This document downloaded from

vulcanhammer.net

since 1997,
your source for engineering information for the deep foundation and marine construction industries, and the historical site for Vulcan Iron Works Inc.

Use subject to the “fine print” to the right.

All of the information, data and computer software ("information") presented on this web site is for general information only. While every effort will be made to insure its accuracy, this information should not be used or relied on for any specific application without independent, competent professional examination and verification of its accuracy, suitability and applicability by a licensed professional. Anyone making use of this information does so at his or her own risk and assumes any and all liability resulting from such use. The entire risk as to quality or usability of the information contained within is with the reader. In no event will this web page or webmaster be held liable, nor does this web page or its webmaster provide insurance against liability, for any damages including lost profits, lost savings or any other incidental or consequential damages arising from the use or inability to use the information contained within.

This site is not an official site of Prentice-Hall, the University of Tennessee at Chattanooga, Vulcan Foundation Equipment or Vulcan Iron Works Inc. (Tennessee Corporation). All references to sources of equipment, parts, service or repairs do not constitute an endorsement.

Don’t forget to visit our companion site http://www.vulcanhammer.org

Site Investigation

Second Edition

C. R. I. Clayton, M. C. Matthews and N. E. Simons

Department of Civil Engineering, University of Surrey

Contents

Preface
1 Planning and procurement
Introduction
Objectives
General design philosophy
Implementation
Planning ground investigations
Procurement
Execution
2 Description and classification of soils and rocks
Introduction
Soil and rock description
Soil description
Soil classification
Rock description
Description of rock material
Description of discontinuities
Methods of collecting discontinuity data
Discontinuity surveys
Presentation of discontinuity data
Description of rock masses
Records of boreholes
State of recovery of core
Records of trial pits and shafts
3 The desk study and walk-over survey
Introduction
Sources of information for desk studies
Air photography and remote sensing
Satellite remote sensing
The walk-over survey
4 Subsurface exploration: engineering geophysics
Introduction
Lateral variability
Profiling
Sectioning
Determination of properties
5 Subsurface exploration: boring, drilling, probing and trial pitting
Introduction
Boring
Drilling
Probing
Examination *in situ*
6 Sampling and sample disturbance
Introduction
Sample sizes
Soil disturbance
Classification of soil samples
British practice, and the BS 5930 classification
7 Undisturbed sampling techniques
Introduction

Samples from pits and exposures
Drive samplers
Rotary samplers
Sand sampling
Sampler selection
8 Laboratory testing
Introduction
The purpose of soil testing
Available tests
Consolidation tests
Accuracy and measuring systems
9 *In situ* testing
Introduction
Penetration testing
Strength and compressibility testing
Permeability testing
10 Basic field instrumentation for site investigation
Introduction
Uses of instrumentation
Requirements for instrumentation
Pore water pressure and groundwater level measurement
Displacement measurement
Other measurements

References and standards
Index

Preface

It is now over 12 years since we completed the first edition of this book. In the intervening time there have been a number of important advances in the way that site investigations are carried out, both in the UK and elsewhere. In this new edition we have described those techniques which are now in regular commercial use, some of which were presaged in our first edition. But, as in the first edition of this book, we have avoided including descriptions of techniques which we believe will remain largely in the research field.

For the second edition we have added substantial new material on:

- specification and procurement;
- desk studies;
- geophysical investigation techniques;
- sample disturbance and sampling methods;
- *in situ* testing; and
- laboratory testing.

The object of this book remains the same: we aim to improve the quality of site investigation by providing a relatively simple and concise reference book intended to be read by civil and structural engineers and engineering geologists, when undergraduates and postgraduates, but particularly when in practice. The text is intended to inform the reader of available techniques, and to illustrate the advantages and disadvantages of these techniques.

Site investigation is a complex process. It is vital to the success of any construction project, since inadequate investigation can lead to very large construction cost overruns. If site investigation is to be effective then it must be carried out in a systematic way, using techniques that are relevant, reliable and cost-effective. We hope that readers will find this text a useful introduction to this important part of the building and construction process.

Chapter 1

Planning and procurement

Unfortunately, soils are made by nature and not by man, and the products of nature are always complex.

Karl von Terzaghi, 1936

INTRODUCTION

Site investigation is the process by which geological, geotechnical, and other relevant information which might affect the construction or performance of a civil engineering or building project is acquired.

Soil and rock are created by many processes out of a wide variety of materials. Because deposition is irregular, soils and rocks are notoriously variable, and often have properties which are undesirable from the point of view of a proposed structure. Unfortunately, the decision to develop a particular site cannot often be made on the basis of its complete suitability from the engineering viewpoint; geotechnical problems therefore occur and require geotechnical parameters for their solution.

Site investigation will often be carried out by specialists in the field of soil mechanics. Soil, in the engineering sense, is the relatively soft and uncemented material which overlies the rock of the outer part of the Earth's crust. Specialists in the mechanical behaviour of soil are normally civil engineers and in the UK they will often have some postgraduate geotechnical education: such people are termed 'soils engineers' or 'geotechnical engineers'. Geologists with an interest in the relevance of geology to civil engineering or building construction are called 'engineering geologists'.

Soil mechanics in its present form is a relatively recent addition to the field of engineering. Interest in the behaviour of earth and rock for engineering purposes can be traced back to Roman times (Palladius in Plommer (1973)), but significant advances in analysis seem to date back to the eighteenth century, when the need for large defensive revetments led to early work on retaining walls. Coulomb's paper, delivered to the Académie Royale des Sciences in 1773 and published in 1776, represents an early work which showed considerable understanding, *inter alia*, of the behaviour of soil, and whose results are still valid and in use (Heyman 1972). Subsequent papers, principally delivered by the French, did much to refine the available solutions but little to increase fundamental knowledge.

By the first quarter of the nineteenth century, it appears that many concepts now associated with the principle of effective stress were intuitively understood. Telford used pre-loading during the construction of the Caledonian Canal in 1809 'for the purpose of squeezing out the water and consolidating the mud', and Stephenson used drains to lower pore pressures during the construction of the Chat Moss embankment on the Liverpool and Manchester Railway in the years 1826 to 1829 'in order to consolidate the ground between them on which the road was to be formed' (Smiles 1874). During the industrial period preceding the twentieth century, many of the currently used geotechnical processes for the improvement of ground, such as piling, pre-loading, compaction and de-watering appear to have been used (Feld 1948; Skempton 1960b; Jensen 1969). These techniques were applied in a purely empirical manner.

At the turn of the twentieth century, a series of major failures occurred which led to the almost

simultaneous formation of geotechnical research groups in various countries. In America, slope failures on the Panama Canal led to the formation of the American Foundations Committee of the American Society of Civil Engineers in 1913 and, in Sweden, landslides during a railway construction resulted in the formation of the State Geotechnical Commission in the same year. Following a number of embankment and dyke failures, a government committee under Buisman was set up in Holland in 1920. Casagrande (1960), however, dates the advent of modern soil mechanics to the period between 1921 and 1925, when Terzaghi published several important papers relating to the pore pressures set up in clay during loading, and their dissipation during consolidation, and also published his book *Erdbaumechanik auf Bodenphysikalischer Grundlage*.

