geotechnical properties of residual soils is a complex subject given the fact that these soil formations are a product of the physical, chemical and biological weathering processes of the rock. This in-situ decomposition of the parent rock and rock minerals produces characteristic features of mechanical behaviour that cannot necessarily be approached by conventional geotechnical design methods due to one or more of the following reasons (Schnaid *et al.*, 2004a, 2004b):

1 The soil state is variable due to complex geological conditions.
2 Classical constitutive models do not offer a close approximation of its true nature.
3 These soil formations are difficult to sample and the soil structure cannot be reproduced in the laboratory. As a consequence, the mechanical behaviour and geotechnical properties are assessed directly from in-situ testing data in most geotechnical design problems.
4 Little systematic experience has been gathered and reported, and values of parameters are outside the range that would be expected for more commonly encountered soils such as sand and clay formations of sedimentary soils.
5 The deposits are often unsaturated and the role of matrix suction and its effect on soil permeability and shear strength has to be acknowledged and accounted for.

Ground investigation of residual soils often reveals weathered profiles exhibiting high heterogeneity in both vertical and horizontal directions, complex structural arrangements, expectancy of pronounced metastability due to decomposition and lixiviation processes, presence of rock blocks and boulders, among others (e.g. Novais Ferreira, 1985; Vargas, 1974). The process of *in situ* weathering of parent rocks (which creates residual soils) gives rise to a profile containing material ranging from intact rocks to completely weathered soils. Rock degradation generally progresses from the surface and therefore there is normally a gradation of properties with no sharp boundaries within the profile. Lateritic and saprolitic horizons have to be distinguished because of their different geological history. Lateritic soils are formed under hot and humid conditions involving a high permeability profile which often results in a bond structure with a high content of oxides and hydroxides of iron and aluminium. In saprolitic profiles, the original disposition of the decomposed crystals of the parent rock is retained. This gives to the soil a peculiar relic structure where the soil grains are well arranged and orientated (e.g. Novais Ferreira, 1985; Vaughan, 1985).

In this highly heterogeneous environment, site investigation campaigns are generally implemented from a mesh of boreholes associated to either standard penetration test (SPT) or cone penetration test (CPT), to depths defined by the capacity of the penetration tool, and followed by continuous rotational coring below the soil-rock interface for a global geological characterisation of weathering patterns. To enhance consistency, recommendations are made to encompass geophysical surveys. In this highly variable environment, both laboratory and in-situ tests still assist in characterizing stress-strain and strength properties. However, the geotechnical practice in residual deposits has tended to diverge from the practice established for sedimentary deposits, both in terms of the applicability of commercial investigation tools and the interpretation of testing data in order to assess geotechnical design parameters. One of the aims of this Chapter is to highlight the differences in practice, procedures and theoretical background.

## 3.2 SAMPLING

Sampling and testing are prerequisites in determining the index and engineering properties of soils. The first collection of state-of-the-art papers in sampling of residual soils was presented at a symposium in Singapore organised by International Society for Soil Mechanics and Foundation Engineering (ISSMFE) in 1979 (Nicholls, 1990). International practice on sampling and testing of residual soils was reviewed by ISSMFE in 1985. ISSMFE emphasises the need to take care in sampling residual soil, which usually behave differently from other conventional soils particularly in relation to properties and structure. Further description on how sampling will be done is given below.

### *Criteria of good quality samples*

Soil samples provide some of the most important evidence from a subsurface investigation. The preparation of soil samples for testing demands special care, and accepted practices may not be suitable and should not be applied both for the preparation of disturbed samples for classification testing or for the handling of disturbed samples for shear strength, compressibility and permeability tests. Conventional index tests should be used with caution when correlating with engineering behaviour. The void ratio of these soils varies considerably and strongly influences their engineering properties.

As stated in Geoguide 3 (1996), assessment for in-situ material is essential. The assessment should be done with great care and should not be influenced either by optimistic estimations of a lack of disturbance or by the requirements of any analytical programs to be used for design. A project requirement for a particular undisturbed parameter value does not necessarily mean that it can or should be supplied by direct sampling and testing methods. Disturbance needs to be assessed both with respect to sampling in the field, transport of the sample and transfer of the recovered sample to the test apparatus in the laboratory. Geoguide 3 (1996) also states that the test procedures require to be selected on the basis of project requirements, general material types, sample quality and the capability of the test laboratory.

The 'Good Quality Sample' or 'Perfect Sample' is defined as a 'sample which has not been disturbed by boring, sampling and trimming but has experienced stress released'. In actual conditions, there is no truly undisturbed sample, as all drilling techniques will eventually initiate some mechanical disturbance and stress relief. Samples are classified as undisturbed or disturbed depending on how much alteration there is to the soil structure after it is removed from its in-situ state. The validity of investigations carried out in laboratory tests rests solely on the quality of the samples and on how far they are representative of the stratum from which they are taken. The samples taken are said to be 'Good Quality Samples' depending on various factors such as purpose of sampling (what type of testing will be performed?), the location of samples taken (must be representative of an area), the method of sampling applied, and how the sample taken was handled before (including preparation for testing) and during testing.

### *Purpose of sampling*

The samples must be taken according to the type of test to be performed, and whether an undisturbed sample is needed. If an undisturbed sample is needed, it must be taken and handled accordingly before and during testing to preserve the sample. Undisturbed samples should represent the actual condition on site where the structure and water content is preserved as far as possible and can be obtained by using a suitable coring method. As far as residual soils are concerned, the disturbance of mechanical force can alter the structure and soil fabric. Therefore, to handle this type of soil, special mechanical tools are used. It is important to minimise disturbance of the samples as far as possible especially for tests of shear strength, compressibility and permeability. The number and spacing of disturbed samples usually depend on the anticipated testing programs and design problems. Some of the methods used to preserve the actual condition of the soil will affect the location of samples taken, techniques for sampling, and sample storage.

For the purpose of sampling which involves remolding soil or changing of moisture condition from the field condition, there is no need to preserve the samples in an undisturbed state. However, the need for undisturbed samples is for soil identification and classification and quality tests, in which case the samples are usually collected and sealed in glass or plastic containers, tins or plastic bags. The representative disturbed samples should be taken vertically less than 1.5 m and at every change in strata.

### *The location of samples taken*

The samples must be representative of an area for the purpose of generalisation of the properties for identification and use in engineering design. Generally, obtaining representative soil samples is still a challenge since, by nature, soil types and their properties vary greatly both vertically and horizontally. It is more difficult when dealing with tropical residual soils due to their non-homogeneous and anisotropic condition. It is important to establish a well-planned sampling program that ensures representativeness of the area.

The correct method of sampling applied is important for good quality samples. The sample for tests which require undisturbed samples should be obtained in the way prescribed, and likewise for disturbed samples. Personnel who are given the task of sampling should be familiar with various sampling techniques, and should be able to decide the right technique for undisturbed sample purposes. The soil fabric, bonding between particles, void ratio and moisture content of the soil must be preserved because these factors have a significant influence on their shear strength. The only way to get good quality samples of tropical residual soils is by fully utilizing the experience, competence and capabilities of personnel involved in the sampling and testing processes. Besides personnel capabilities, the laboratory and testing equipment must also be capable of producing good quality results. It is important to note that some of the conventional testing methods might not be reliable for tropical residual soils which may need some calibration and special tools.

*Drilling and sampling techniques*

The most popular sampling technique is by drilling. There are various methods of drilling widely used in obtaining soil samples, namely hollow-stem auger, solid flight and bucket augers, direct air rotary, cable and rotary diamond drilling. Different drilling techniques are necessary depending on whether the goal of the investigation is to collect undisturbed samples or if disturbed samples will suffice. A principal drilling requirement is that the driller must be prepared to encounter different consistencies of material from very soft to extremely hard rock, and for sudden and repeated changes from one to another. Other alternative sampling techniques are by open hand-dug pit or machine-dug pit. Brand and Phillipson (1985) stated the main reason for using this method is that the detailed inspection of a relatively large exposure of residual soil is highly desirable to enable an examination to be made of the occurrence of relict joints and other structural features, which can often dominate the engineering behaviour of the material. This method is most suitable at shallow depths in the completely weathered material. The pit must have enough space for person access. Through this technique, a visual inspection of the soil profile can be made. Hand-cut samples can be taken and other in-situ testing can be performed as required. As mentioned above, several sampling techniques can be applied to tropical residual soil, but with careful drilling techniques. Some practices use core barrels for high quality drilling and sampling. Types of core barrels used include a 63 mm diameter standard triple tube core barrel with retraction shoe and split steel liners (HMLC), an 83 mm diameter non-rectractable triple tube core barrel incorporating a wireline mechanism for withdrawing the liner barrel (PQ-Wireline) and a 74 mm Mazier automatic core barrel. Use of water in drilling, especially in dry tropical residual soils above the water table, must be avoided because it can disturb the samples. Sometimes, water from drilling activities in the hole is misunderstood as groundwater table. The water flush during coring causes erosion and loss of core. Water also increases moisture content on the surface of the sample. It is suggested that foam, mud, air or special drilling fluid is used. In the case of water being used in the drilling process, it is suggested that a Mazier automatic core barrel be used. A Mazier automatic core barrel using triple wall core barrel permits removal of the sample as it is taken from the ground, guaranteeing the 'in-situ condition' of the core. Figure 3.1 shows the diagram of a typical Mazier automatic core barrel.

A driven sample is also another alternative in obtaining undisturbed samples. The quality varies depending on the drilling tube used and its material, the condition of the cutting shoe, the area ratio and the method of driving. Samples of 35 mm diameter are frequently obtained in conjunction with the Standard Penetration Test. A strong open-driver sampler which is sometimes used, and which produces samples of much higher quality than the SPT, is the British U100. This gives 100 mm diameter samples up to 450 mm long. In order to obtain least disturbed samples, thin-walled stainless steel tubes of low area ratio are used on clayey saturated materials.

*Sample storage*

As suggested by Head (1992), samples should be kept in a cool room to protect them from extremes of cold and heat. Disturbed samples in glass jars can be conveniently stored in milk bottle crates. Large disturbed samples in polythene bags should not be

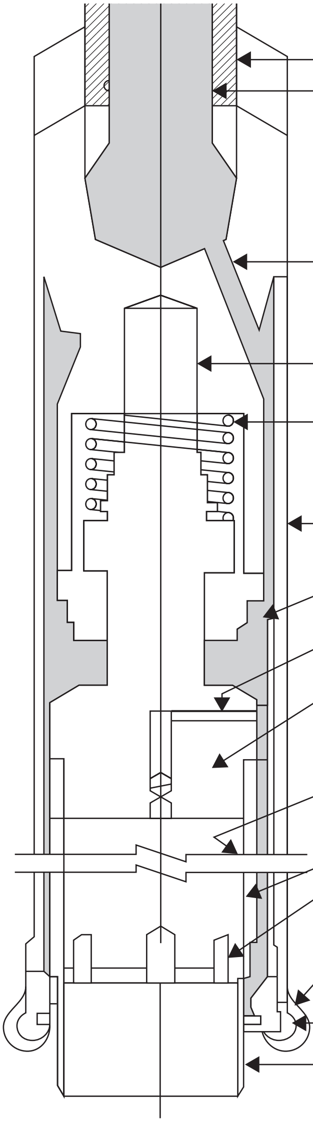


*Figure 3.1* Typical details of Mazier triple-tube core barrel

piled one on top of another, but placed individually on shelves or racks. Undisturbed samples must be laid on racks designed for storage; however tubes containing wet sandy or silty soil should be stored upright to prevent segregation of water.

### *Sampling of undisturbed samples*

Undisturbed samples are needed to obtain properties of tropical residual soils from laboratory testing. A complete disturbed sample is better than a bad undisturbed sample.

It is important to preserve the samples in an undisturbed state as close as possible to site conditions because some of the characteristics of tropical residual soils are sensitive, or have significant influence especially on their shear strength. Laboratory test results from a bad sample will distort the real properties of the soil, and the data cannot be used for safe and economic engineering design.

In tropical residual soils, the presence of bonding between particles either by cementation or interlocking gives a component of strength and stiffness which can be easily destroyed by any disturbance during sampling and testing of the sample. A second characteristic of tropical residual soils that may be affected by disturbance during sampling and testing of the sample is the void ratio. Tropical residual soils have widely variable void ratios in their mass which are unrelated to stress history. Changes of this void ratio due to rough handling may give false results in shear strength and permeability tests. Another characteristic of tropical residual soil is partial saturation, possibly to considerable depth, which can be responsible for disturbance during sampling as well as the behaviour observed in a test. Moisture in the samples must be preserved so it can be as close as possible to the moisture condition at the site upon testing. General types of disturbance that can cause significant effect on samples are:

1 Friction from cutting shoe during sampling,
2 Reduction of pore water pressure when the samples are brought to the surface,
3 Shock and vibration during transportation,
4 Storage, preparation and testing.

*Sampling of disturbed sample*

Disturbed samples can be readily gathered from all methods of site investigation. Tests for classification of soils such as moisture content, plasticity tests, shrinkage tests, particle size distribution and other related tests normally use disturbed or remoulded samples. All these tests involve drying and disaggregation of samples, which needs special attention.

Drying of samples at temperatures between 105 to 110°C to determine water/moisture content has a substantial effect on soil properties. Tropical residual soils are very sensitive to drying. Drying even at moderate temperatures may change the structure and physical and chemical behaviour of tropical residual soils. These changes are strongly influenced by the alteration of clay particles on partial dehydration and the aggregation of fine particles to form larger particles. The reformed particles remain in bonded position even on a re-wetting process. Results that reflect the drying effect are overviewed in the following section.

Crushing or splitting technique must be avoided in disaggregating tropical residual soils for purposes of the classification test (Blight, 1997; Fookes, 1997). The disaggregation process must be done with care and with regard for what is meant by 'individual particles'. The samples should be soaked in the water overnight. For particles which have a cemented characteristic, the disaggregation should be done by applying finger pressure only. The need for dispersion of fine particles is discussed in the particle size distribution test section below.

The degree of disturbance that occurs in a fabric-influenced material may vary considerably depending on the project or sampling conditions. A fundamental aspect

*Table 3.1* General divisions of geotechnical behaviour (*After* Nik Ramlan *et al*., 1994 in Geoguide 3, 1996)

| *Behaviour pattern* | *Description* | *Laboratory modelling* | *Project activity* |
|---|---|---|---|
| Instrinsic remoulded, de-structured material | Behaviour a function of particle type (mineralogy), shape and size (texture). Dependent on moisture condition | Completely remoulded index tests | Well compacted fill, road haul performance, erosion |
| Meso-structured undisturbed material | Behaviour is a function of intrinsic properties and the material fabric and meso-structure | Standard 'undisturbed' testing, triaxial, shear box, oedometer etc. | Possibly lightly compacted fill, erosion, aggregates |
| In-situ mass macro-structured mass | Behaviour a function of intrinsic, meso- and macro-structural properties of the mass and component materials, allied to the influence of relict mega-discontinuities and material boundaries | Only possible directly by combining relevant material tests with macro-structural data to give a mass character. Indirectly by semi-empirical, terrain correlation or back analysis procedure | Cut slopes, foundations |

Notes:

| | |
|---|---|
| Texture: | The morphology, type and size of component particles |
| Fabric: | The spatial arrangement of component particles |
| Discontinuities: | The nature and distribution of surfaces separating elements of fabric, material or soil-rock mass |
| Structure: | The fabric, texture and discontinuity patterns making up the soil-rock material, mass or unit. |

The above may be described at number of scale levels:

| | |
|---|---|
| Micro: | $<0.5$ mm Generally only described with the aid of SEM or petrographic microscope |
| Meso: | 0.5–5 mm Generally seen with the aid of field microscope or a good hand lens |
| Macro: | 5–50 mm Patterns visible to the naked eye in the field |
| Mega: | $>50$ mm Patterns that become apparent by means of maps or remote sensing, although individual elements may be visible at field level. |

of the characterisation of materials for civil engineering construction is in recognizing and establishing various levels of behaviour based not only on scale but also upon the influencing elements. The recognition of these levels of behaviour allows an important distinction to be made between inherent in-situ characteristics and those that become apparent during various aspects of construction. This facilitates a more relevant correlation between in-situ character, laboratory testing and likely project performance as shown in Table 3.1.

The following aspects should be considered in preparing remoulded residual soil; drying, disaggregation and sub-dividing. As soil is formed by the decomposition of rock in-situ by chemical decay, and may retain signs of its original structure, residual soils are likely to be highly variable and testing programs need to consider the use of both soil mechanics and rock mechanics testing procedures. The material might be considered in terms of being soil, rock or a soil-rock mixture. General classification should give an early indication of the general range of tests methods that it will be appropriate to conduct (Geoguide 3, 1996).

*Drying*

Partial drying at moderate temperatures may change the structure and physical behaviour of tropical residual soils. Some of the structures are changed by chemical means and not reversed when re-mixed with water. Physical changes can be seen according to these aspects:

1 Alteration of the clay minerals
2 Aggregation of fine particles to become larger particles that remain bonded even on re-wetting.

Fookes (1997) reported that clay soils often become more silt- or sand-like with a lower plasticity; although in some instances the opposite can occur. Oven drying from 105 to 110°C frequently has a substantial effect on soil properties, but drying at a lower temperature (e.g. 50°C) and even partial air-drying at ambient laboratory temperature can also produce significant changes. Blight (1997) and Fookes (1997) both agreed that generally all tropical residual soils will be affected in some way by drying. In preparation for a classification test, natural soil with as little drying as possible should be applied, at least until it can be established from comparative tests that drying has no significant effect on the test result. The method of preparation should be reported from time to time.

*Disaggregation*

Fookes (1997) stated that disaggregation should be handled with care. The objective of this process is to separate the discrete particles without the act of crushing and splitting. It is suggested that some soil should be soaked overnight without any interference from mechanical force to obtain better results. Particles with cemented bonds should be split only by using finger pressure.

*Sub-dividing*

Samples are sub-divided by a riffling box and poured evenly by using a scoop or shovel. The sub-dividing procedures must be acceptable for residual soil use so that representative samples are obtained and ready for testing. The soil should be evenly distributed along all or most of the slots to ensure that each container receives an identical sample. Other accepted quartering procedures can also be used to obtain required samples.

## 3.3 LABORATORY TESTING

Only routine laboratory tests will be described in this chapter. The specialised laboratory tests will be described in Chapters 5 and 6 of the Handbook.

*Index and engineering properties test*

It is accepted that residual soils behave differently from conventional soils (e.g. transported soil) as the soils are formed in-situ and have only slight changes in stress

*Table 3.2* Categories of water (*After* Geoguide 3, 1996)

| *Category* | *Description* | *Availability* |
|---|---|---|
| Structural water | Water held within the structure of component minerals | Generally not removable below 110°C except for clays such as halloysite, allophane and gypsum |
| Strongly adsorbed water | Held on particle surface by strong electrical attraction | Not removed by drying at 110°C |
| Weakly adsorbed water | Held on particle surface by weak electrical attraction | Can be removed by drying at 110°C but not by air drying |
| Capillary water (Free water) | Held by surface tension | Removed by air drying |
| Gravitational water (Free water) | Moveable water held in the materials | Removed by drainage |

history. Any changes strongly depend on mineral bonding and soil suction. Geoguide 3 (1996) distinguished major differences in undertaking and interpreting geotechnical laboratory test results in tropically weathered soil as opposed to sedimentary soils as follows:

1 The materials are chemically altered and sometimes bonded rather than produced by a physical sedimentation process.
2 The materials are in many cases non-saturated, exhibiting negative pore water pressures (soil suction).
3 There is difficulty in obtaining high quality undisturbed samples in these materials which may have a sensitive fabric.
4 There is difficulty in obtaining truly representative geotechnical parameters from these heterogeneous materials and masses.
5 Therefore, modelling residual soil testing might have to take into consideration these aspects which would require calibration of test equipment.

### *Moisture content*

The conventional definition of moisture content is based on the amount of water within the pore space between the soil grains when a soil or rock material is dried to a constant mass at a temperature between 105 and 110°C, expressed as a percentage of the mass of dry soil. This loss in weight due to drying is associated with the loss of the 'free water' as listed in Table 3.2.

For some tropical residual soils, in addition to 'free water' that is available to influence engineering behaviour there may exist crystallised water within the structure of minerals that is released at these drying temperatures. As suggested by Fookes (1997), in order to identify this type of soil, comparative tests should be carried out on duplicate samples taking measurements of moisture content by drying to a constant mass at between 105 and 110°C and at a temperature not exceeding 50°C until successive weighing shows no further loss of mass. A significant difference should indicate that intraparticle water is present. This water exists as a part of solid particles and should be excluded from the calculation of moisture content. The releasibility of this additional

*Table 3.3* Some examples of the effects on index properties of tropical residual soils (*After* Fookes, 1997)

| | | *Liquid limit* | | | *Plasticity index* | | |
|---|---|---|---|---|---|---|---|
| *Soil Location* | *Soil type* | *AR* | *AD* | *OD* | *AR* | *AD* | *OD* |
| Costa Rica | Laterite | 81 | – | 56 | 29 | – | 19 |
| | Andosol | 92 | – | 67 | 66 | – | 47 |
| Dominica | Allophane | 101 | 56 | – | 69 | 43 | – |
| | Latosolic | 93 | 71 | – | 56 | 43 | – |
| | Smectoid | 68 | 47 | – | 25 | 21 | – |
| Hawaii | Humic Latosol | 164 | 93 | – | 162 | 89 | – |
| | Hydrol Latosol | 206 | 61 | – | 192 | NP | – |
| Java | Andosol | 184 | – | 80 | 146 | – | 74 |
| Kenya | Red Clay, Sasumua | 101 | 77 | 65 | 70 | 61 | 47 |
| Malaysia | Weathered Shale | 56 | 48 | 47 | 24 | 24 | 23 |
| | Weathered Granite | 77 | 71 | 68 | 42 | 42 | 37 |
| | Weathered Basalt | 115 | 91 | 69 | 50 | 49 | 49 |
| New Guinea | Andosol | 145 | – | NP | 75 | – | NP |
| Vanuatu | Volcanic Ash, Pentecost | 261 | 192 | NP | 184 | 121 | NP |

AR: As Received, AD: Air Dried, OD: Oven Dried, NP: Indicates Non-Plastic.

water varies with mineral types and in some cases results in highly significant differences in moisture content between conventional testing temperature and engineering working temperatures.

### *Plasticity*

Plasticity or Atterberg limits, namely the Liquid Limit (LL) and Plastic Limit (PL) are often employed in the classification of fine-grained soils. Although water in a soil sample can be removed by oven-drying at a temperature of $110 \pm 5°C$ as in normal practice, it can also be removed by air-drying or it can be tested at its natural moisture condition as suggested in BS 1377:1990 (BSI, 1990). There are significant effects in determining plasticity of the soil with regards to pre-test drying, duration and mixing methods. Many researchers and writers state that tropical residual soils are very sensitive to drying because drying may change the physical and chemical properties of the affected soils. Table 3.3 shows some examples of the effects on index properties of tropical residual soils (Fookes, 1997).

### *Shrinkage*

Some tropical residual soils exhibit considerable volume change in response to wetting or drying and the shrinkage limit test may provide an indication of an intrinsic capacity for shrinking or swelling. The shrinkage limit test, as in BS1377 (1990), was initially intended for undisturbed samples although remoulded material can be used. Linear shrinkage to BS1377 (1990) is a simpler test on remoulded materials, which gives a

*Table 3.4* Effect of sample preparation on particle size distribution test (*After* Fookes, 1997; Blight, 1997)

| *Aspect* | *Description* |
|---|---|
| Drying | Drying can cause a reduction in the percentage of clay fraction. It is accordingly recommended that drying of the soil prior to testing be avoided. The initial soil sample should be weighed and a duplicate sample taken for moisture content determination so that the initial dry mass can be calculated. |
| Chemical pre-treatment | This treatment should be avoided wherever possible. Pretreatment with hydrogen peroxide is only necessary when organic matter is present. To eliminate carbonates or sesquioxides, pretreatment with hydrochloric acid is used. |
| Sedimentation | The dispersion of fine particles should be carried out by using alkaline sodium hexametaphosphate. In some instances a concentration of twice the standard value may be required. The solution must be made before conducting any sedimentation process. |
| Sieving | If accepted standard procedure is to be used, experience is needed to judge carefully. Care is necessary at every stage, especially to avoid breakdown of individual particles. |

linear rather than volumetric shrinkage. The established relationship between linear shrinkage and the Plasticity Index for sedimentary soils may not hold true for tropical residual soils. It is important to differentiate between materials that shrink irreversibly and those that expand again on re-wetting.

Studies by Mutaya and Huat (1993) on the effect of the degree of drying on linear shrinkage of Malaysian tropical residual soils show that when the degree of drying is increased, the linear shrinkage value reduces. This is due to the presence of moisture and clay content. A soil having higher moisture and clay content tends to shrink more compared to that of lower moisture and clay content.

### *Particle size distribution*

A particle size distribution test is done to determine the range of soil particles within a mass of coarse grained soil sample by the act of sieving. The complete test procedure can be reviewed in BS1377 (1990). Residual soil can be visualised as complex functions of particle size, fabric and the nature of the particles. The standard particle size distribution test should be applied with extra care concerning the aspects discussed and described in Table 3.4.

In one soil sample, the variation of particles sizes encountered may vary widely. Although natural soils are mixtures of various-sized particles, it is common to find a high proportion of types of soil occurring within a relatively narrow band of sizes. In some test procedures, some of the soils might undergo a sedimentation process by applying dispersants into the solution. The need for sedimentation of fine particles is to ensure that discrete particles are separated. Geoguide 3 (1996) explains that soils which undergo a sedimentation process should have proper dispersion of the fine particles. Alkaline sodium hexametaphosphate has been found to be suitable for a wide range of soils. Alternative dispersants such as trisodium phosphate may be more effective.

*Table 3.5* Methods of density measurement (*After* Geoguide 3, 1996)

| *Method* | *Reference* | *Comment* |
|---|---|---|
| Measured dimensions. Hand trimmed from block or tube | Part 2: 7.2 BS 1377 (1990) | Material has to be suitable for trimming; e.g. robust soil or weak rock. |
| Measured dimensions. Sample within tube | Part 1: 8.4 BS 1377 (1990) | Used where extrusion may disturb sample; e.g. loose or weakly bonded soil material. |
| Water displacement (waxed sample) | Part 2: 7.4 BS 1377 (1990) | Simple test used for irregular shaped water sensitive samples. |
| Weighed in water (waxed sample) | Part 2: 7.3 BS 1377 (1990) | As above, generally more accurate. |
| Weighed in water (non waxed sample) | ISRM (1981), Part 1 | Used for irregular lumps of rock-like material not susceptible to swelling or slaking. |

*Density*

Measurement of the quantity of material related to the amount of space it occupies is referred to by the term density. It is also normally understood as mass per unit volume. Density is widely used to obtain the relation between density and moisture content in the determination of compaction characteristics. Another test, the in-situ density, is another requirement for the assessment of structural stability and determination of the void ratio. This value can prove to be a useful index test, particularly as it may be used to correlate between soil and rock materials. Bulk density may be recommended using a variety of test procedures as summarised in Table 3.5.

*Specific gravity*

Specific gravity refers to the ratio of the mass of a given volume of a material to the mass of the same volume of water. This term however has been replaced by particle density as described below.

*Particle density*

The term 'particle density' (Ps) is replacing the previously used term 'specific gravity' (Gs) in current British practice, BS 1377:1990 (BSI, 1990) to comply with international usage. This term refers to the average mass per unit volume of the solid particles in a sample of soil, where the volume includes any preserved voids contained within solid particles. In other words, particle density is a measure of the average density of the solid particles which make up a soil mass. Particle density has the same numerical value as specific gravity although it has the units $Mg/m^3$ rather than being dimensionless. Particle density value is needed as the value can be used to determine other soil properties such as void ratio, clay fraction and porosity, which can be related to fabric structure.

Tropical residual soils may have highly variable particle densities. Some soils indicate unusually high, and some unusually low, densities. For this reason the value should be measured whenever it is needed in the calculation. An assumed valued is

*Table 3.6* Typical particle densities of minerals in tropical residual soils (*After* Geoguide 3, 1996)

| *Mineral* | *Particle density (Mg/m³)* |
|---|---|
| Calcite | 2.71 |
| Feldspar-orthoclase | 2.50–2.60 |
| Feldspar-plagioclase | 2.61–2.75 |
| Gibbsite | 2.40 |
| Haematite | 4.90–5.30 |
| Halloysite | 2.20–2.55 |
| Kaolinite | 2.63 |
| Magnetite | 5.20 |
| Quartz | 2.65 |

not encouraged. The test should be conducted at its natural moisture content and due regard should be taken of moisture availability problems discussed above. Natural moisture content is to be used in obtaining particle density. Any pre-treatment is not advisable as the value could be distorted and it tends to reduce the measured value as compared with the natural moisture content of samples. The dry mass of solid particles should be taken after the particle density test has been conducted. The dry mass is taken by oven drying at 105 to 110°C. Note that whenever coarse grain particles are present (gravel size), the gas-jar method is to be used so that the whole sample will be presented (Head, 1992). Some particle density values gathered from regional studies by Geoguide 3 (1996) are shown in Table 3.6.

### *Samples for compaction test*

Fookes (1997) reported that tropical residual soils with coarser particles are susceptible to crushing and therefore a separate sub-sample of soil is needed for compaction at each value of moisture content to avoid successive degradation. This also applies to the breakdown of the <425 μm fraction.

Care should be taken with soil samples that are sensitive to drying methods. Problems can arise when these are applied for compaction field control. The soil should not be dried before testing in the laboratory. If it is necessary to compact the soil lower than the natural moisture content, partial drying at room temperature is essential until the desired moisture content is achieved. Excessive drying, requiring re-wetting, should be avoided. For a given degree of compaction, drying has generally been found to increase the maximum dry density and to reduce the optimum moisture content. Data obtained from tests on dried soil are not applicable to the field behaviour and could result in inappropriate criteria being applied in field conditions (Fookes, 1997).

### *Shear strength tests*

Shear strength problems are usually encountered in calculation and analysis for foundation and earthwork stability. The shear strength parameters can be found by using laboratory and field tests, and by approximate correlations with size, water content, density, and penetration resistance. Some of the laboratory shear strength tests

*Table 3.7* Types of shear box tests

| *Soil sample* | *Type of shear box test* |
|---|---|
| Coarse-grained soil | CD test – strength parameter $c'$ and $\phi'$ |
| Fine-grained soil, cohesive soils (clays and clayey silts) | UU, CU and CD test<br>For UU and CU tests, the strength parameters in terms of total stress and the shearing rate have to be as rapid as possible |

applicable to residual soil are discussed below. The relevant in-situ tests are described in the following section.