These works largely stemmed from Terzaghi's appreciation of the need to supplement geological information with numerical data, following two years spent collecting geological information on the construction sites of US dams (Terzaghi 1936).

Terzaghi's first professional work in England was in 1939, when he was retained to investigate a slope failure at the Chingford reservoir (Cooling and Golder 1942). As a result, the first commercial soil mechanics laboratory in the UK was established by John Mowlem and became Soil Mechanics Ltd in 1943. Whyte (1976) reports that by 1948 five other contractors and one consultant had soils divisions. Major encouragement was given to soils research in the UK by Cooling, who influenced a number of engineers (for example Skempton, Bishop and Golder) who worked at the Building Research Station in the 1940s. In 1948, *Géotechnique* commenced publication, and by 1955 a great number of significant papers on soil mechanics had been published covering topics such as site investigation, seepage, slope stability and settlement.

According to Mayniel (1808), Bullet was the first to try to establish an earth pressure theory, in 1691. More importantly from our point of view, Bullet notes the importance of *site investigation* for the foundations of earth-retaining structures and recommends the use of trial holes in order to determine the different beds of soil beneath a site, and in order to ensure that poor soil does not underlie good soil. Where trial holes could not be made, Bullet recommended the use of an indirect method of investigation whereby the quality of the soil was determined from the sound and penetration achieved when it was beaten with a 6—8 ft length of rafter.

Whilst the use of trial holes to investigate sub-soil may, not unexpectedly, date from centuries ago, it is more surprising to note that the equipment for boring holes in soft ground also has a long history. Jensen (1969) and Whyte (1976) illustrate types of drilling equipment in use around 1700, and many of the tools bear a striking resemblance to those used in light percussion drilling at the present time in the UK.

Modern site investigation differs from its forbears principally because of the need to quantify soil behaviour. Terzaghi, in his James Forrest lecture to the Institution of Civil Engineers in London (1939) noted that in 1925 sampling methods in the USA were 'primitive', with sealed tube samples being almost unheard of. The work of Casagrande between 1925 and 1936 demonstrated the influence of soil disturbance during sampling (see, for example, Casagrande (1932)) and led to the development in the USA of 'elaborate and ingenious procedures for furnishing almost undisturbed samples up to a diameter of 5-inches' (Terzaghi 1939). At the same time considerable advances were made in Denmark, France, Germany, Sweden and England.

In the UK, Cooling and Smith (1936) reported an early attempt at the acquisition of 'undisturbed' soil samples using a 105 mm dia. split tube forced into the ground from the back of a lorry. By 1937 the tool was a 105 mm dia. tube which was driven into the soil (Cooling and Golder 1942; Cooling 1942), and which had an area ratio (the ratio of displaced soil area to sample area) of about 20%. Boring was by well-boring apparatus, 'sunk in the usual way with augers, chisels, etc.' (Cooling 1942). By 1945 the sampling tube had become the U100 which is still in use today (Longsdon 1945).

In 1949, the first draft *Civil Engineering Code of Practice for Site Investigations* was issued for

comment. At that time Harding (1949) delivered a paper to the Works Construction Division of the Institution of Civil Engineers in which he detailed the methods of boring and sampling then available. The recommendations made in that paper, and in discussions on the paper by Skempton, Toms and Rodin form the basis of the majority of techniques still in use in site investigation in the United Kingdom. For example, in his discussion on methods of boring, Harding notes that:

> the boring equipment used in site investigations is criticized by some who have not been exposed to the need to carry it themselves, as being primitive and lacking in mechanization. Whilst it is possible to think of many ingenious contrivances for removing articles at depths below ground, in practice simple methods usually prove to be more reliable.

while Skempton confirmed this view:

> with that simple equipment [shell and auger gear and 102 mm dia. sampler] the majority of site investigations in soils could be carried out and, moreover, sufficient experience was now available to enable the positive statement to be made that, in most cases, the results obtained by that technique (in association with laboratory tests) were sufficiently reliable for practical engineering purposes.

By 1953, Terzaghi stated in connection with site investigation that ‘we have acquired all the knowledge which is needed for a rational interpretation of the observational and experimental data’. The reader may reasonably ask what is to be gained from this book, since techniques are so well established. In reality, since 1950, four main changes have taken place. First, many of the methods introduced before and since have been the object of criticism as a result of differences between predictions and subsequent observations. Secondly, a considerable number of the lessons learnt before 1950 have been forgotten: few U100 samplers in use today are of the standard required by Hvorslev (1949) for undisturbed sampling, and much fieldwork remains unsupervised by engineers. Thirdly, few engineers have an experience or understanding of the techniques of boring and drilling holes for site investigations, and most clients remain unaware of the importance of this part of the work. Finally, recent years have seen the introduction of sophisticated and expensive methods of testing and computer analysis which cannot be sensibly applied to samples and predictions of soil conditions of indeterminate quality.

The Civil Engineering Code of Practice No. 1: Site Investigations was issued in 1950, and revised as British Standard Code of Practice CP 2001 in 1957. This code has now been extended, completely rewritten and re-issued as British Standard 5930:1 981. At the time of writing (1992) BS 5930 is under revision. The code contains much valuable information, but it is perhaps necessary to ask whether it is wise to codify in this way. Terzaghi (1951) argued that:

> since there is an infinite variety of subsoil patterns and conditions of saturation, the use of the different methods of subsoil exploration cannot be standardised, but the methods themselves still leave a wide margin for improvement, as far as expediency and reliability are concerned.

OBJECTIVES

The objectives of site investigation have been defined by the various *Codes of Practice* (*BS CP 2001:1950, 1957; BS 5930:1981*). They can be summarized as providing data for the following.

1. *Site selection*. The construction of certain major projects, such as earth dams, is dependent on the availability of a suitable site. Clearly, if the plan is to build on the cheapest, most readily available land, geotechnical problems due to the high permeability of the sub-soil, or to slope instability may make the final cost of the construction prohibitive. Since the safety of lives and property are at stake, it is important to consider the geotechnical merits or demerits of various sites before the site is chosen for a project of such magnitude.
2. *Foundation and earthworks design*. Generally, factors such as the availability of land at the right price, in a good location from the point of view of the eventual user, and with the

planning consent for its proposed use are of over-riding importance. For medium-sized engineering works, such as motorways and multistorey structures, the geotechnical problems must be solved once the site is available, in order to allow a safe and economical design to be prepared.

3. *Temporary works design*. The actual process of construction may often impose greater stress on the ground than the final structure. While excavating for foundations, steep side slopes may be used, and the in-flow of groundwater may cause severe problems and even collapse. These temporary difficulties, which may in extreme circumstances prevent the completion of a construction project, will not usually affect the design of the finished works. They must, however, be the object of serious investigation.
4. *The effects of the proposed project on its environment*. The construction of an excavation may cause structural distress to neighbouring structures for a variety of reasons such as loss of ground, and lowering of the groundwater table. This will result in prompt legal action. On a wider scale, the extraction of water from the ground for drinking may cause pollution of the aquifer in coastal regions due to saline intrusion, and the construction of a major earth dam and lake may not only destroy agricultural land and game, but may introduce new diseases into large populations. These effects must be the subject of investigation.
5. *Investigation of existing construction*. The observation and recording of the conditions leading to failure of soils or structures are of primary importance to the advance of soil mechanics, but the investigation of existing works can also be particularly valuable for obtaining data for use in proposed works on similar soil conditions. The rate of settlement, the necessity for special types of structural solution, and the bulk strength of the sub-soil may all be obtained with more certainty from back-analysis of the records of existing works than from smallscale laboratory tests.
6. *The design of remedial works*. If structures are seen to have failed, or to be about to fail, then remedial measures must be designed. Site investigation methods must be used to obtain parameters for design.
7. *Safety checks*. Major civil engineering works, such as earth dams, have been constructed over a sufficiently long period for the precise construction method and the present stability of early examples to be in doubt. Site investigations are used to provide data to allow their continued use.