*Shear box test*

The most common test used for determining shear strength of a soil in the laboratory is the shear box test. The shear box test is preferred because of its simplicity, ease of conducting compared with other tests, and less potential to disturb the sample preparation procedure than in the triaxial test. Another advantage is that the test can be used in fabric-sensitive materials by reverting to the use of a circular shear box which eliminates problems of sample disturbance at box corners. Despite its advantages, it does have disadvantages in some respects: drainage conditions cannot be controlled, determination of pore water pressure is not available, and thus only total normal stress can be determined. Total stress is equal to effective stress if full drainage is allowed and this requires the adoption of a suitable strain rate. This test however, enables relatively large strains to be applied and thus the need to determine both peak and residual soil strength. The principle is the action of a sliding movement to the soil sample while applying a constant load to the plane of relative movement. The soil sample can be directly sheared unconsolidated and undrained, but can also be consolidated and test drained or undrained. Various shear tests can be conducted depending on the designated model to present the actual condition. The tests are mainly unconsolidated undrained (UU), consolidated undrained (CU) and consolidated drained (CD) direct shear tests. Table 3.7 shows variations of test that can be conducted in the laboratory depending on the soil sample.

In slow 'drained' shear tests, the specimen is consolidated prior to shearing and a slow rate of displacement is applied during shearing. This method enables the consolidated drained or effective stress shear strength parameters to be determined.

Sampling of residual soils to verify shear strength of weathered zones should be carried as an additional concern. In almost all cuttings in tropical residual soils, there exist weathering zones which exhibit various degrees of alternating weathering grades, which is due to the nature of the parent rock. It is therefore inevitable that failures occur at the interfaces or relict joints. In anticipation of this situation specimens are obtained, as far as possible, at the positions of these discontinuities where the joint coincides with the sliding plane of the shear box. Undisturbed samples were obtained from the site using different types of sampling, from which test specimens were prepared. Briefly, the following test procedures are suggested (Blight, 1997):

- Test specimens are prepared and placed in the shear box with minimum disturbance. The specimens are then inundated with water.

- The normal stress is applied. In certain cases, the normal stress is equivalent to the overburden pressure of the specimen. When relatively soft specimens are tested, the load is applied in increments.
- Prior to shearing, in certain cases, the normal stress is reduced to the design overburden. Readings are taken until a stabilised condition is achieved (swelling has ceased).

Typical sizes of the square box sample for shear box test are 60 mm, 100 mm and, rarely, 300 mm or more. For circular shear boxes, common sizes are 50 mm and 75 mm diameter. The maximum particle size of the soil dictates the minimum thickness of the test sample. A study by Brenner *et al.* (1997) found that the drained strength of fissured dense soil (residual basalt) from 500 × 500 mm and 290 mm high shear box samples was 1.5 to 3 times less than the strength from 36 mm diameter triaxial samples in the normal stress range of 50 to 350 kPa. With relatively uniform samples, the size of the shear box was found to be less significant.

### *Triaxial tests*

Triaxial test procedures play a large role in geotechnical testing programmes but have largely been derived for use on traditional sedimentary soils in temperate climates. The triaxial test is beneficial for obtaining a variety of test results such as triaxial strength, stiffness and characteristics of the stress ratios of soil specimens. The samples are either remoulded or from undisturbed samples trimmed and cut into cylindrical shape. Commonly used samples have a height/diameter ratio of 2:1. Most common sizes used are 76 mm × 38 mm and 100 mm × 50 mm. Samples will be sealed in a thin rubber membrane and subjected to fluid pressure. Axial load is then applied through a piston acting on the top cap and controlling the deviator stress.

Types of triaxial tests conducted in the laboratory are unconsolidated undrained (UU) test with or without pore pressure measurement, isotropically consolidated undrained compression (CIU) test with or without pore pressure measurement, and isotropically consolidated drained compression (CID) test. Unconfined compression (UCS) test is also an accepted method to test the strength of the more robust tropically weathered materials, from hard soils to strong rocks. Good samples recovered from high quality drilling techniques are particularly adaptable to this method provided steps are taken to preserve the in-situ moisture condition.

Since a variety of conditions of residual soil exist in reality, especially the partially saturated condition, an erratic result will be given if common methods of triaxial test are carried out. Some of the routine triaxial tests require the sample to be fully saturated to present a saturated condition but this is rather controversial for residual soils. Application to partially saturated fabric-influenced materials in climatic environments that impose rapid changes in moisture condition can cause difficulties both in establishing relevant test procedures and in the modelling of site conditions. The standard procedure of imposing saturation on under-saturated materials appears difficult to justify on the grounds of modelling site conditions. As reviewed by Brand and Phillipson (1985), pre-saturation which normally needs high back pressure is actually severe compared to the actual conditions. This technique can only be applied to achieve consistent effective strength parameters. However, Blight (1985) stated that the saturation condition can be applied considering that in unusually wet weather years the water table can rise for several meters. Saturation therefore represents the least favourable condition of the residual soil.

A soil that is partly saturated consists of a three phase system: gas (including air and water vapour), water and solid particles. Analysis of partial saturation is complex. The determination of effective stresses in partly saturated soils requires measurement of air pressures as well as pore water pressure. The following extended Mohr-Coulomb equation has been proposed for the solution of partial saturation problems (Fredlund, 1987):

$$\tau = c' + (\sigma_n - u_a)\tan\sigma_n + (u_a - u_w)\tan\phi^b \tag{3.1}$$

where $\tau$ = shear strength; $c'$ = effective cohesion, $\sigma_n$ = normal stress; $\tan\sigma_n$ = effective friction angle; $\phi^b$ = angle of shear strength change with a change in matric suction; $u_a$ = pore air pressure; $u_w$ = pore water pressure.

Undisturbed triaxial testing of suitable samples can be of practical use in a tropical residual soil environment although the resulting parameter must be interpreted in the light of field data and may in some cases serve only as a back up to empirical established figures (Geoguide 3, 1996). The following comments apply to the general use of triaxial testing of tropical residual soils:

a. Multistage triaxial testing is not recommended for tropical residual soils especially in those with an unstable fabric liable to collapse, brittle soils and those that show strain-softening characteristics. Multi-stage tests might give misleading strength values for design.
b. Quick undrained tests are not suitable for unsaturated materials. However, the test is appropriate for partially saturated soils.
c. Special procedures are likely to be required for high void ratio or bonded materials, e.g. low confining pressure, slow loading rate.
d. Significant numbers of slope failures in tropical environments are shallow in nature and analysis of these would require parameters derived at appropriate (low) confining pressures.
e. For undrained tests with pore pressure measurements, the rate of deformation must be slow enough to allow the non-uniform pore pressure to equalise, and in drained tests complete drainage condition must be achieved.

Sample size for triaxial testing in tropical residual soils should be about 75 mm in diameter. The common test specimen 38 mm in diameter is not applicable due to the disturbance caused in extruding or trimming small diameter specimens from borehole samples. Brand and Phillipson (1985) stated that a specimen of 100 mm is commonly used in Australia, Brazil, Germany, Hong Kong and UK. Samples with smaller diameters are not considered representative, because of the scale effect relating to fissures and joints in the soil (Brenner *et al.*, 1997). In addition, the sample diameter should not be less than eight times the maximum particle size. The ratio of sample length to diameter must be at least 2 to 1.

### *Permeability tests*

Permeability of undisturbed samples can be derived from data obtained from consolidation tests; either triaxial, standard oedometer or Rowe cell. It may also be obtained from specific procedures using the permeameter equipment. Applications of the value of permeability are for drainage, analysing influence of seepage on slope stability, consolidation analysis and design of foundations for dams and excavations. Extrapolation to mass in-situ permeabilities from laboratory-derived material permeameters for tropical residual soils should be viewed with extreme caution. Despite conducting laboratory permeability tests, in-situ tests are more likely to represent the actual condition of residual soil (Brand and Phillipson, 1985). Field permeability will consider the relict joint and other preferential drainage paths that are most likely not to be identified in laboratory testing. Field permeability will be described in further detail in the following section of this chapter.

The laboratory permeability test for residual soils is observed to be best performed under back pressure in a triaxial cell, but should have a diameter of more than 75 mm and be 75 mm high as reviewed by Brand and Phillipson (1985).

A laboratory permeability test for residual soil can be conducted on compacted soil and more uniformly structured mature residual soils, particularly when the coefficient of permeability is determined in both directions, for vertical and horizontal trimmed samples. The test will then result in an estimate of the mass permeability of uniformly textured soils. Other advantages of using laboratory tests are for indications of the variation in the coefficient of permeability with changes in effective stress. Results obtained from the test can be used in designing earthworks.

#### *Pore water pressure and suction test*

Many soil-rock profiles of tropical residual soils, particularly on slopes, are known to be in an unsaturated condition. The stability of slope is a major concern in most tropical residual soil countries due to the frequent periods of heavy rainfall. Gasmo *et al.* (1999) stated that rainfall has a detrimental effect on the stability of residual soil slopes.

Negative pore pressures or matric suction is found to play an important role in the stability of slopes. These suctions have an important bearing on water entry, structural stability, stiffness, shear strength and volume change. The additional shear strength that exists in unsaturated soils due to negative pore-water pressures is lost as a result of rainwater infiltration into the soil. As a result, their in-situ geotechnical performance is likely to be influenced by variations in soil suction, in response to rainfall infiltration. Another study by Richards (1985) found that fine grained residual soils have high solute contents which also affect the physical properties of the soil, and the total soil suction, due to the solute component. The total soil suction, the water content and the solute content and how they vary with time are often the most important variables in soil engineering design.

In-situ soil suction can be measured with suitable sophisticated methods and equipment such as various types of tensiometers. The installation of a tensiometer is also incorporated with the use of piezometers to measure groundwater level, and a rain gauge to estimate rainfall intensities on the slope. A tensiometer generally comprises a water-filled plastic tube with a high air entry ceramic cup sealed at one end and a vacuum pressure gauge and a jet-fill cup sealed at the other end. When installed in soil, the pore-water pressure in the soil equilibrates with the water pressure in the tube and the pressure is measured by the vacuum pressure gauge or by a pressure transducer. The jet-fill cup is used as a reservoir to allow for easy refilling and de-airing of the tensiometer. Details of field suction measurements are included in Chapter 4 of this Handbook.

Another alternative is to measure suction indirectly in the laboratory by means of the filter paper method. This method involves placing Whatman's No. 42 filter paper in contact with the soil for a period of 7 days and measuring the amount of moisture taken up by the paper (*Wfp*). Matrix suctions may be arrived at by the following empirical relationships: (Chandler *et al.*, 1992).

$$\text{Suction (kPa)} = 10^{(4.84-0.0622\,Wfp)}; \quad \text{for } Wfp < 47\% \tag{3.2}$$

$$\text{Suction (kPa)} = 10^{(6.05-2.48\log Wfp)}; \quad \text{for } Wfp > 47\% \tag{3.3}$$

Further descriptions on tests for soil in unsaturated conditions are included in the following section of the Handbook.

### *Compressibility and consolidation*

There are a number of methods, both in-situ and laboratory, that can be used to determine the compressibility of tropical residual soils. The oedometer and triaxial compression tests are the main laboratory methods of testing, while for in-situ testing the standard penetration test, pressure meter test and plate loading test have been used. These field tests are described later in this chapter.

The oedometer is a one-dimensional consolidation test where complete lateral confinement is used to determine total compression of fine-grained soil under an applied load. The test is also used to determine the time rate of compression caused by a gradual volume decrease that accompanies the squeezing of pore water from the soil. An undisturbed sample is usually used for the consolidation test to obtain high quality results. The test first requires samples representative of principal compressible strata. Some two to eight tests should be conducted depending on the complexity of conditions (Brand and Phillipson, 1985).

The oedometer test is accepted for direct application for full analysis of amounts and rate of settlement only for intact clays. Oedometer tests are not suitable for measuring the compressibility of coarse grained soil and are thus not advisable for testing predominantly coarse grained residual soils. Whenever the consolidation or compression characteristic is needed, the use of triaxial test data is applicable. Brand and Phillipson (1985) noted that residual soils support many high-rise buildings and the foundations are usually taken down to below the residual soils, so that consolidation tests are regularly used. Trimming the residual soil samples is truly challenging because of the gravel content in some soils. There are various consolidation tests that have been carried out and the recognised features are described as follows:

- An oedometer can be used to assess the swelling characteristics of residual soil.
- Suction-controlled consolidometers can be used to control stress, soil suction and equilibrium electrolyte solution.

Geoguide 3 (1996) suggests the use of the more adaptable Rowe Cell which can accommodate larger samples. This is due to substantial evidence that larger, good quality undisturbed samples provide a better model of in-situ behaviour. Rowe cells also provide much greater versatility in terms of drainage conditions.

## 3.4 IN-SITU TESTS

Some of the most common in-situ tests available for routine investigation are listed in Table 3.8 (Schnaid *et al.*, 2004a; 2004b). In-situ testing equipment and testing procedures have spread worldwide for a wide range of geotechnical applications, and engineers can rely on a variety of commercial tools, Standards, guidelines and specifications supported by International Reference Test Procedures and well established national codes of practice.

The in-situ test techniques listed in Table 3.8 are all practical for the investigation of tropical residual soils, providing the necessary information for determining the sub-surface stratigraphy and for assessing the geotechnical parameters. Since most interpretation methods for estimating soil parameters are related to either sand (fully

*Table 3.8* Commercial in-situ testing techniques (*After* Schnaid *et al.*, 2004a).

| *Test* | *Designation* | *Measurements* | *Common Applications* |
|---|---|---|---|
| Geophysical Tests: | | | |
| Seismic refraction | SR | P-waves from surface | Ground characterisation |
| Surface waves | SASW | R-waves from surface | Small strain stiffness, $G_o$ |
| Crosshole test | CHT | P & S waves in boreholes | |
| Downhole test | DHT | P & S waves with depth | |
| Standard Penetration Test | SPT | Penetration (*N* value) | Soil profiling<br>Internal friction angle, $\phi'$ |
| Cone Penetration Test | | | |
| Electric | CPT | $q_c, f_s$ | Soil profiling |
| Piezocone | CPTU | $q_c, f_s, u$ | Undrained shear strength, $s_u$ |
| Seismic | SCPT | $q_c, f_s, V_p, V_s, (+u)$ | Relative density/friction angle, $\phi'$ |
| Resistivity | RCPT | $q_c, f_s, \rho$ | Consolidation properties<br>Stiffness (seismic cone) |
| Pressuremeter Test | | | |
| Pre-bored | PMT | $G$, $(\psi \times \varepsilon)$ curve | Shear modulus, $G$ |
| Self-boring | SBPMT | $G$, $(\psi \times \varepsilon)$ curve | Undrained shear strength, $s_u$ |
| Push-in | PIPPMT | $G$, $(\psi \times \varepsilon)$ curve | Internal friction angle, $\phi'$ |
| Full-displacement | FDPMT | $G$, $(\psi \times \varepsilon)$ curve | In-situ horizontal stress<br>Consolidation properties |
| Flat Dilatometer Test | | | Stiffness |
| Pneumatic | DMT | $p_o, p_1$ | Shear strength |
| Seismic | SDMT | $p_o, p_1, V_p, V_s$ | |
| Vane Shear Test | VST | Torque | Undrained shear strength, $s_u$ |
| Plate loading test | PLT | $(L \times \delta)$ curve | Stiffness and strength |
| Combined Test | | | Shear modulus, $G$ |
| Cone pressuremeter | CPMT | $q_c, f_s, (+u), G, (\psi \times \varepsilon)$ | Shear strength |

drained conditions) or clay (fully undrained conditions), this chapter attempts to extend this existing background by summarizing the available experience on residual soils where the bonded structure and the cohesive-frictional nature of the soil has to be accounted for. It is here worth recalling that although most geomaterials are recognised as being 'structured', the natural structure of residual soils has a dominant effect on their mechanical response (Vaughan, 1985; 1997). In structured soils exhibiting a cohesive-frictional behaviour, a limited ground investigation based on penetration tests (CPT, CPTU or SPT) will not produce the necessary database for any rational assessment of soil properties, for the simple reason that two strength parameters ($\phi'$, $c'$) cannot be derived from a single measurement ($q_c$ or $N_{60}$). In practice, however, limited investigations are often the preferred option. In such cases involving cohesive frictional soil, engineers tend to (conservatively) ignore the $c'$ component of strength and correlate the in-situ test parameters with the internal friction angle $\phi'$. Average $c'$ values may be later assessed from previous experience and backanalyses of field performance. More consistent approaches for characterizing the mechanical properties of cohesive-frictional materials are given by pressuremeter and plate load tests, despite the fact that analyses are complicated by a number of factors such as the influence of bonding on the stress-dilatancy response of soils and the effects of destructuration. These aspects are explored here and alternatives for testing interpretation are given.