According to US 5930: 1981, site investigation aims to determine all the information relevant to site usage, including meteorological, hydrological and environmental information. Ground investigation aims only to determine the ground and groundwater conditions at and around the site; this is normally achieved by boring and drilling exploratory holes, and carrying out soil and rock testing. In common engineering parlance, however, the terms site investigation and ground investigation are used interchangeably.

GENERAL DESIGN PHILOSOPHY

Site investigation should be an integral part of the construction process. Unfortunately it is often seen as a necessary evil — a process which must be gone through by a designer if he or she is to avoid being thought incompetent, but one which gives little of value and takes precious time and money. This is an unfortunate by-product of the way in which site investigation is often carried out, and it can hardly be surprising that if no effort is put into targeting the investigation to precise issues, then little of value emerges.

Site investigation should be a carefully considered process of scientific discovery, tailored both to the conditions existing on site and to the form of construction which is expected to take place. In order to make the most of site investigation, it is important that the design team (who may be led by architects, quantity surveyors and other non- engineering professionals) obtain at the conceptual design stage the advice, however briefly, of a geotechnical engineer. This geotechnical specialist can give the initial and most important guidance on the likely risks associated with the project, and the way in which they

may be investigated and dealt with. For most construction projects, the natural variability of the ground and groundwater conditions represent a major risk, which if not properly addressed can endanger not only the financial viability, but also the physical stability of the construction, either during construction or during the use of the building.

In principal then, all sites must be investigated if construction is to be safe and economical. In practice, the way in which they are investigated can vary very widely, and the costs and time necessary will also be significantly different. The keys in selecting the most effective method of dealing with the inevitable uncertainties which must arise are geotechnical knowledge and experience. Possible approaches which have been successfully used include the following.

Approaches to site investigation

Approach 1: Desk-study and geotechnical advice

The minimum requirement for a satisfactory investigation is that a desk study and walk-over survey are carried out by a competent geotechnical specialist, who has been carefully briefed by the lead technical construction professional (architect, engineer or quantity surveyor) as to the forms and locations of construction anticipated at the site.

This approach will be satisfactory where routine construction is being carried out in well-known and relatively uniform ground conditions. The desk study and walk-over survey (see Chapter 3) are intended to:

1. confirm the presence of the anticipated ground conditions, as a result of the examination of geological maps and previous ground investigation records;
2. establish that the variability of the sub-soil is likely to be small;
3. identify potential construction problems;
4. establish the geotechnical limit states (for example, slope instability, excessive foundation settlement) which must be designed for; and above all, to
5. investigate the likelihood of unexpected' hazards (for example, made ground, or contaminated land).

It is unlikely that detailed geotechnical design parameters will be required, since the performance of the proposed development can be judged on the basis of previous construction.

Approach 2: 'Standard' ground investigation

For most projects a more elaborate approach is needed, and will generally follow the following course.

1. A desk study and walk-over survey must first be carried out, to establish the likely conditions on and below the site, as described above (and see Chapter 3).
2. The details of the proposed construction must be ascertained, in as much as they have been decided. Particular care should be taken to establish the probable loading conditions and the sensitivity of any structures to be built, or those already existing on, around or below the site, to the changes that will occur as a result of construction. For example, services and tunnels passing below or alongside a proposed excavation for a basement may be damaged by the movements caused by excavation, and buildings above a proposed tunnel may be damaged by changes in the groundwater conditions and any ground loss caused by construction.
3. From the combinations of construction and ground conditions, the need for particular foundation types, for retaining walls, for cut slopes, and for special construction processes (such a grouting, dewatering and ground improvement) should be determined. These will bring with them particular limit states, and where limit states cannot be avoided (for example,

by changing the configuration of the proposed construction) there will be a need to carry out geotechnical analyses.

EXAMPLE: POTENTIAL LIMIT STATES

- Bearing capacity failure of foundations
- Differential settlement of foundations leading to structural damage
- Instability of clay slopes
- Sulphate attack on concrete
- Damage by mining subsidence
- Damage to surrounding structures as a result of excavating or dewatering an excavation
- Ground collapse over pre-existing natural solution features
- Collapse of excavations as a result of excessive water inflow.

The identification of potential limit states is a matter of experience, education and pessimism. 'Confidence may impress the Client, but it has little effect on the forces of nature' (Skempton 1948).

4. At this stage the geotechnical designer for the project will need to estimate (from experience, or from published values, in papers, or from previous investigations in the same strata) the likely values of parameters required for analyses of limit states, for the various types of ground expected to occur at the site. Some preliminary geotechnical design of the project is required, in order to recognize that only a few of the possible limit states are likely to have to be faced, and therefore that more detailed investigations will not be required for many parameters:
   - where possible, limit states should be avoided, by choosing an appropriate form of structure (for example, by piling through soft clays, rather than designing for bearing capacity failure of shallow foundations);
   - it will be recognized that certain limit states will not be a problem (e.g. the bearing capacity of shallow foundations on rock).

At this stage critical parameters, essential to the successful completion of the project, must be recognized.

EXAMPLE: PARAMETERS REQUIRED FOR THE DESIGN OF A FOUNDATION IN CLAY

- Bulk unit weight of clay
- Undrained strength of clay
- Compressibility of clay
- Variability of the above, both laterally and with depth
- Groundwater level
- Sulphate content of groundwater
- Acidity of groundwater.

5. From a knowledge of the probable ground conditions and the required parameters, the geotechnical specialist should now identify all possible ways of determining the required parameters. Many tests that might be used (see Chapters 8 and 9) will only work satisfactorily in limited ground conditions, so limiting the available choice.

In principle, the parameters may be obtained:

- based on published data from other sites;
- based on previous site investigation data;
- back-analysis of performance of nearby construction;

- back-analysis of observed performance during construction;
- laboratory testing on samples taken during ground investigation; and
- *in situ* testing during ground investigation.

In order to optimize the investigation, estimates of:

- relative accuracy;
- relative cost;
- availability, and
- relevance to the problem

should be assessed for each way of determining the parameters.

EXAMPLE: DETERMINATION OF THE COMPRESSIBILITY OF FRACTURED WEAK ROCK

SPT	Cheap, readily available, widely accepted, usable at any depth, inaccurate
Plate test	Expensive, readily available, accurate, widely accepted, difficult to use at depth
Surface-wave geophysics	Cheap, not readily available, relatively accurate, shallow only, not widely accepted
Back analysis	Virtually free, readily available, relatively accurate, any depth, may not be relevant if site conditions are unusual.

At the same time the degree of sophistication and the accuracy required for each type of geotechnical analysis should be determined.

For *ultimate limit states* (i.e. where collapse is involved) consider the cost of failure, in terms of:

- legal;
- political; and
- financial consequences.

For *serviceability limit states* (i.e. where collapse does not occur, but the use of the structure is impaired) consider:

- savings which might be made in construction costs if parameters were better known; and
- the reduction in risk that might be achieved by using better analytical methods, based upon sounder engineering, with more sophisticated parameters.

EXAMPLE: ESTIMATION OF GROUND MOVEMENTS AROUND DEEP EXCAVATIONS IN THE CITY OF LONDON

Despite the generally large cost of civil engineering construction it is common to base routine design on basic parameters obtained from the SPT (Chapter 9), and routine undrained triaxial and oedometer testing (Chapter 8), both of which generally give very conservative (i.e. over-safe) estimates of ground movements. It is relatively unusual to base design on back-analysed parameters, or on more sophisticated and well-instrumented laboratory stress-path testing, despite the proven ability of these forms of parameters, in conjunction with finite element analysis, to give good predictions of movements around large excavations in the London clay.

The cost of using higher quality ground investigation and analytical techniques is typically less than

0.1% of the total cost of land purchase, architectural and structural design, and construction. Therefore it is worth considering whether these techniques may be used to justify greater site usage, such as building more basements and/or building closer to neighbouring structures. As will be seen below, increasing expenditure on geotechnical engineering can also be used to reduce the complexity of the construction process, which will lead directly to reductions in construction cost.

6. The details of the ground investigation can now be decided. The investigation boreholes should be of sufficient depth and distribution to establish the position of interfaces between different types of soil (within the zone likely to affect the construction), and the *in situ* testing and soil sampling should be planned so that the soil can be grouped into different categories (for example, rock, clay, sand, organic material — see Chapter 2) as well as tested to provide the specific parameters necessary for design calculations. This facet of the planning and design of a ground investigation is considered later in this chapter.

Approach 3: Limited investigation, coupled with monitoring

In some projects, it may be possible to carry out redesign during construction, in order to reduce costs. Given the natural variability of the ground, geotechnical engineers routinely use 'moderately conservative' soil parameters in design calculations, and do not normally attempt genuine predictions of values such as settlement, ground movements adjacent to excavations, etc. The example below illustrates how the demolition and reconstruction process was modified during construction, on the basis of moderately conservative design using finite element and boundary element analysis, and observations of ground movements.

EXAMPLE: SITE INVESTIGATION AND REDESIGN DURING CONSTRUCTION, FOR A BUILDING OVER A TUNNEL IN CENTRAL LONDON

The development of Grand Buildings, in Trafalgar Square, London, required demolition and reconstruction techniques which could guarantee that damage to underlying Underground railway tunnels would be avoided (Clayton *et al*. 1991). The relative location of Grand Buildings, with respect to the underlying tunnels, can be seen in Fig. 1.1—the closest tunnels, approximately l0m in diameter, lie only 5m below the basement of the new building. It was thought that the effects of construction on the underlying tunnels would be acceptably small if ground movements at the tunnel level were less than 15 mm.

Initial designs were based upon limited and rather routine' ground investigation, involving just two boreholes. Strength and compressibility values were determined from standard triaxial and oedometer testing (see Chapter 8). These values were not, however, used in estimating ground movements around the structure, since it is known (Fig. 1.2) that in this part of the London clay deposit they very significantly underestimate the stiffness of the ground. Instead, the movements were calculated using finite element and boundary element computer methods, incorporating the ground stiffness values back analysed from observations of movements at the Hyde Park Cavalry Barracks, a site some distance away, but still in similar London clay.

Even using these, much higher, stiffness parameters the estimated ground movements were large. In order to limit the predicted tunnel movements a complex 20-stage sequence of demolition and construction was developed, which involved construction of foundations from within the existing building, in a number of small areas, with underpinning, and the intermixing of construction with demolition, the provision of some kentledge to limit the effects of unloading, and extensive temporary works to support the partly demolished building.

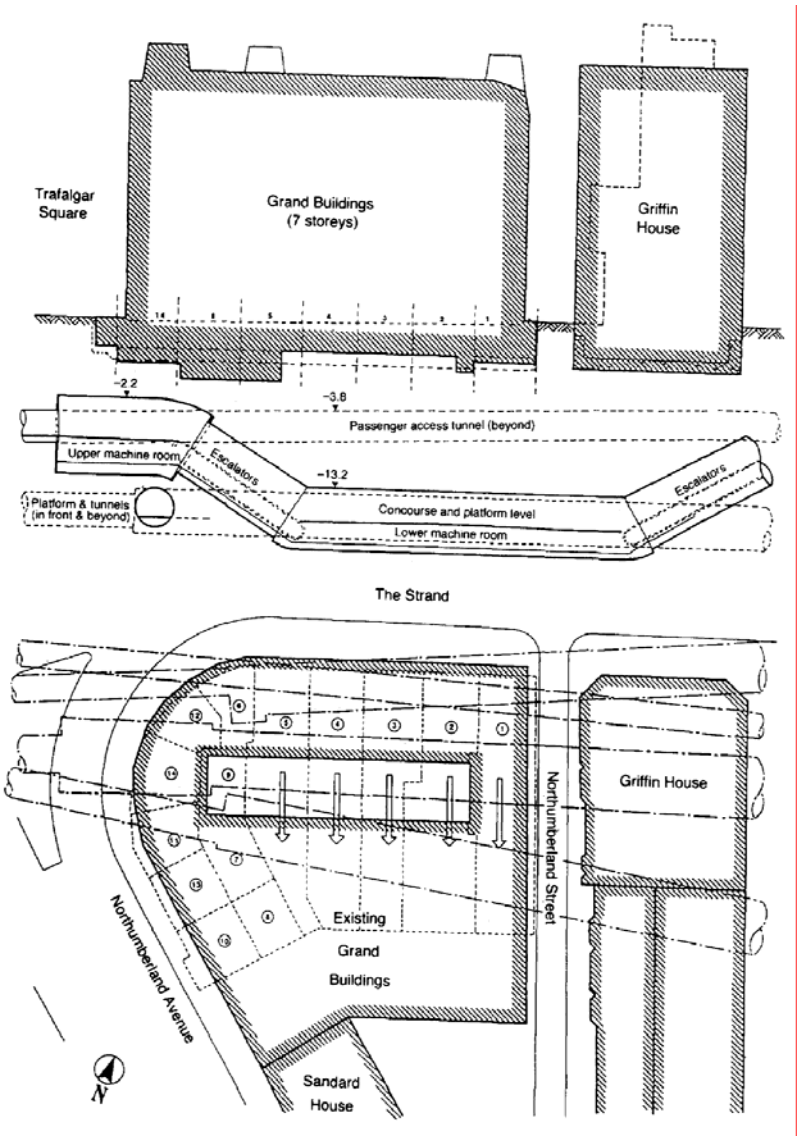


Fig. 1.1 Elevation showing a W—E section through Grand Buildings (above), and plan showing proposed demolition and raft construction sequence (below).