### *Seismic tests*

A geophysical survey is regarded as a powerful technique for subsurface exploration. Tests are generally non-destructive in nature and can be performed from the ground surface. Despite due recognition of its risks and limitations, there has been a steady increase in the perceived value of geophysics in representing complicated subsurface conditions involving large spatial variability and stratified soils. In addition, cross and downhole methods have been extensively used in geotechnical engineering, including the adaptation of sensors in the seismic cone.

The theoretical bases upon which seismic and other geophysical measurements are found are not within the scope of this chapter. For that purpose, there are a number of reference textbooks that extensively cover this subject area such as Richard *et al.* (1970), Sharma (1997) and Santamarina *et al.* (2001). For us, it is important to recall that geophysical methods rely on a significant contrast in physical properties of materials under investigation. Intrinsic properties such as density, resistivity or electrical conductivity, magnetic susceptibility and velocity of shock waves of the subsurface materials should be considered when evaluating the suitability of a given technique. Frequently used geophysical techniques are seismic refraction, high resolution surface wave reflection, vibration, down-hole and cross-hole, electrical resistivity, magnetic and gravity tests (e.g. Santamarina *et al.*, 2001; Becker, 2001; Stokoe *et al.*, 2004).

The primary applications in the use of geophysical methods in geotechnical engineering are (Becker, 2001): to map stratigraphy, determine thickness of strata, depth of bedrock and define major anomalies such as channels and cavities; to locate deposits of aggregates and other construction materials; and to determine engineering properties of strata and their spatial variation. Geo-environmental projects complement the list of applications. It is always necessary to bear in mind that geophysical techniques are intended to supplement ground investigation methods. To enhance its consistency, a site investigation campaign should always encompass a combination of geophysical surveys with a mesh of boreholes and/or penetration tests.

In cross-hole (CHT) and down-hole (DHT) tests, attention is given to the measurement of shear wave velocities from which it is possible to obtain the small-strain stiffness of the soil at induced strain levels of less than 0.001%:

$$G_o = \rho V_s^2 \tag{3.4}$$

where, $G_o$ is the shear modulus, $\rho$ the mass density and $V_s$ the velocity of shear waves for a linear, elastic, isotropic medium. The CHT and DHT enable the velocity of horizontally propagating, vertically polarised ($S_{hv}$), vertically propagating, horizontally polarised ($S_{vh}$) and horizontally propagating, horizontally polarised ($S_{hh}$) shear waves to be measured.

### *Standard Penetration Test (SPT)*

The SPT is the most widely used in-situ testing technique, primarily because of its simplicity, robustness and its ability to cope with difficult ground conditions in addition to providing disturbed soil samples. In many residual deposits the SPT, combined with continuous rotational coring, is the only investigation tool that is able to explore the stiff and hard layers of residual deposits.

Comprehensive reviews of the procedures and applications of the SPT are given by Decourt *et al.* (1988) and Clayton (1993). There is a range of types of SPT apparatus in use around the world (e.g. those employing manual and automatic trip hammers) and, consequently, variable energy losses cannot be avoided. Variability due to unknown values of energy delivered to the SPT rod system can now be accounted for properly by standardizing the measured N value to a reference value of 60% of the potential energy of the SPT hammer ($N_{60}$), as suggested by Skempton (1986). In many countries, however, this recommendation has not been incorporated into engineering practice. Moreover, even an SPT N value normalised to a given reference energy is not 'standard' because of the presently contentious issue of the influence of the length of the rod string.

The energy transferred to the composition of SPT rods was recently investigated by Odebrecht *et al.* (2004) and Schnaid (2005). This study has prompted a number of recommendations outlined to interpret the test in a more rational way on the basis of wave propagation theory. Recommendations are summarised as follows:

a. The energy transferred to the rod and to the sampler due a hammer impact should be obtained through integration of equation (3.5), calculated by the F-V method, and known as the Enthru energy:

$$E = \int_0^\infty F(t)V(t)\,dt \tag{3.5}$$

with an upper limit of integration equal to infinity (1/10 s is practical but may require longer time intervals of integration (1/5 s) in soft soils or long composition of rods).

b. the sampler energy can be conveniently expressed as a function of nominal potential energy $E^*$, sampler final penetration and weight of both hammer and rods.

The influence of rod length produces two opposite effects: wave energy losses increase with increasing rod length and in a long composition of rods, the gain in potential energy from rod weight is significant and may partially compensate measured energy losses.

c. efficiency is accounted for by three coefficients $\eta_1$, $\eta_2$ and $\eta_3$ that should be obtained from calibration. The hammer efficiency $\eta_1$ is obtained from measurement at the top of the rod stem. Efficiency factor $\eta_2$ can be assumed as unit. The energy efficiency $\eta_3$ is negatively correlated to the length of rods.

The *maximum potential energy*, $PE^*$ delivered to the soil, should therefore be expressed as a function of the *nominal potential energy* $E^*$, and an additional energy related to the sampler penetration and the weight of both hammer and rods.

$$PE^* = E^* + (M_h + M_r)g\Delta\rho \tag{3.6}$$

where: $M_h$ = hammer weight;
$M_r$ = rod weight;
$g$ = gravity acceleration;
$\Delta\rho$ = sample penetration under one blow;
$E^*$ = nominal potential energy = 0.76 m 63.5 kg 9.801 m/s = 474 J

The *nominal potential energy* $E^*$ = 474 J (ASTM, 1986) represents a part of the hammer potential energy to be transmitted to the soil. An *additional hammer potential energy* is given by $M_h, g, \Delta\rho$ which cannot be disregarded for tests carried out at great depths in soft soils, i.e. conditions in which $\Delta\rho$ and $M_r$ are significant. For convenience, equation (3.6) can be written in two parts where the first represents the *hammer potential energy* (*nominal* + *additional*) and the second the *rod potential energy*:

$$PE^* = (0.76 + \Delta\rho)M_h g + \Delta\rho M_r g \tag{3.7}$$

Equation (3.7) deals with an ideal condition, where energy losses during the energy transference process are not taken into account. However, it is well known in engineering practice that these losses occur; they should not be disregarded and can be considered by efficiency coefficients. Equation (3.7) becomes:

$$PE_{h+r} = \eta_3[\eta_1(0.76 + \Delta\rho)M_h g + \eta_2\Delta\rho M_r g] \tag{3.8}$$

where:

$$\eta_1 = \text{hammer efficiency} = \frac{\int_0^\infty F(t)V(t)\,dt}{(0.76 + \Delta\rho)M_h g}$$

$$\eta_2 = \beta_2 + \alpha_2\ell \approx 1$$

$$\eta_3 = \text{energy efficiency} = 1 - 0.0042\ell$$

It follows from the foregoing that normalisation to a reference value of $N_{60}$ is no longer sufficient to fully explain the mechanism of energy transfer to the soil. Furthermore,

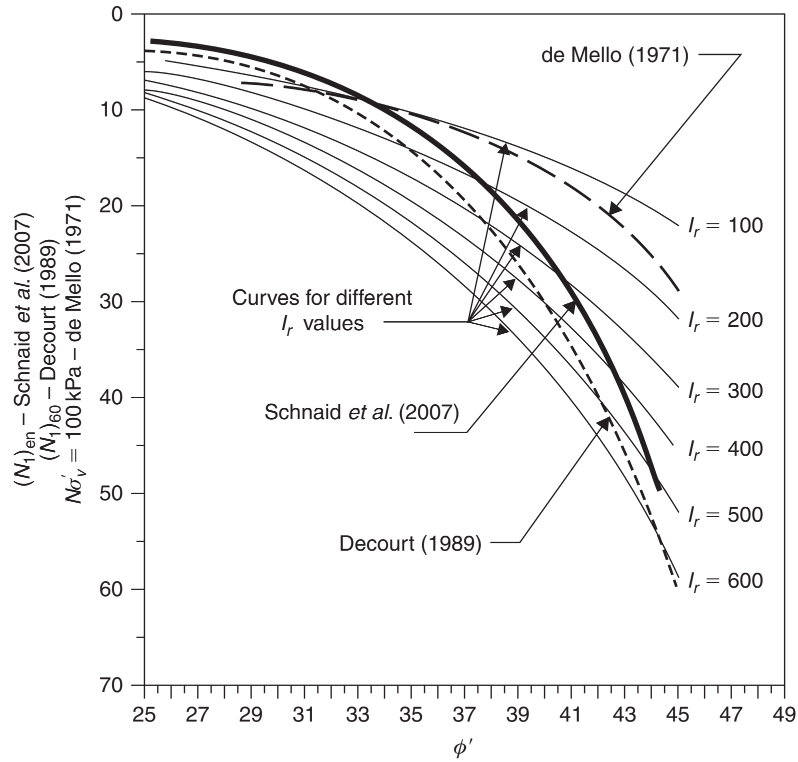


*Figure 3.2* Prediction of friction angle $\phi'$ from the SPT (*After* Schnaid *et al.*, 2009)

it interesting to recall that maximum potential energy can be transformed into work by the non-conservative forces ($W_{nc}$) acting on the sampler during penetration, and since the work is proportional to the measured permanent penetration of the sampler, it is possible to calculate the dynamic force transmitted to the soil during driving:

$$PE_{h+r} = W_{nc} = F_d \Delta\rho \quad \text{or} \quad F_d = PE_{h+r}/\Delta\rho \tag{3.9}$$

The dynamic force $F_d$ can be considered as a fundamental measurement for the prediction of soil parameters from SPT results.

The dynamic force applied to drive the sampler into the soil combined with bearing capacity and cavity expansion theories allows the internal friction angle $\phi'$ to be estimated directly from ordinary SPT sampler penetration (Schnaid *et al.*, 2009). The blow count number is defined as $(N_1)_{en}$ because the interpretation relies on correcting the measured value both to the energy transferred to the soil and the applied mean stress. The relationship between $(N_1)_{en}$ and $\phi'$ shown in Figure 3.2 is fairly sensitive to the variations in the rigidity index, an effect that is more significant for dense materials than for loose.

The general procedure to estimate $\phi'$ from bearing capacity formulation can be summarised as follows (Schnaid *et al.*, 2009):

a. record sampler penetration during driving;
b. from the average permanent penetration, calculate the driving penetration force $F_d$ from equation (3.9);
c. estimate soil parameters and stress level either from experience or existing geotechnical data, and adopt this set of input parameters to calculate the ultimate static load from bearing capacity formulation:

$$F_d \approx F_e = A_p(p'N_q + 0.5\gamma d N_\gamma) + A_f(\gamma L K_s \tan\delta) \tag{3.10}$$

where $N_q$ and $N_\gamma$ = bearing capacity factors, $A_p$ = area of sampler base, $A_f$ = area of sampler shaft, $d$ = sampler diameter, $L$ = test depth, $\gamma$ = unit weight of the soil, $p'$ = mean stress and $\delta$ = soil-pile interface friction angle.
d. Estimate the friction angle $\phi'$ from equation (3.10), adopting the values from steps b and c as input parameters.

Alternatively, there is a family of empirical correlations established directly from comparisons between drained triaxial compression tests and SPT field data. The expression derived by Hatanaka and Uchida (1996) yields the following expression where the $N$ values are corrected to an energy efficiency of 60(%):

$$\phi' \approx 20^\circ + \sqrt{15.4(N_1)_{60}} \tag{3.11}$$

In all these correlations, the cohesion intercept is neglected and tests are interpreted as a cohesionless material. In addition, it is necessary to recognise that equations such as (3.11) have been validated on sedimentary sand deposits only. Application to an undisturbed sand database is presented in Figure 3.3, in which strength values quoted from these equations are shown to yield values that would not be unreasonable for granular materials. Its applicability for residual soils might depend on the degree of cementation of the deposit and may have to be locally validated.

The degree of cementation can be accessed from the ratio of penetration resistance and soil stiffness, since in principle, a material that is stiffer in deformation may be stronger in strength (e.g. Tatsuoka and Shybuya, 1991). Based on this concept, a methodology has been developed to identify the existence of distinctive behaviour emerging from ageing or cementation of residual soils based on the ratio of the elastic stiffness to ultimate strength, $G_o/N_{60}$. A guideline formulation to compute $G_o$ from SPT tests is given by the following equations:

$$\frac{(G_0/p_a)}{N_{60}} = \alpha N_{60}\sqrt{\frac{p_a}{\sigma'_{vo}}} \quad \text{or} \quad \frac{(G_0/p_a)}{N_{60}} = \alpha (N_1)_{60} \tag{3.12}$$

where:

$$(N_1)_{60} = N_{60}\left(\frac{p_a}{\sigma'_{vo}}\right)^{0.5}$$

where $\sigma'_{vo}$ is the mean vertical effective stress and $p_a$ the atmospheric pressure. The $\alpha$ value is a dimensionless number that depends on the level of cementation and age as

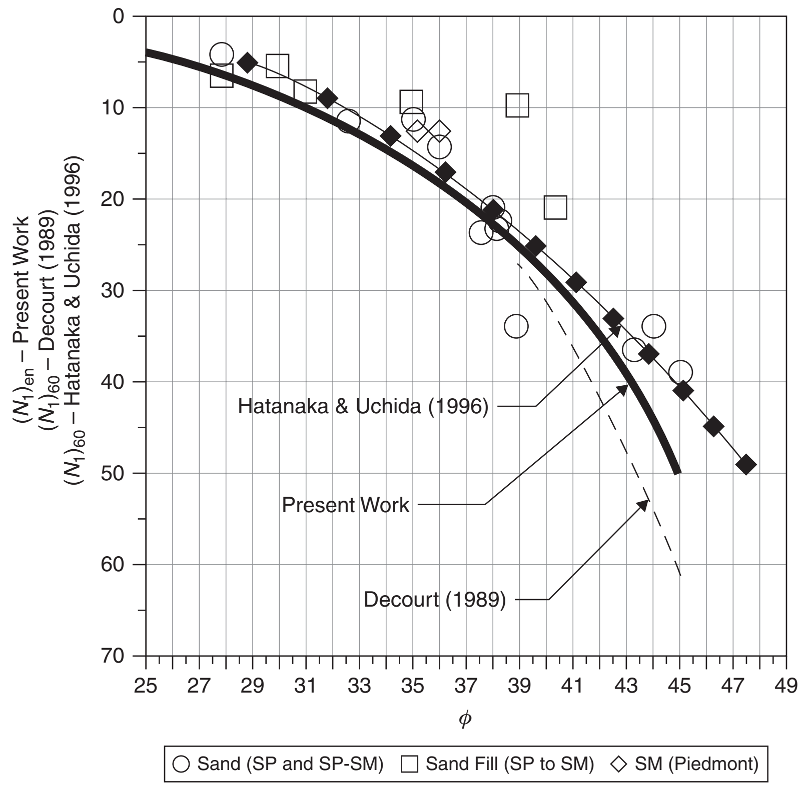


*Figure 3.3* Friction angle of cohesionless soils from SPT resistance (*After* Schnaid *et al.*, 2009; data from Hatanaka and Uchida, 1996)

well as the soil compressibility and suction. The small strain stiffness to strength ratio embodied within the $G_o/N_{60}$ term is seen on Figure 3.4, at a given $(N_1)_{60}$ (or relative density), to be generally appreciably higher for lateritic soils than that of the saprolites, primarily because the latter generally exhibit higher $N_{60}$ (or strength) values.