During planning of the construction process it soon became apparent that the proposed sequence of demolition and reconstruction would prove very complex and time consuming in execution, and that therefore economies of time and cost might be achieved through a redesign. In the absence of good-quality site-specific soil parameters for use in further analyses, an observational approach was developed. This was not the Observational Method *sensu stricto* (see below), but a strategy based firmly upon measurement of a critical parameter, vertical displacement, at the level of the most critical tunnel. The strategy involved:

1. assessment of the available information on the London clay, including experience gained by the design-and-build contractor in constructing the adjacent Griffin House;
2. adoption of moderately conservative soil stiffness parameters, and a conservative demolition and reconstruction scheme starting at the least-sensitive (Griffin House) end of the existing Grand Buildings;
3. boundary element analysis to predict the movements at various levels beneath the structure, and especially at the most critical tunnel location, and along the Passenger Access Tunnel which runs at the same level from the Upper Machine Room towards Griffin House;
4. incorporation in the plans of elements of work which could be abandoned if the predicted ground movements were proved to be pessimistic;

5. monitoring of movements within the Passenger Access Tunnel, especially during the early stages of demolition; and
6. re-assessment of ground stiffness parameters, and re-design of the demolition and reconstruction programme, as the demolition proceeded.

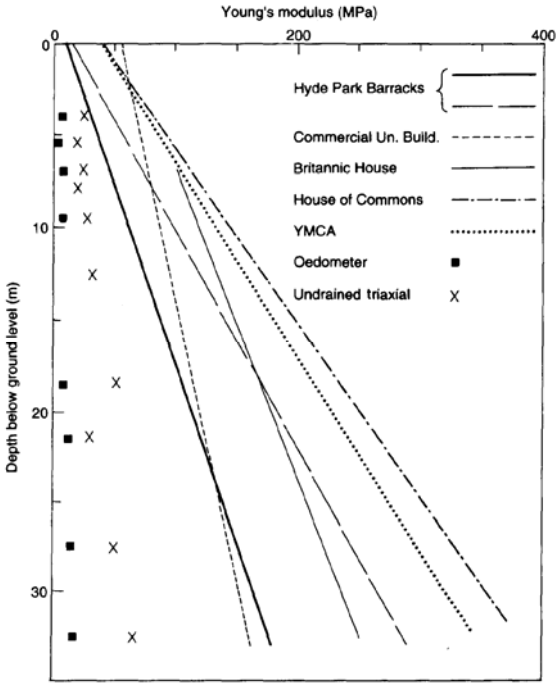


Fig. 1.2 Comparison of Young's modulus values for the London clay at Grand Buildings, obtained from routine undrained triaxial and oedometer testing, with values back analysed from observed movements around other excavations in the London area.

The resulting demolition areas (in numbered circles, according to sequence) are shown in Fig. 1.1. As a result of early measurements, during the demolition and excavation of strip 1, it became clear that the design analysis had significantly overestimated the heave. Therefore the planned 'back-load' kentledge was not used, except on strip 5 and immediately above the Upper Machine Room, and demolition was allowed to proceed simultaneously over the entire site. Monitoring continued throughout demolition and reconstruction. A maximum heave of the order of 4.3mm was measured, compared with values of the order of 10—15mm predicted by finite element and' boundary element analyses for the original design.

Approach 4: The observational method

This is a carefully considered approach to geotechnical design, developed by Peck (1969).

Peck (1969) ascribed Terzaghi's great success to his use of observation, coupled with his insistence on full, personal responsibility and authority on critical jobs. Clearly variations in financial constraints, the complexity of soil conditions, and time restrictions mean that very different approaches can be taken during site investigation. Peck argued that the methods available for coping with the inevitable uncertainties which arise as a result of the natural variability of soil and rock conditions broadly form three groups.

1. *Method 1*: Carry out limited investigation, and adopt an excessive factor of safety during design.
2. *Method 2*: Carry out limited investigation, and make design assumptions in accordance with general average experience.

3. *Method 3*: Carry out very detailed investigation.

In the first two methods only the vaguest approximations to the values of the physical properties of the sub-soil can be obtained. The variability of the soil properties, together with the degree of continuity of the individual layers of soil are almost certainly unknown, and groundwater conditions will not usually be adequately defined. Under these conditions it is almost certain that method 1 will be wasteful, while method 2 can frequently be dangerous. Only in the cases of investigations of major projects is there any likelihood that sufficient funds will be available for very detailed investigations, and in many cases the financial return will not merit this approach. Peck (1969) gives the ingredients of the 'observational method' as follows:

1. exploration sufficient to establish at least the general nature, pattern and properties of the deposits, but not necessarily in detail;
2. assessment of the most probable conditions and the most unfavourable conceivable deviations from these conditions. In this assessment geology often plays a major role;
3. establishment of the design based on a working hypothesis of behaviour anticipated under the most probable conditions;
4. selection of quantities to be observed as construction proceeds, and calculation of their anticipated values on the basis of the working hypothesis;
5. calculation of values of the same quantities under the most unfavourable conditions compatible with the available data concerning the subsurface conditions;
6. selection in advance of a course of action or modification of design for every foreseeable significant deviation of the observational findings from those predicted on the basis of the working hypothesis;
7. measurement of quantities to be observed and evaluation of actual conditions during construction; and
8. modification of the design to suit actual conditions.

A simple example of the observational method is given by Peck (1969). The pressures applied by soil to a strutted excavation are, to this day, a matter of considerable uncertainty. Conventional design methods assume worst conditions, as determined by various instrumented sections (for example, Peck (1943)). The Harris Trust building was to be constructed in Chicago, and the contractor had to design a bracing system (Fig. 1.3) for the excavation for foundations. He had at his disposal various measurements of strut loads on similar ground in Chicago and could therefore predict with some certainty the maximum strut loads that would occur.

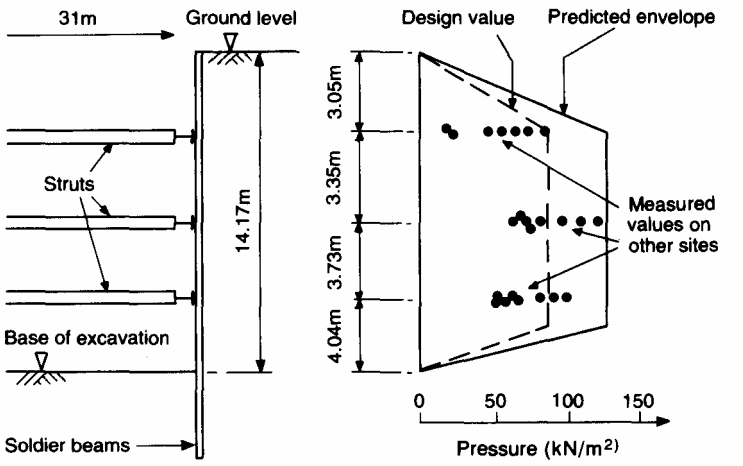


Fig. 1.3 The Harris Trust Excavation (Peck 1969).

The design of the struts could have been based on the trapezoidal diagram, providing a safe but uneconomical design since most of the struts would have carried much less load than their capability.

The contractor proposed to design the struts at a relatively low factor of safety, for loads of about two-thirds the envelope values, or about average measured load conditions. This achieved considerable economy. To guard against higher loads the contractor measured the axial load in every strut during construction, and had available extra struts for immediate insertion if necessary. Only three struts were required in addition to the thirty-nine originally designed for the whole project.

Not only did this approach produce a large saving in construction costs; it also, and perhaps more importantly gave the absolute certainty that no strut in the system was overloaded.

The observational method is now frequently claimed to be used, when in fact all the essential components described above have not been adhered to. In 1985 Peck noted that:

> the observational method, surely one of the most powerful weapons in our arsenal, is becoming discredited by misuse. Too often it is invoked by name but not by deed. Simply adopting a course of action and observing the consequences is not the observational method as it should be understood in applied soil mechanics. Among the essential but often overlooked elements are to make the most thorough subsurface explorations that are practicable, to establish the course of action on the basis of the most probable set of circumstances and to formulate, in advance, the actions that are to be taken if less favorable or even the most unfavorable conditions are actually encountered. These elements are often difficult to achieve, but the omission of any one of them reduces the observational method to an excuse for shoddy exploration or design, to dependence on good luck instead of good design. Unhappily, there are far too many instances in which poor design is disguised as the state of the art merely by characterizing it as an application of the observational method.

IMPLEMENTATION

It has already been noted that early site investigation in Britain was associated with work by the Building Research Station, and by contractors. During this period the response of contractors rather than consultants in setting up geotechnical organizations meant that by the late 1940s a high proportion of the experience, expertise and facilities available for site investigation was held by contracting firms. As a result, site investigation in the UK became a contractual operation. At the present time, much of the work of site investigation is carried out on the basis of a competitive tender.

At the present time then, most site investigation in Britain is commissioned by local authorities, government organizations or consulting engineers, on behalf of their clients. Typically the engineer produces conditions of contract, a specification, and a bill of quantities, and the tenderer receives a plan showing the proposed borehole locations. Provisional borehole depths and sampling routines are normally given, and the contractor will be told whether he is to provide a factual report, or whether both factual and interpretative reports on the project are required. Whether interpretation is required or not, it can be seen that the contractor is under great pressure to work quickly and efficiently, for the company will have quoted fixed prices for work to be carried out in uncertain ground and groundwater conditions.

It has been found that the best site investigations involve a considerable number of activities, some of which may become relatively unimportant in some cases, but should never be forgotten. An ideal order of events might be as shown in Table 1.1.

The sequence of geotechnical site investigation might be:

1. preliminary desk study, or fact-finding survey;

2. air photograph interpretation;
3. site walk-over survey;
4. preliminary subsurface exploration;
5. soil classification by description and simple testing;
6. detailed subsurface exploration and field testing;
7. the physical survey (laboratory testing);
8. evaluation of data;
9. geotechnical design;
10. field trials; and
11. liaison by geotechnical engineer with site staff during project construction.

Table 1.1 Order of events for site investigations

Project design team	Geotechnical designers	Geotechnical contractor
Definition of project	Geotechnical advice on likely design issues	
Site selection	Preliminary desk study to advise on relative geotechnical merits of different sites	
Conceptual design	Geotechnical advice on optimizing structural forms and construction methods, in order to reduce sensitivity of proposed construction to ground conditions	
	Detailed desk study and walk-over survey to produce a report giving: • expected ground conditions • recommended types of foundations • geotechnical design problems needing analysis	
	Ground investigation plan	
		Ground investigation: • profiling • classification • determination of parameters
Detailed structural / architectural design	Detailed geotechnical design	
Construction	Comparison of actual and anticipated ground conditions-assessment of new risks	Additional ground investigation
Performance	Geotechnical monitoring	Instrumentation

Unfortunately, in practice, British site investigation today more closely resembles the dream of engineers working on soil mechanics before World War I. According to Terzaghi (1936):

> engineers imagined that the future science of foundations would consist in carrying out the following program: Drill a hole into the ground. Send the soil samples obtained from the hole through a laboratory with standardized apparatus served by conscientious human automatons. Collect the figures, introduce them into equations, and compute the result.

After a period of optimism between the wars, the inevitable pressures of competitive tendering have reduced the average level of British site investigation to the state where reputable companies, with considerable geotechnical experience and expertise to offer, find financial survival difficult.

The financial pressures faced by British site investigation contractors are inevitable, whilst clients do not understand the value of good ground investigation, and prefer economy to sound engineering.

The major part of the college training of civil engineers consists in the absorption of the laws and rules which apply to relatively simple and well-defined materials, such as steel or concrete. This type of education breeds the illusion that everything connected with engineering should and can be computed on the basis of *a priori* assumptions (Terzaghi 1936).

PLANNING GROUND INVESTIGATIONS

The process of all site investigation should be, above all, one of scientific method. Sufficient factual information should be gathered (from the desk study and walk-over survey) to form hypotheses regarding the ground conditions, and from this and a reasonable knowledge of what is to be built on the site, the problems likely to be encountered both during the construction and the life of the development must be predicted. The design of the proposed construction should then, ideally, take into account the project's geotechnical setting, in order to avoid as many difficulties as possible, and minimize the remainder. Finally, ground investigation should be carried out in order, if necessary, to determine the actual ground conditions on the site, and where necessary to obtain parameters for engineering calculations.

Field investigation, whether by geophysics, or by boring or drilling, must have clearly identified aims if it is to be worthwhile. In some situations it may be necessary to make extensive and detailed ground investigations, but it is also perfectly conceivable that in other situations very few (if any) trial pits or boreholes or soil testing will be required before the start of construction. At present, ground investigation is poorly targeted, and it is because of this that it is sometimes regarded as a necessary but rather unrewarding expense. Yet it must be remembered that the majority of unforseen costs associated with construction are geotechnical in nature. Tyrell *et al.* (1983) carried out an appraisal of 10 UK highway construction projects where cost over-runs were substantial, averaging some 35% of the tender sum. They went through contract records to determine the cause of the additional costs, and found that approximately one-half of the increase in cost could be attributed to just two factors:

1. inadequate planning of ground investigation; and
2. inadequate interpretation of the results of ground investigations.

Because the planning of ground investigation is so important, it is essential that an experienced geotechnical specialist is consulted by the promoter of the project and his leading technical designer very early during conceptual design (see Procurement, below). The planning of a ground investigation is broken down into its component parts in Table 1.2. The geotechnical specialist may be an independent consultant, but more often in the UK will work for a specialist geotechnical consultancy practice, for a - general civil engineering consultancy, or for one of the larger specialist ground engineering contractors. In the UK, the British Geotechnical Society's 1992 *Geotechnical Directory of the United Kingdom* obtainable from the BGS at the Institution of Civil Engineers in London, gives a list of suitable individuals and the companies that employ them. The qualifications and experience required, before an individual may achieve an entry in the Directory, are shown in Table 1.3.