The database comprises soils from Brazil and Portugal (Barros, 1998; Schnaid, 1997; Schnaid *et al.*, 2004a). The bond structure is seen to have a marked effect on the behaviour of residual soils, producing values of normalised stiffness ($G_o/N_{60}$) that are considerably higher than those observed in fresh cohesionless materials.

This database can be also useful for assessing the stiffness of natural deposits. Considering the variation observed in tropical residual soils, it is preferable to express correlations in terms of lower and upper boundaries designed to match the range of recorded $G_o$ values (Schnaid *et al.*, 2004a):

$$\left.\begin{array}{ll} G_o = 1200\sqrt[3]{N_{60}\sigma'_{vo}p_a^2} & \text{upper bound: cemented} \\ G_o = 450\sqrt[3]{N_{60}\sigma'_{vo}p_a^2} & \text{lower bound: cemented} \\ & \text{upper bound: uncemented} \\ G_o = 200\sqrt[3]{N_{60}\sigma'_{vo}p_a^2} & \text{lower bound uncemented} \end{array}\right\} \tag{3.13}$$

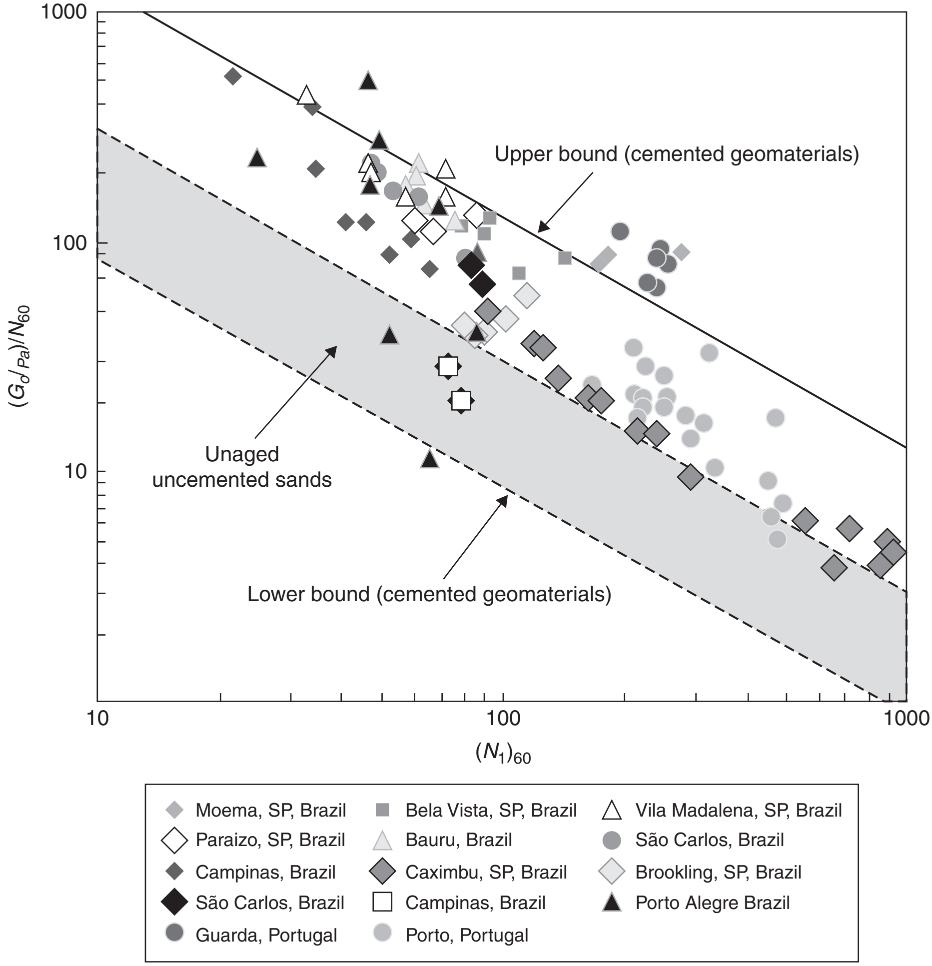


*Figure 3.4* $G_o/N_{60}$ *vs* $(N_1)_{60}$ space to identify cementation on residual soils (*After* Schnaid, 2005)

It is here important to emphasise that given the considerable scatter observed for different soils, correlations such as given in equation (3.13) are only approximate indicators of $G_o$ and do not replace the need for in-situ shear wave velocity measurements. The reduction in the ratio of $G/G_o$ with shear stress and shear strain is known to be sensitive to degradation of cementation and structure, among several other factors (e.g. Tatsuoka *et al.*, 1997). The moduli degradation can be measured in the laboratory with high resolution sensors provided that high-quality undisturbed samples can be obtained.

*Piezocone penetration test (CPTU)*

The CPT, with the possible inclusion of pore water pressure, shear wave velocity and resistivity measurements, is recognised worldwide as an established, routine and

*Figure 3.5* Range of CPT tools available for site investigation (courtesy of A.P. van den Berg)

cost-effective tool for site characterisation and stratigraphic profiling, and a means by which the mechanical properties of the subsurface strata may by assessed. CPTs were particularly popular in sands and in marine and lacustrine sediments in coastal regions, but are now also commonly used in residual soils provided that penetration is achieved. For a general review on the subject, the reader is encouraged to refer to Lunne *et al.* (1997) – *CPT in Geotechnical Practice*, and the *Proceedings of the Symposia on Penetration Testing* (1981, 1988, 1995, 1998, 2004).

Routine penetrometers have employed either one midface element for pore water pressure measurement (designated as $u_1$) or an element positioned just behind the cone tip (shoulder, $u_2$). The ability to measure pore pressure during penetration greatly enhances the profiling capability of the CPTU, but in unsaturated soil conditions, this measurement is of very little application. Geotechnical site characterisation can be further improved by independent seismic measurements, adding the downhole shear wave velocity ($V_s$) to the measured tip cone resistance ($q_t$), sleeve friction ($f_s$) and pore water pressure ($u$). The combination of different measurements into a single sounding provides a particularly powerful means of assessing the characteristics of unusual materials (Schnaid, 2005). Various penetrometers are available, for example a miniature electric element with a cone tip area of 2 cm$^2$, a 5 cm$^2$ minicone, a standard 10 cm$^2$ piezocone, a 10 cm$^2$ pore-water sensors piezocone, a seismic piezocone and a resistivity cone, some of which are shown in Figure 3.5.

Evaluation of the peak friction angle of cohesionless soils from CPT can be based on analytical, numerical or empirical approaches (e.g. Yu and Mitchell, 1998; Yu, 2004). There are a number of correlations often adopted in engineering practice such as (Lunne *et al.*, 1997):

$$\phi'_p = \arctan\left[0.1 + 0.38\log\left(\frac{q_t}{\sigma'_{vo}}\right)\right] \tag{3.14}$$

More recently, an expression was derived by Mayne (2006) where the cone is normalised by the square root of the effective overburden stress following recommendations from Jamiolkowski *et al.* (1985):

$$\phi'_p = 17.6^\circ + 11.0 \log\left[\frac{\left(q_t/\sigma_{atm}\right)}{\left(\sigma'_{vo}/\sigma_{atm}\right)^{0.5}}\right] \tag{3.15}$$

While these correlations appear to work well for clean sands, there is very little systematic experience in intermixed and bonded soils and local validation is therefore recommended.

In bonded soils the contribution of the cohesive component linking particles is disregarded in equations (3.14) and (3.15), which may impact predictions of shear strength. The important feature of characterizing the presence of the bonding structure can be achieved by combination of strength and stiffness measurements. It follows from the foregoing on the SPT that a bonded/cemented structure produces $G_o/q_c$ and $G_o/N_{60}$ ratios that are systematically higher than those measured in cohesionless soils. These ratios provide a useful means of assisting site characterisation. Typical results from residual profiles are presented in Figure 3.6, in which the $G_o/q_c$ ratios are plotted against the normalised parameter $q_{c1}$ for CPT data (*After* Schnaid, 1999; Schnaid *et al.*, 2004a), where:

$$q_{c1} = \left(\frac{q_c}{p_a}\right)\sqrt{\frac{p_a}{\sigma'_v}} \tag{3.16}$$

$p_a$ being the atmospheric pressure. The bond structure generates normalised stiffness values that are considerably higher than those for uncemented soils and as a result, the datapoints for residual soils fall outside and above the band proposed for sands by Eslaamizaad and Robertson (1997) and theoretically determined by Schnaid and Yu (2007).

The upper and lower bounds designed to match the range of recorded $G_o$ values in residual soils can be used for a direct assessment of the variation of $G_o$ with $q_c$ (Schnaid *et al.*, 2004a):

$$\left.\begin{array}{ll} G_o = 800\sqrt[3]{q_c\sigma'_v p_a} & \text{upper bound: cemented} \\ G_o = 280\sqrt[3]{q_c\sigma'_v p_a} & \text{lower bound: cemented} \\ & \text{upper bound: uncemented} \\ G_o = 110\sqrt[3]{q_c\sigma'_v p_a} & \text{lower bound uncemented} \end{array}\right\} \tag{3.17}$$

Since considerable scatter is observed in the residual soils database, correlations such as given in equation (3.17) are only approximate indicators of $G_o$ and do not replace the need for in-situ shear wave velocity measurements.

The magnitude of the small strain stiffness in bonded soils is better understood in comparison with values determined from natural sands. A reference equation adopted in the comparison is:

$$\frac{G_o(\text{MPa})}{F(e)} = S[p'(\text{MPa})]^n \tag{3.18}$$

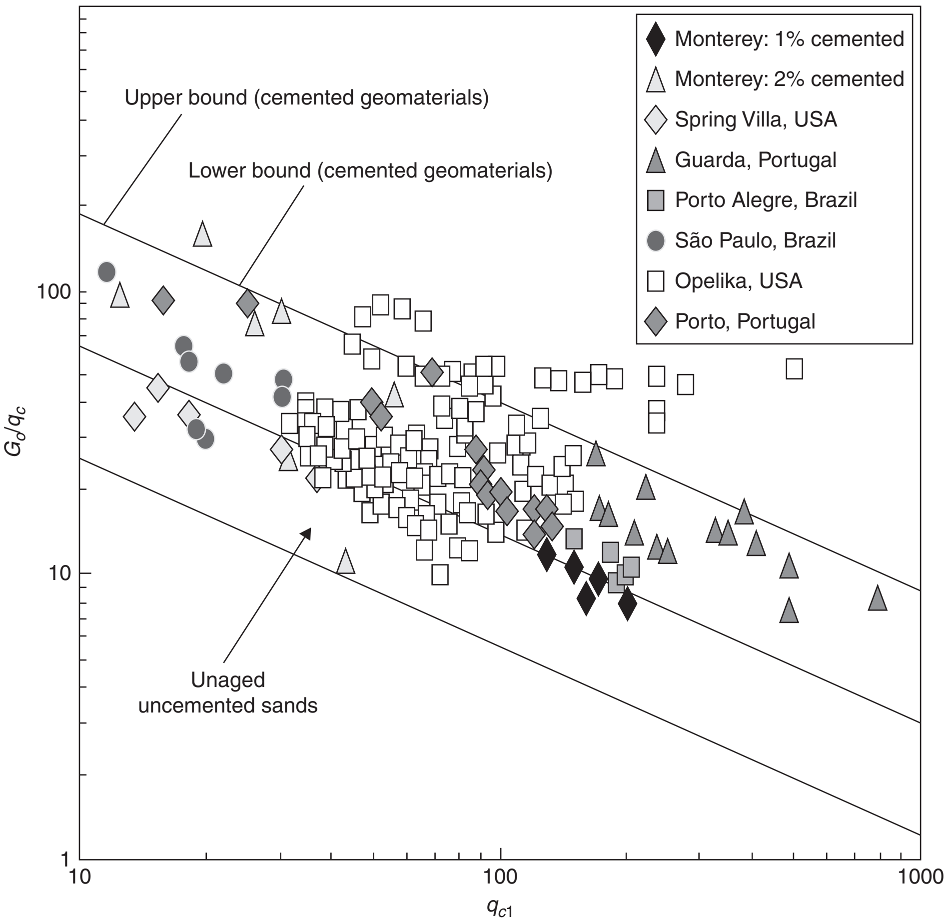


*Figure 3.6* $G_o/q_c$ ratios plotted against normalised parameter $q_{c1}$ for CPT data (Schnaid, 2005)

with a void ratio function:

$$F(e) = \frac{(2.17 - e)^2}{1 + e} \tag{3.19}$$

Values of parameters $S$ and $n$ are given in Table 3.9 and a direct comparison is shown in Figure 3.7, having the data for alluvial sands from Ishihara (1982) as reference. Values of $G_o$ diverge significantly from those established for transported soils when they exhibit the same granulometry but are uncemented. Parameter $S$ is much higher than the value adopted for cohesionless soils, whereas $n$ varies significantly as a result of local weathering conditions (e.g. Gomes *et al.*, 2004). Given the variations in both $S$ and $n$, the need for site-specific correlations is demonstrated.

*Table 3.9* Stiffness coefficients (*After* Schnaid, 2005)

| *Soil* | *S* | *n* | *Reference* |
|---|---|---|---|
| Alluvial sands | 7.9 to 14.3 | 0.40 | Ishihara (1982) |
| Porto saprolite granite | 65 to 110 | 0.02 to 0.07 | Viana da Fonseca (1996) |
| Guarda saprolite granite | 35 to 60 | 0.30 to 0.35 | Rodrigues and Lemos (2004) |

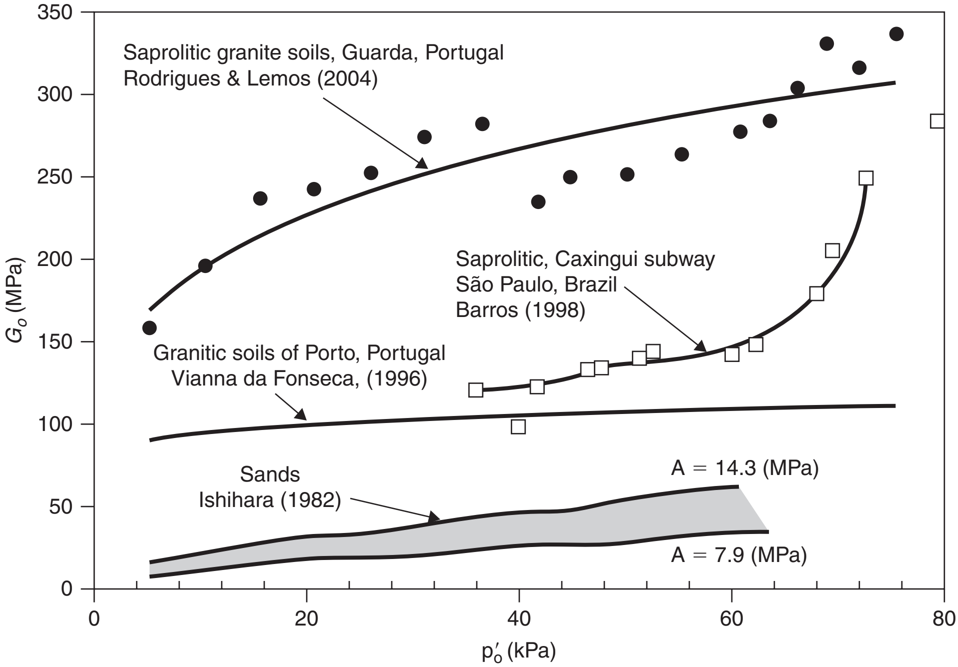


*Figure 3.7* Relation between $G_o$ and $p'_o$ for residual soils (*After* Schnaid, 2005; modified from Gomes *et al.*, 2004)

*Pressuremeter*

Pressuremeters are cylindrical devices designed to apply uniform pressure to the wall of a borehole by means of a flexible membrane. Both pressure and deformation at the cavity wall are recorded and interpretation is provided by cavity expansion theories under the assumption that the probe is expanded in a linear, isotropic, elastic, perfectly plastic soil. Under this assumption, the soil surrounding the probe is subjected to pure shear only. Acknowledging that the greatest potential of the pressuremeter lies in the measurement of modulus, it is a common practice to carry out a few unloading-reloading cycles during the test. If the soil is perfectly elastic in unloading, then the unloading-reloading cycle will have a gradient of $2G_{ur}$, where $G_{ur}$ is the unload-reload shear modulus. Numerous papers have been published on this theme and there are important textbooks such as Baguelin *et al.* (1978), Mair and Wood (1987), Briaud (1992), Clarke (1993) and Yu (2000).

*Figure 3.8* Cambridge self-boring pressuremeter (courtesy of Cambridge Insitu)

Pressuremeters are generally classified in three groups according to the method of installation into the ground. Pre-bored pressuremeter, self-boring pressuremeter and push-in pressuremeter are the three broad categories. The Menard pressuremeter is the most well known example of a pre-bored probe in which the device is lowered into a pre-formed hole. In a self-boring probe the device bores its own way into the ground with minimal disturbance (Figure 3.8), whereas in a push-in device the pressuremeter is pushed into the ground attached to a cone tip. The method of interpretation should take account of the installation process. Theoretical interpretation methods developed for pressuremeters involve axially symmetric expansion and contraction of an infinitely long cylindrical cavity. Under this fundamental assumption the cavity-expansion/contraction curve can be analytically modelled to obtain soil properties. The symmetry of the well defined boundary conditions of a pressuremeter is the main advantage of this technique over other in-situ tests.

The pressuremeter offers the possibility of characterizing the mechanical properties of residual soils, particularly as a means of inspecting the accurateness of a given set of design parameters. All the theories for the interpretation of the pressuremeter in bonded soils make use of the in-situ horizontal stress, soil stiffness and strength parameters: angle of internal friction, angle of dilation and cohesion intercept (which reduces with destructuration at high shear strains). The pressure expansion curve represents therefore a combination of all these parameters that cannot be assessed independently. In residual soils, instead of attempting to derive a set of parameters from a single test, the pressuremeter should be viewed as a "trial" boundary value problem against which a theoretical pressure-expansion curve, predicted using a set of independently measured parameters, is compared to a field pressuremeter tests. A good comparison between the measured and predicted curves gives reassurance to the process of selecting design parameters, whereas a poor comparison indicates that one or more of the constitutive parameters are unrealistic.

The solution for cavity expansion and contraction problems is extensive, and hence only a brief review is presented here to illustrate potential applications. Adequacy of interpretation methods largely depends upon the constitutive model adopted to represent soil behaviour which, for a cohesive-frictional material, is complicated by a number of factors such as the influence of bonding on the stress-dilatancy response of soils and the effects of destructuration. Ideally the $c'$ and $\phi'$ should be coupled to stiffness, dilatancy and mean stress level, and for that reason the cylindrical cavity expansion analysis developed by Mantaras and Schnaid (2002) and Schnaid and Mantaras (2003) is recommended. In these solutions, the concept introduced by Rowe (1962) that plastic dilatancy is inhibited by the presence of soil bonding was used to describe the plastic components of the tangential and radial increments in an expanding cavity. The solution is formulated within the framework of non-associated plasticity in which the Euler Method is applied to solve simultaneously two differential equations that leads to the continuous variations of strains, stresses and volume changes produced by cavity expansion.

In cohesive-frictional soils the strength parameters cannot be extracted directly from the experimental curve. The method to assess parameters recognises that strength, stiffness and in-situ stresses interact to produce a particular pressuremeter expansion curve through a procedure that can be summarised as follows:

a. Record a field pressuremeter curve;
b. Compute values of stiffness and in-situ stresses directly from the average pressuremeter measurements of the three strain arms;
c. Input shear parameters should be from independent test data;
d. Calculate a pressure-expansion curve and compare to the experimental curve measured in-situ.
e. Re-evaluate the set of input parameters if a good comparison between measured and calculated data is not obtained and produce new comparisons.

By reproducing this process for tests carried out at several different depths, it is possible to estimate average constitutive parameters from a given deposit, bearing in mind that engineering judgment is required to avoid the selection of a set of *doubtful* parameter values that may produce a good fit to the data.

Case studies in the Hong Kong gneiss (Schnaid *et al.*, 2000), Porto granite (Mantaras, 2002) and São Paulo gneiss (Schnaid and Mantaras, 2003) have validated the application of the described methodology. Take the case study summarised by Schnaid and Mantaras (2004) which reports an extensive site investigation programme comprising laboratory triaxial tests, SPT and a large number of high quality pressuremeter tests in a residual gneiss soil profile. A typical example of the fit provided by the analytical solution is shown in Figure 3.9 and a summary of the shear strength data obtained from the interpretation of 15 such tests is presented in Figure 3.10, which also plots SPT $N_{60}$ and the pressuremeter limit pressures ($\psi_L$). The SBPM yielded $\phi'_{ps}$ from 27 to 31° with considerable data scatter but within the range measured from laboratory testing data. The curve fitting applied to the loading portion of the SBPM tests gave results which are rather consistent, being slightly above the assumed critical state values and compatible with laboratory data. The presence of mica at given locations has yielded a lower boundary for predicted $\phi'_{ps}$ values, compatible with evidence provided by $N$ values.

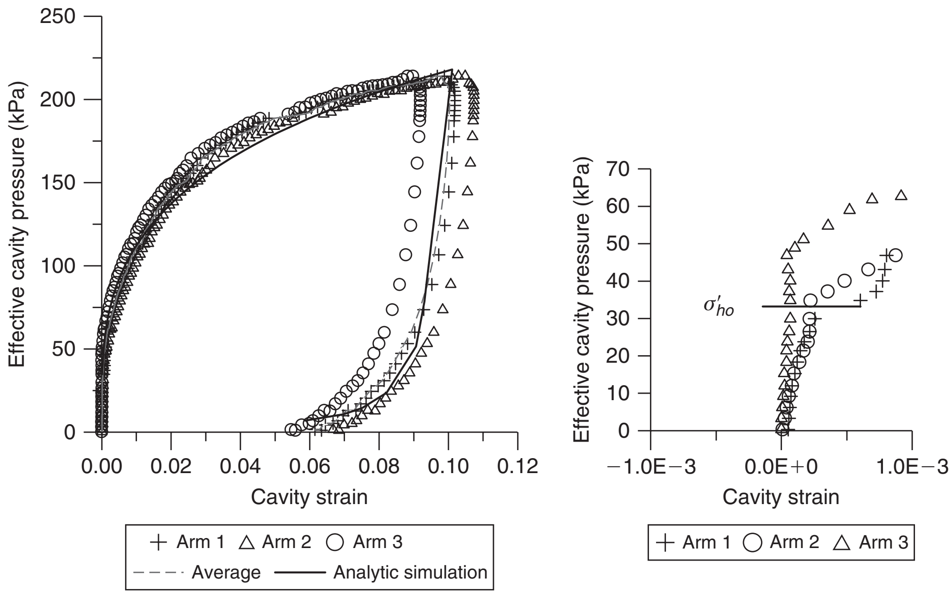


*Figure 3.9* Typical example of a pressuremeter test carried out in the saprolite gneiss residual soil of Sao Paulo (*After* Schnaid and Mantaras, 2004)

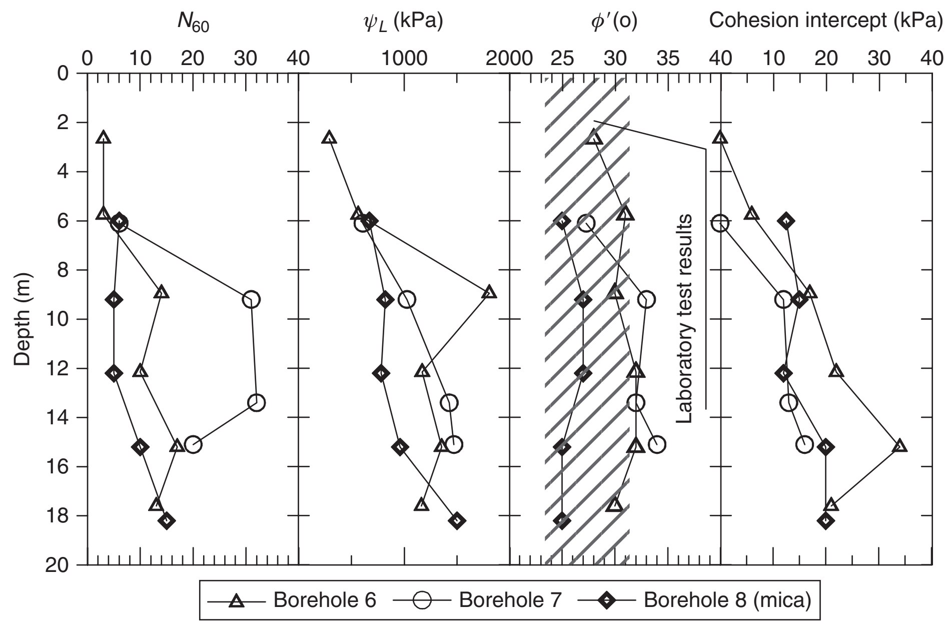


*Figure 3.10* Prediction of soil properties for the Sao Paulo gneiss residual soil (*After* Schnaid and Mantaras, 2004)

*Plate load test*

The plate load test (PLT) is a routine field method to determine the soil bearing pressure and stiffness for foundation design. In a standard procedure, the load is applied in uniform increments, with the magnitude of each load increment being kept constant until the rate of deflection is stabilised. Load-deflection points should be sufficient to produce an accurate load-deflection curve. The field test apparatus requires a loading device (truck, tractor, anchored frame, or other structure loaded with sufficient mass to produce the desired reaction on the surface under test) and a hydraulic jack assembly acting against a bearing circular steel plate. A deflection beam upon which dial gauges or LVTs are mounted and a load cell provide the necessary measuring system.

However, this standardised and straightforward procedure has not been systematically tested to determine the strength parameters in residual soils, a summary of existing experience being outlined by Barksdale and Blight (1997). Since the boundary conditions and the failure mechanism of plate tests in bonded soils are not easily modeled, rigorous interpretation of test data may require sophisticated numerical analysis with appropriate constitutive models. In more cemented geomaterials, a punching failure mechanism has been observed with tension cracks being developed around the bottom to the bearing plate (e.g. Thomé *et al.*, 2005). This distinct mechanism cannot be evaluated from conventional methods, and attempts to interpret the PLT as an inverse boundary value problem from which friction and cohesion are assessed have proved unrealistic.

On the other hand, plate-load tests are a common field method to estimate soil stiffness. Solution for footings resting on a homogeneous, isotropic, linear-elastic, semi-infinite half space is well known, with the Young Modulus $E$ being expressed as:

$$E = \frac{qB(1-\nu^2)}{\rho_o} I_f \tag{3.20}$$

where, $q$ is the pressure applied to the plate, $B$ is the width, $\rho_o$ is the corresponding settlement at the plate surface, $\nu$ is the Poisson's ratio, and $I_f$ the elastic influence factor (see Poulos and Davis, 1980). Considering the relative drained conditions of a load test in a residual soil profile and taking the Poisson's ratio as 0.2, the Young Modulus of a rigid footing would be approximately:

$$E = 0.75\frac{qB}{\rho_o} \tag{3.21}$$

Since assessment of an operational stiffness from plate tests has been the key element to the design of structures such as foundations and retaining walls, few attempts have been made to produce a direct relationship between this operational stiffness and N-SPT. Sandroni (1991) compiled a number of plate loading tests carried out in gneissic residual soils using the Theory of Elasticity to derive an operational Young modulus $E$ representative of ground movements under shallow foundations. Jones and Rust (1989) compiled data for a saprolitic weathered diabase. Despite the scatter, results are presented in Figure 3.11 and are used to support empirical correlations between $E$ and $N$ values such as (Barksdale and Blight, 1997):

$$E = a\mathrm{N}_{60}(\mathrm{MPa}) \tag{3.22}$$

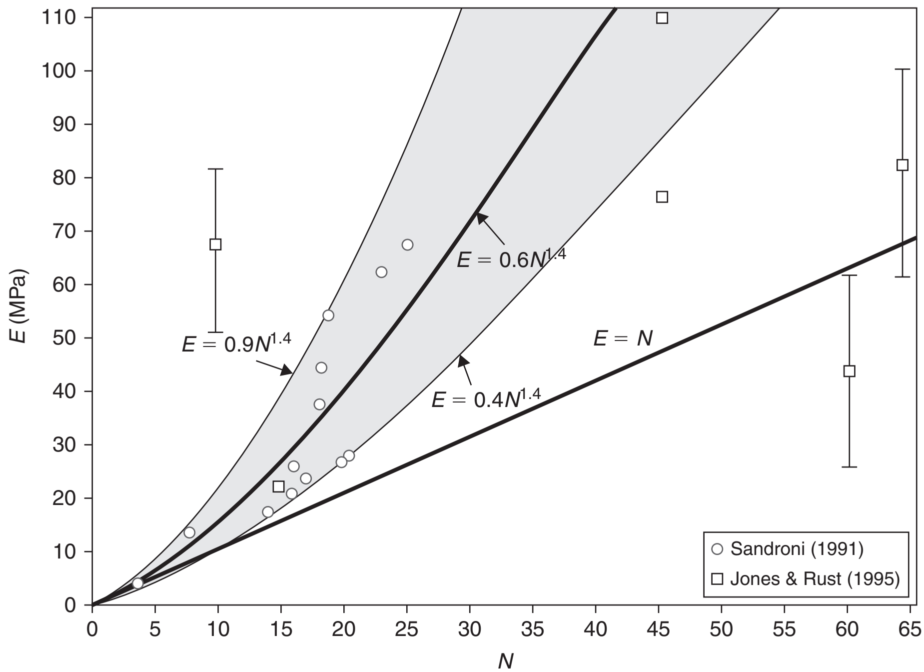


*Figure 3.11* Relationships between $N$ and Young modulus for residual soils $N_{72}$ for Sandroni (1991) and $N_{60}$ for Jones and Rust (1995) (*After* Schnaid, 2009)

with $a$ ranging from 1 to 1.6 and (Sandroni, 1991):

$$E = bN_{72}^{c} \text{ (MPa)} \tag{3.23}$$

with coefficients $b = 0.6$ and $c = 1.4$.

*Dilatometer*

The dilatometer consists of a stainless steel blade having a flat, circular steel membrane mounted flush on one side (Figure 3.12).