Table 1.2 Planning a ground investigation

Stage	Action	Carried out by
I	Obtain the services of an experienced geotechnical specialist	Developer/client
II	Carry out desk study and air photograph interpretation, to determine the probable ground conditions at the site	Geotechnical specialist
III	Conceptual design: optimize construction to minimize geotechnical risk	Architect, structural engineer, geotechnical specialist
IV	Identify parameters required for detailed geotechnical calculations	Geotechnical specialist
V	Plan ground investigation to determine ground conditions, and their variation, and to obtain geotechnical parameters	Geotechnical specialist
VI	Define methods of investigation and testing to be used	Geotechnical specialist
VII	Determine minimum acceptable standards for ground investigation work	Geotechnical specialist
VIII	Identify suitable methods of procurement professional	Geotechnical specialist, lead design, developer/client

The most important step in the entire process of site investigation is the appointment, at a very early stage in the planning of a construction project, of a geotechnical specialist. At present, much site investigation drilling and testing is carried out in a routine way, and in the absence of any significant plan. This can result in a significant waste of money, and time, since the work is carried out without reference to the special needs of the project.

Table 1.3 Requirements for organizations and individuals to appear in the British Geotechnical Society's Geotechnical Directory of the UK

Organizations	Individuals
For an organization to appear in the Directory it must be active in the UK offering services in geotechnical engineering (as opposed to manufacturing geotechnical equipment, for example). It must also employ at least one person whose name appears as an individual entry in the Directory. Organizations which belong to one of the Trade Associations featured in the Directory are identified in the lists by means of the Association's logo.	For an individual's name to appear in the Directory, he or she must be resident in the UK and be a member of the British Geotechnical Society, the Engineering Group of the Geological Society or a regional Geotechnical Society. He or she must also fulfil one of the three sets of criteria given below. Chartered Engineer through Corporate Membership of the Institution of Civil Engineers, the Institution of Structural Engineers, the Institution of Mining and Metallurgy or be a Chartered Geologist or a Corporate member of an equivalent overseas Institution and a minimum of five years' experience as a practising geotechnical specialist or: a professional qualification, as above and a further degree (a Master's degree or Doctorate) in a relevant subject area, for example soil mechanics, geotechnical engineering, foundation engineering or engineering geology and a minimum of three years' experience as a practising geotechnical specialist or: a minimum of twenty years' experience as a practising geotechnical specialist

Once a geotechnical specialist has been appointed, work can start on determining the ground conditions at the site. The first stages of this process are the desk study, air photograph interpretation, and a site walk-over survey (see Chapter 3). In geotechnical work, descriptions of soil and rock are made in accordance with very specific guidelines (Chapter 2), which have been devised to indicate their performance under engineering conditions, in terms of strength, compressibility and

permeability. If previous site investigation reports exist for construction in the same soil, this allows the geotechnical engineer to judge (albeit in a general way) the likely performance of the ground under and around the proposed development. In any case, geological maps coupled with experience will give a considerable amount of information, of great value in the initial stages of design.

At this stage there should also be interaction between the client and all of his design professionals. Where possible, the design should be modified to reduce possible geotechnical problems. For example, if a large site is to be developed as a business park, the buildings might be re-aligned with their long sides parallel to the contours; this will reduce the amount of cut and fill, thus keeping the cost of foundations and retaining structures to a minimum, while also reducing the risks of slope instability. Structures may be relocated to avoid areas of potentially difficult ground, such as infilled quarries, pre-existing slope instability, or where old foundations or contaminated ground may exist below previously demolished structures. Appropriate foundation types and structural connexions can be chosen.

From a knowledge of the probable ground and groundwater conditions, and the required structural form(s), the geotechnical engineer should predict the types of foundations and earth-retaining structures required on the project, and any possible problems (such as slope instability, chemical attack on foundation concrete, construction difficulties) which can be foreseen, and which may therefore require further investigation. The planning of a ground investigation requires a knowledge both of the ground conditions at and around the site, and of the form of the proposed construction. If the design of the construction is to be optimized, then the form of construction should, as far as possible, take the expected ground conditions into account.

At the end of the desk study, air photo interpretation and walk-over survey, the geotechnical specialist should make a written report, giving he expected ground conditions across and around the site, the uncertainties in these predictions, and the extent of ground investigation proposed for their investigation. In addition, he or she should make proposals for suitable types of foundations for any proposed structures, and should identify areas where other geotechnical structures (such as retaining walls or slopes) will be expected. For these areas there will be a need to obtain geotechnical parameters for design. Other potential problems requiring investigation should also be identified. The parameters to be obtained during ground investigation, and the methods to be used to obtain those parameters, should be described, and justified, in detail.

Planning trial pitting, boring and drilling

Drilling and trial pitting are normally carried out for a number of reasons, such as:

1. to establish the general nature of the strata below a site;
2. to establish the vertical or lateral variability of soil conditions;
3. to verify the interpretation of geophysical surveys;
4. to obtain samples for laboratory testing;
5. to allow *in situ* tests to be carried out; and
6. to install instruments such as piezometers, or extensometers.

Frequently, most if not all of these objectives will control the method of drilling on site. All the objectives must be achieved with the minimum of expense and disruption to occupiers of the site.

In the UK, drilling, sampling and testing are normally carried out by a specialist site investigation contractor. The most convenient method of organizing the work is for the engineer controlling the contract to decide on the position and depth of boreholes, the sampling routine for each soil type that is likely to be found, and the number and type of *in situ* and laboratory tests that are required. A number of contractors can then provide competitive bids, the cheapest price can be selected, and the work carried out.

The scheme described in the preceding paragraph is ideal from the contractual viewpoint, because it allows a fixed price to be obtained by competitive tender. As a method of achieving the aims of site investigation, it is rarely satisfactory, however, because soil conditions are not very well known at tender stage and because competitive tendering favours contractors who have the lowest overheads and are therefore less likely to be able to bring a high level of engineering expertise to bear on the work. When specifying and controlling drilling, it is important that the drilling and testing programme can be modified while work is in progress, as new information is made available by each borehole or test pit. Therefore both office and site staff should be aware of the reasons for the decisions made during the initial planning of the work, in order that they do not hesitate to alter drilling and testing schedules where this is appropriate.

The principal factors which allow a logical drilling programme to be planned and successfully executed are:

1. a relationship between structure, borehole layout, frequency and depth;
2. a need for sample quality and quantity related to the required geotechnical parameters and the soil type and variability;
3. site supervision, to ensure that drilling and sampling are carried out to a high standard and that good records are kept; and
4. prompt sample description and preparation of borehole and pit records in order that the drilling programme can be modified as the work proceeds.

These factors are considered in turn below.

Borehole layout and frequency

Borehole layout and frequency are partly controlled by the complexity of the geological conditions. The complexity of geological structure and the variability of each of the soil or rock units should be at least partially known after the fact-finding or desk study. If soil conditions are relatively uniform, or the geological data are limited, the following paragraphs will give an initial guide. Borehole layout and frequency may need to be changed as more information emerges.

Investigation will normally be carried out by machine or hand-excavated trial pits, where only shallow depths are to be investigated, for example for low-rise housing projects, or for shallow instability problems. The use of pits in these situations allows a detailed engineering description of soil conditions, and will also permit block samples to be taken. Most boreholes will be considerably deeper than can be excavated by an open trial pit, and these will normally be carried out by light percussion or hollow stem auger drilling.