The test starts by driving the blade into the soil at a constant rate, generally between 10 and 20 mm/s. After penetration, the membrane is inflated through a control unit and a sequence of pressure readings are made at prescribed displacements, corresponding to (a) the $A$-pressure at which the membrane starts to expand ("lift-off") and (b) the $B$-pressure required to move the centre of the membrane by 1.1 mm against the soil. Values of $A$ and $B$ readings are used to determine the pressures $p_0$ and $p_1$ from the following expressions:

$$p_0 = 1.05(A - Z_m + \Delta A) - 0.05(B - Z_m - \Delta B) \tag{3.24}$$

$$p_1 = (B - Z_m - \Delta B) \tag{3.25}$$

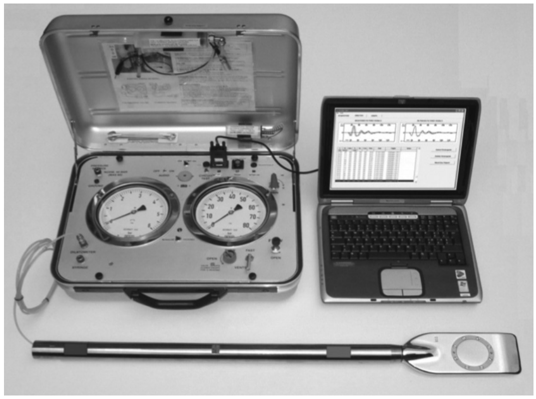

*Figure 3.12* The seismic dilatometer (courtesy of Marchetti)

where, $Z_m$ is the gauge zero offset when vented to atmospheric pressure and $\Delta A$ and $\Delta B$ calibration coefficients.

A general overview of the dilatometer test (DMT), guidelines for proper execution and basic interpretation methods are given by Marchetti *et al.* (2001) in a report issued under the auspices of ISSMGE Technical Committee TC16.

Interpretation methods are essentially based on correlations obtained by calibrating DMT pressure readings against high quality parameters (e.g. Lutenegger, 1988; Lunne *et al.*, 1989; Marchetti, 1997). These correlations are essentially empirically based, established against clay and sand material and are supported by a limited number of numerical studies (e.g. Baligh and Scott, 1975; Finno, 1993; Yu *et al.*, 1993; Smith and Houlsby, 1995; Yu, 2004).

There is no systematic experience of DMT in residual soils and it is not clear how the bonded structure of cohesive-frictional materials impacts parameters derived from the test. One potential application is the determination of the soil stiffness for settlement calculations. The dilatometer modulus is obtained by relating the displacement $s_0$ to $p_0$ and $p_1$ by the theory of elasticity (Gravesen, 1960). The solution assumes that (a) the space surrounding the dilatometer is formed by two elastic half-spaces in contact along the plane of symmetry of the blade and (b) zero settlement is computed externally to the loaded area:

$$s_0 = \frac{2D(p_1 - p_0)}{\pi} \frac{(1 - \nu^2)}{E} \tag{3.26}$$

where, $E$ is the Young modulus and $\nu$ the Poisson's ratio.

For a diameter membrane $D$ equal to 60 mm and a displacement $s_o$ equal to 1.1 mm, equation (3.26) approaches:

$$E_D = 34.7(p_1 - p_0) \tag{3.27}$$

Although $E_D$ is inherently an operational Young Modulus (both $E$ and $E_D$ are calculated from Elastic Theory), it is recognised that the expansion of the membrane from $p_0$ and $p_1$ reflects the disturbed soil properties around the blade produced by DMT penetration.

### *In-situ permeability*

Permeability of undisturbed samples can be obtained by in-situ testing (falling head method) at various depths as the drilling of the borehole proceeds. The falling head, rising-head and constant head tests can be used in boreholes and employing packers to isolate a particular hole length. It is important that the inside surface of the hole used for permeability testing is free of loose or smeared material which can make the results of testing imprecise.

Permeabilities of residual soils are affected by the variations in grain size, void ratio, mineralogy, degree of fissuring and the characteristics of the fissures. Garga and Blight (1987) explained that the permeability of some residual soils is strongly controlled by the relict structure of the material, where the flow takes place along relict joints, quartz veins, termite and other biochannels. A permeability test is needed when seepage problems are involved or expected.

There are two methods concerning permeability, which are feed water and extract water. General guidelines for in-situ permeability tests are described as follows:

- Ponding test or infiltration test can be conducted when the water table is low.
- Pumping test from test pits or holes can be used when the water table is near to surface.

### *Tests for soil in unsaturated condition*

Standard in-situ test interpretation does not consider the matric suction effect that emerges from the unsaturated conditions of various tropical residual soil deposits. In this case, the role of matrix suction and its effect on soil permeability has to be acknowledged and accounted for. The influence of partial saturation imparts a very distinct behaviour to a soil and should be considered in test interpretation. Few aspects of significance deserve attention: (a) suction measurement and its practical significance, (b) suction control in field tests and (c) soil collapsibility.

An important contribution in the analysis of unsaturated soils has been the extension of the elastic-plastic critical state concepts to unsaturated soil conditions by Alonso *et al.* (1990). In this method, the frame of reference is described by four variables – net mean stress $(p - u_a)$, deviator stress $q$, suction $s$ $(u_a - u_w)$ and specific volume $v$, where $u_a$ is the air pressure and $u_w$ the pore water pressure. Several constitutive models have subsequently been proposed following these same concepts (Josa *et al.*, 1992; Wheeler and Sivakumar, 1995). These constitutive models allow derivation of the yield locus in the $(p, q, s)$ space, an analysis that requires nine soil parameters. Model parameters are assessed from laboratory suction-controlled testing such as isotropic compression tests and drained shear strength tests. For isotropic conditions, the model is characterised by the loading-collapse (LC) yield curve whose hardening laws are controlled by the total plastic volumetric deformation. A third state parameter has to be incorporated to include the effect of the shear stress $q$. The yield curve for a sample at a constant

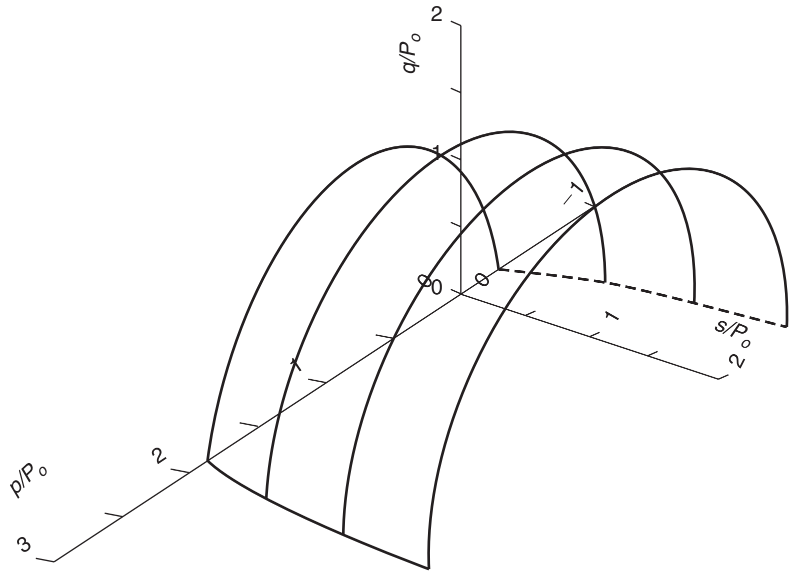


*Figure 3.13* Three-dimensional view of yield surfaces in ($p/P_o$, $q/P_o$, $s/P_o$) space

suction $s$ is described by an ellipse, in which the isotropic preconsolidation stress is given by the previously defined $p_o$ value that lies on the loading-collapse yield curve. The critical state line (CSL) for non-zero suction is assumed to result from an increase in (apparent) cohesion, maintaining the slope $M$ of the CSL for saturated conditions, as illustrated in Figure 3.13.

For incorporating these concepts into the analysis of in-situ tests in unsaturated soil conditions, the first necessary step is to measure the in-situ matric suction. Several techniques have been developed recently to measure the matric suction, such as the non-flushable vacuum tensiometer, the flushable piezometer and the miniature non-flushable tensiometer (e.g. Ridley and Burland, 1995, Marinho, 2000). However, these techniques are normally used in the laboratory in compacted soils or in the field in sedimentary clay deposits.

Suction measurements in a granular granite residual soil site in southern Brazil, in which experience of such measurements is scarce, have been presented by Schnaid *et al.* (2004). At this site, the measured suction ranged from 25 to 70 kPa with a trend of increasing suction during any dry period, which can be attributed to evaporation processes. A typical result is illustrated in Figure 3.14 for measurements of up to 50 kPa. There is no marked difference between the readings recorded from the different instruments tested in the site, which implies that any technique can be used with some confidence in engineering practice for suctions less than 100 kPa in coarse grained soils. The relationship between matric suction and gravimetric water content for in-situ and laboratory specimens is shown in Figure 3.15, over the range of suctions being considered. In general, the data agree well with the general equation proposed by Fredlund and Xing (1994). This relationship is the so-called soil

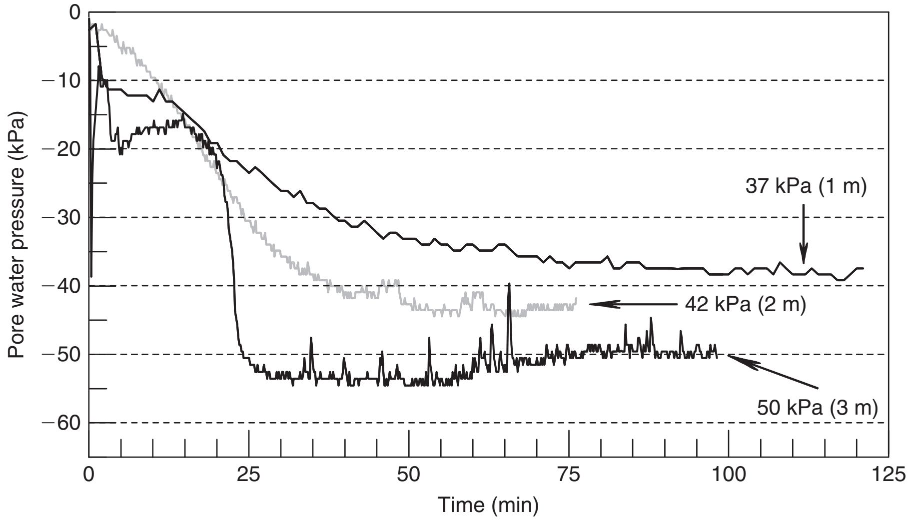


*Figure 3.14* Typical in-situ suction measurements using the miniature tensiometer for the granite residual soil (*After* Schnaid *et al.*, 2004)

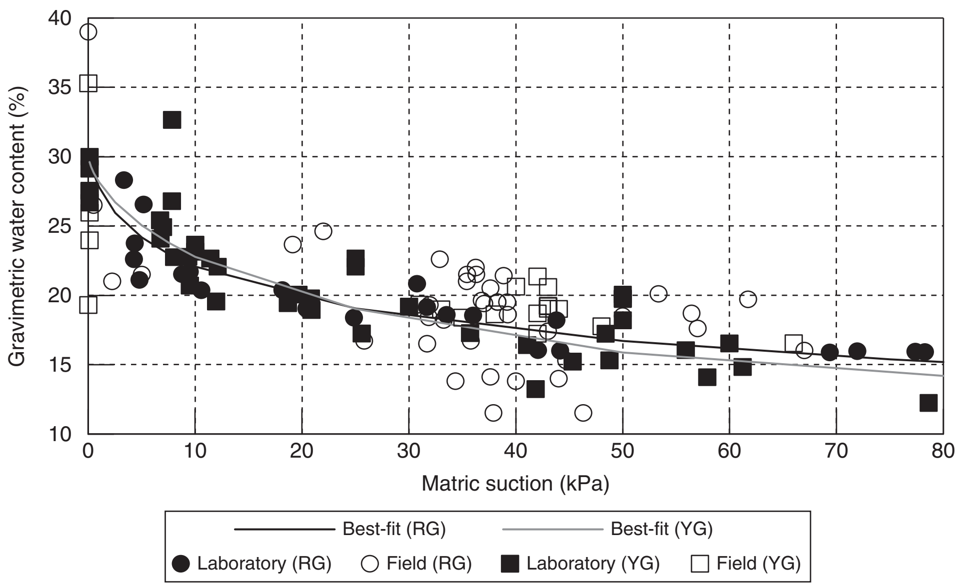


*Figure 3.15* Relationship between matric suction and gravimetric water content for two decomposed granites

water characteristic curve and provides vital information concerning the hydraulic and mechanical behaviour of partially saturated soils.

The recognition that matric suction produces an additional component of effective stress suggests the need to link the magnitude of in-situ suction to the observed response

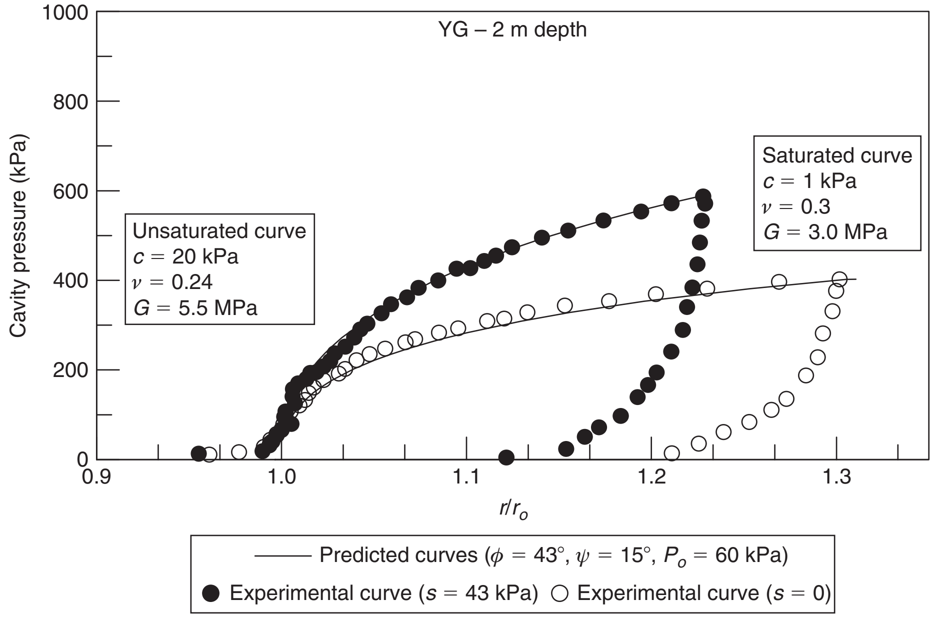


*Figure 3.16* Typical pressuremeter curves for a gneiss residual soil (*After* Schnaid *et al.*, 2004)

of field tests. This led to the development of monitored suction pressuremeter tests, *SMPMT*, in which the in-situ suction is monitored throughout the test by tensiometers positioned close to the pressuremeter probe. First, it is necessary to recognise that the standard self-boring technique cannot be applied to unsaturated soil conditions. The drilling technique using either a flushing fluid or compressed air would produce changes in the pore water pressure $u_w$ or in the pore air pressure $u_a$, affecting the in-situ soil suction $u_a - u_w$ in the vicinity of the pressuremeter probe. The pre-bored technique appears to be a viable option despite its well known limitations.

Typical pressuremeter curves in a granite residual soil are illustrated in Figure 3.16 (Schnaid *et al.*, 2004a). The first test was performed at an in-situ suction of 40 kPa. After soaking the area, another test was carried out; producing a marked reduction in both pressuremeter initial stiffness response and cavity limit pressure. A straightforward conjecture is that stiffness degradation with shear strain is likely to be shaped by changes in matric suction. The response of tensiometers installed at the same depth of the pressuremeter tests at 30 and 60 cm from the centre of the *SMPMT* borehole are also shown in the figure. Suction measurements remained approximately constant throughout the expansion phase in the tests carried out both in soaked and unsaturated soil conditions.

Similar patterns are observed during wetting-induced collapse investigated using both conventional suction-controlled oedometer tests and plate loading tests. Data suggest that shear strains induced by loading do not produce significant changes in matric suction and this enables cavity expansion theory to be extended to accommodate the framework of unsaturated soil behaviour in the interpretation of pressuremeter tests. As a consequence, it is possible to demonstrate that the pressuremeter system is not only suitable for estimating the potential collapse of soils but also for assessing the constitutive parameters that are necessary to describe the 3D-yield surfaces in a $(p, q, s)$ space in unsaturated soils (Schnaid *et al.*, 2004a). The same cavity expansion theoretical background discussed for saturated drained materials remains valid (Schnaid *et al.*, 2004a; Gallipoli *et al.*, 2001).

Schnaid *et al.* (2004a) adapted the theoretical framework from Alonso *et al.* (1990) showing that the mean and deviator stresses on the elasto-plastic boundary becomes

$$p = \frac{P_o}{3}\left(2 + \frac{1}{K_o}\right) - u_a \tag{3.28}$$

and

$$q = P_o\left[\frac{(4K_o^2 - 2K_o + 1)}{K_o^2} - 3\cos\phi'_p\left(\frac{P_o^2\cos\phi'_p - 2P_o c\sin\phi'_p - c^2\cos\phi'_p}{P_o^2}\right)\right]^{1/2} \tag{3.29}$$

where, $\phi'_p$ is the peak frictional angle, $c$ the cohesion intercept, $P_o$ the in-situ horizontal stress or hydrostatic pressure and $K_o$ the coefficient of earth pressure. The yield function for Cam clay, the yield pressure at the isotropic stress state to each given suction level and critical state slope can be calculated to describe the three-dimensional yield surfaces of the soil in a $(p, q, s)$ space.

Model predictions from field and laboratory tests carried out in a gneiss, unsaturated residual soil site are shown in Figure 3.17 (Schnaid *et al.*, 2004) and are considered to be valuable in reproducing features of behaviour of these unsaturated soils. In this figure two- and three-dimensional plots correlating suction and mean net stress are presented in order to compare the load-collapse ($LC$) curve derived from pressuremeter testing data with the $LC$ curve assessed from oedometer data. A discrepancy is observed between these two curves with the oedometer data yielding a collapsible response at much lower mean stress, for any given suction value. There is no reason to assume that these two tests would yield the same $LC$ curve since (a) they reproduce rather distinctive stress paths with the oedometer giving rise to vertical displacements in a constrained ring whereas in the pressuremeter displacements are predominantly radial and (b) the oedometer is carried out in small samples whereas the pressuremeter tests a large body of soil that reflects its macro-structure including fissures and discontinuities. However, both oedometer and pressuremeter delivered the same qualitative information by indicating the collapse potential of the soil even at small suctions within the measured range up to about 100 kPa. The pressuremeter produces this evidence at a much lower cost and faster time, which makes it an attractive tool for practical applications.

In addition, a straightforward approach can be used to assess the collapse of residual soils. The term collapse is applied to unsaturated soils that exhibit a drastic rearrangement of particles and great loss of volume upon wetting with or without additional loading (Jennings and Knight, 1957). The collapse is likely to occur in a soil that has an open fabric with large void spaces giving rise to a metastable structure that emerges from a temporary strength due to capillary tensions in the pore water.

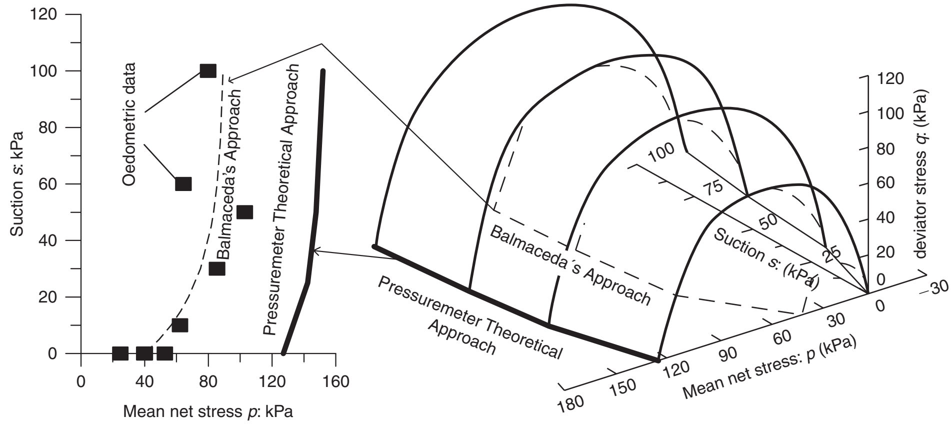


*Figure 3.17* Yield surface prediction for the unsaturated granite residual soil (*After* Schnaid *et al.*, 2004a)

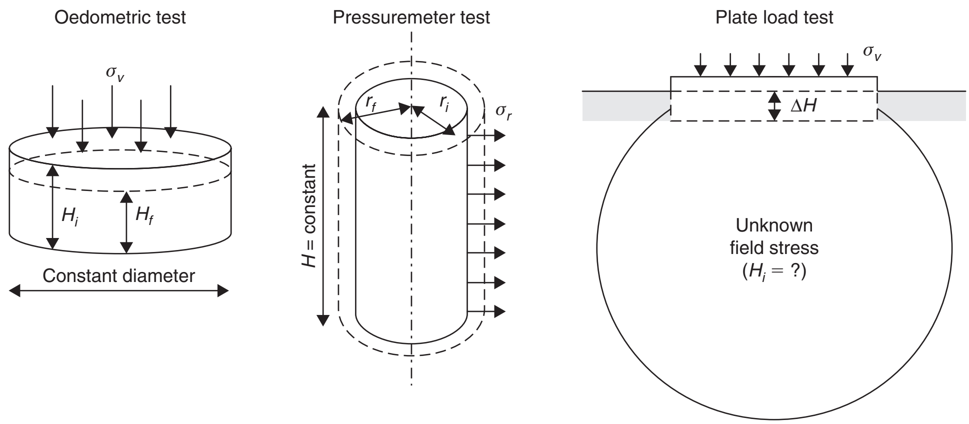


*Figure 3.18* Boundary conditions at different collapsibility tests (*After* Schnaid *et al.*, 2003)

Several engineering problems are often associated with the collapse phenomenon, which can cause extensive damage to buildings, embankments, tunnels and many other engineered structures.

Results from both laboratory and in-situ tests in unsaturated soils can be used directly to assess the collapse potential by means of empirical approaches, or they can be used to determine the parameters associated with specific constitutive models. The three main testing techniques are the oedometer, the plate loading and the pressuremeter tests. Boundary conditions of the three tests during a wetting path are identified in Figure 3.18.

Jennings and Knight (1957) proposed the first method to predict the collapse potential using results from the double oedometer test. The method consists of running two oedometer tests, being one sample at constant natural water content and another

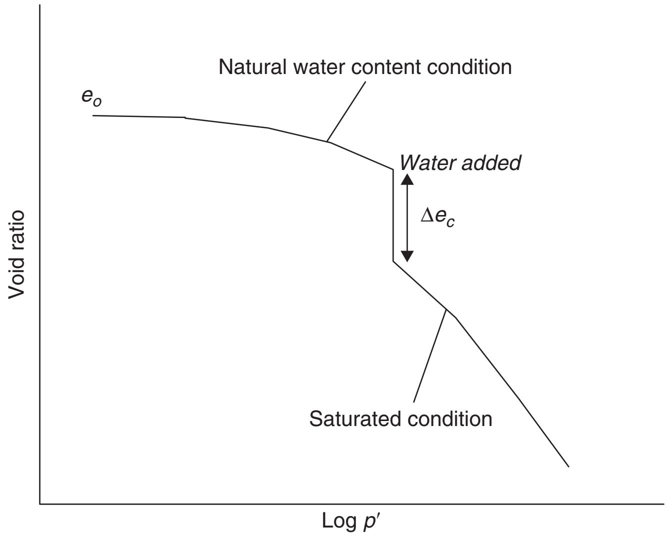


*Figure 3.19* Single oedometer collapse test (*After* Jennings and Knight, 1957)

sample at soaked conditions. The collapse potential can be predicted by quantifying the volume decrease difference, at any given stress level, between the two compression curves. The collapse potential ($CP$) is then defined as

$$CP = \frac{\Delta e}{1 + e_o} \tag{3.30}$$

where, $\Delta e$ is the difference in void ratio between saturated and natural water content conditions, at any stress level, and $e_o$ is initial void ratio. Since the early stages, the authors recognised the difficulty in evaluating the collapse potential in natural soils based on this technique due to differences in the initial void ratio, $e_o$, of different laboratory specimens. But it was just in 1975 that Jennings and Knight presented an alternative testing procedure to assess the collapse potential in the laboratory. An ordinary oedometer test at natural water content is conducted at any load level, and then, the sample is flooded with water, left for 24 hours and the test is then carried on to its normal maximum loading limit. The collapse potential is still determined by equation (3.30), but $\Delta e$ is conveniently replaced by $\Delta e_c$ that signifies the change in void ratio upon wetting. Figure 3.19 shows an idealised view of this single oedometer collapse test. Later in the 1980s, constant suction oedometer tests became a routine in research enabling a more rational interpretation of test results by combining each oedometer curve to a single suction level.

As an alternative procedure, the collapse of soils can be evaluated from results of in-situ plate loading tests. The test is conducted by applying loading to a rigid circular plate at the base of a borehole; at a given stress level the water is introduced and the load displacement response of the soil is monitored (Ferreira and Lacerda,

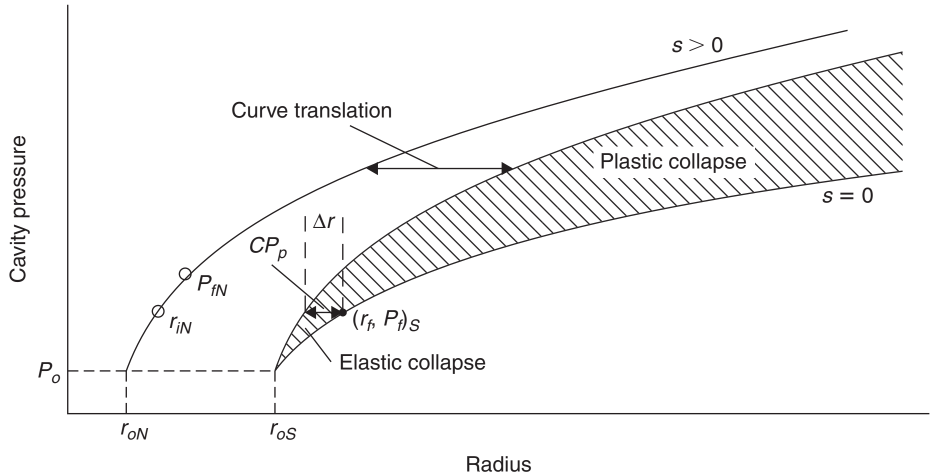


*Figure 3.20* Pressuremeter collapse potential

1993; Houston *et al.*, 1995; Mahmoud *et al.*, 1995; Fonseca and Coutinho, 2008). Interpretation would require assessment to the depth of influence of the stress field, the change in stresses due to wetting and the depth of wetting throughout the test. Because evaluation of the stress field requires numerical analysis where coupling of flow and deformation takes place, a more empirical route has been taken.

A final remark is made to highlight the possibility of obtaining some direct and straightforward assessment of the collapse potential from pressuremeter data (Rollins *et al.*, 1994; Smith *et al.*, 1995; Smith and Rollins 1997; Schnaid *et al.*, 2004a; 2009). Different pressuremeter curves obtained both at the saturated and unsaturated states can give a direct assessment of soil collapsibility, being the collapse potential defined as the distance between a saturated and an unsaturated pressuremeter test (at a given in-situ matric suction), taking the yield stress at the saturated state as reference. This definition is similar to that adopted for the interpretation of the double oedometer test (equation (3.30)) and relies on the assumption that once wetted, the collapse sample follows approximately the stress-strain path of the initially saturated sample in compression or in shear. Since the initial cavity radius varies from one test to another, it is necessary to translate the pressuremeter curves for the same initial reference value. Figure 3.20 illustrates the method of empirically interpreting the collapse potential from the pressuremeter ($CP_p$) which can be calculated as:

$$CP_p = \frac{\Delta V}{V} = \frac{r_{fS}^2 - r_{iN}^2}{r_{iN}^2} - \frac{r_{oS}^2 - r_{oN}^2}{r_{oN}^2} \tag{3.31}$$

where $\Delta V$ and $V$ are the volume change and total volume, $r_{fS}$ and $r_{iN}$ the radius at saturated and unsaturated states (taking the yield saturated pressure $P_{fS}$ as reference) and $r_{oS}$ and $r_{oN}$ the initial radius of the saturated and unsaturated states, respectively.

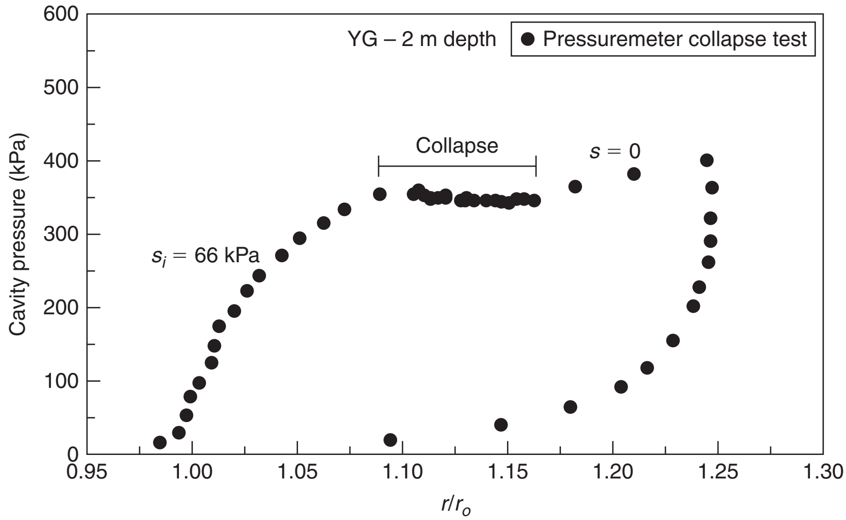


*Figure 3.21* Typical collapse pressuremeter test in residual soil

An alternative testing procedure to assess the collapse potential from pressuremeter tests is presented in Figure 3.21, which is in general terms similar to the concepts adopted for the single oedometer test. An ordinary pressuremeter test at natural water content is performed up to a pressure close to the yielding pressure $P_f$, and then the soil is flooded with water until suction measurements at the tensiometers around the probe drops to values close to zero; the test is then carried on to its normal maximum loading limit. Observation of the increasing radial displacements at a constant radial pressure that results from reducing suction gives a direct assessment of the collapse potential of soils.

## 3.5 SUMMARY AND CONCLUSIONS

This chapter has briefly summarised the main laboratory and in-situ techniques, procedures and interpretation methods applied to residual soils. The unusual stress-strain-strength behaviour that deviates from classical textbook materials has been highlighted and has prompted the development of interpretation methods required in the detailed design of saturated and unsaturated soil conditions, bounded and structured organisation of particles and metastable collapsible response, among other characteristic features.

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