Most projects will fall into one of the following categories:

a) *isolated small structures*, such as pylons, radio masts, or small houses, where one borehole may be sufficient;
b) *compact projects*, such as buildings, dams, bridges or small landslips, will require at least four boreholes. These will normally be deep and relatively closely spaced;
c) *extended projects*, such as motorways, railways, reservoirs and land reclamation schemes will require shallower, more widely spaced boreholes, but these will normally be expected to verify the depth of ‘good’ ground. In the case of road projects this will mean either rockhead, or a soil with a ‘stiff’ consistency. In the case of reservoirs, borings should be continued until an adequate thickness of impermeable ground is found. The frequency of borings on extended sites must be judged on the basis of the uniformity or otherwise of the site geology and its expected soil variability. On a highway project the recommendations for borehole spacing

vary from 30 to 60 m (Hvorslev 1949) to 160 m in changeable soils and 300m in uniform soils (*Road Research Laboratory* 1954).

Many projects, such as highways, are a combination of the categories described above. Structures on extended projects should be treated as compact projects. For example, a typical investigation for a motorway in the UK might use 5—l0m deep borings every 150m along the proposed road line, with four 25—30m deep borings at the proposed position of each bridge structure. Additional boreholes might be placed on the basis of soil information found during the fact-finding survey, on the basis of:

1. the geological succession in the area. Thin beds of limited outcrop may require closer boreholes;
2. the presence of drift deposits such as alluvium or glacial till, whose vertical and lateral extent may require close inspection;
3. problem areas, for example where pre-existing slope instability is suspected.

The layout of the borings should aim not only to provide soil profiles and samples at positions related to the proposed structures and their foundations, but should also be arranged to allow the hypotheses formed during the fact-finding survey to be checked. The borings should be positioned to check the geological succession and to define the extent of the various materials on site, and they should be aligned, wherever possible, in order to allow cross-sections to be drawn (Fig. 1.4). Where structures are to be found on slopes, the overall stability of the structure and the slope must obviously be investigated, and to this end a deep borehole near the top of the slope can be very useful.

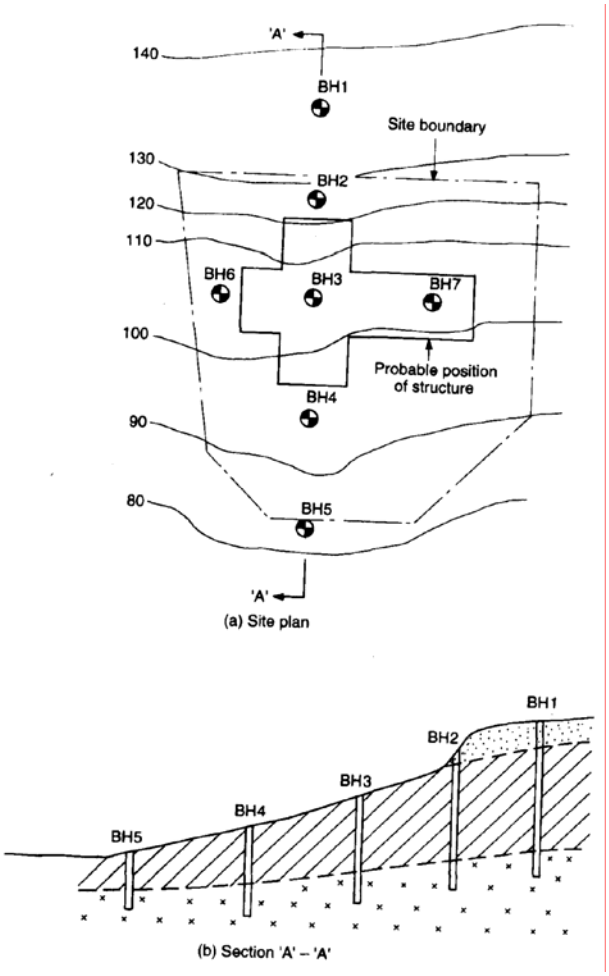


Fig. 1.4 Alignment of boreholes.

Depth of borings

It is good practice on any site to sink at least one deep borehole to establish the solid geology. On extended projects several of these may be necessary, partly in order to establish the depth of weathering, which may be up to 100 m below ground level and may be irregular, and also to establish the depth to which cavernous or mined areas descend.

Hvorslev (1949) suggested a number of general rules which remain applicable:

> The borings should be extended to strata of adequate bearing capacity and should penetrate all deposits which are unsuitable for foundation purposes — such as unconsolidated fill, peat, organic silt and very soft and compressible clay. The soft strata should be penetrated even when they are covered with a surface layer of high bearing capacity.
> When structures are to be founded on clay and other materials with adequate strength to support the structure but subject to consolidation by an increase in the load, the borings should penetrate the compressible strata or be extended to such a depth that the stress increase for still deeper strata is reduced to values so small that the corresponding consolidation of these strata will not materially influence the settlement of the proposed structure.
> Except in the case of very heavy loads or when seepage or other considerations are governing, the borings may be stopped when rock is encountered or after a short penetration into strata of exceptional bearing capacity and stiffness, provided it is known from explorations in the vicinity or the general stratigraphy of the area that these strata have adequate thickness or are underlain by still stronger formations. When these conditions are not fulfilled, some of the borings must be extended until it has been established that the strong strata have adequate thickness irrespective of the character of the underlying material.
> When the structure is to be founded on rock, it must be verified that bedrock and not boulders have been encountered, and it is advisable to extend one or more borings from 10 to 20 ft into solid rock in order to determine the extent and character of the weathered zone of the rock.
> In regions where rock or strata of exceptional bearing capacity are found at relatively shallow depths — say from 100 to 150 ft — it is advisable to extend at least one of the borings to such strata, even when other considerations may indicate that a smaller depth would be sufficient. The additional information thereby obtained is valuable insurance against unexpected developments and against overlooking foundation methods and types which may be more economical than those first considered.
> The depth requirements should be reconsidered, when results of the first borings are available, and it is often possible to reduce the depth of subsequent borings or to confine detailed and special explorations to particular strata.

As a rough guide to the necessary depths, as determined from considerations of stress distribution or seepage, the following depths may be used.

1. *Reservoirs*. Explore soil to: (i) the depth of the base of the impermeable stratum, or (ii) not less than 2 x maximum hydraulic head expected.
2. *Foundations*. Explore soil to the depth to which it will be significantly stressed. This is often taken as the depth at which the vertical total stress increase due to the foundation is equal to 10% of the stress applied at foundation level (Fig. 1.5).
3. For *roads*. Ground exploration need generally only proceed to 2—4 m below the finished road level, provided the vertical alignment is fixed. In practice some realignment often occurs in cuttings, and side drains may be dug up to 6 m deep. If site investigation is to allow flexibility in design, it is good practice to bore to at least 5 m below ground level where the finished road level is near existing ground level, 5 m below finished road level in cut, or at least one-and-a-half times the embankment height in fill areas.
4. For *dams*. For earth structures, Hvorslev (1949) recommends a depth equal to one-half of the base width of the dam. For concrete structures the depth Of exploration should be between one-and-a-half and two times the height of the dam. Because the critical factor is safety against seepage and foundation failure, boreholes should penetrate not only soft or unstable materials, but also permeable materials to such a depth that seepage patterns can be predicted.

5. For *retaining walls*. It has been suggested by Hvorslev that the preliminary depth of exploration should be three-quarters to one-and-a-half times the wall height below the bottom of the wall or its supporting piles. Because it is rare that more than one survey will be carried out for a small structure, it will generally be better to err on the safe side and bore to at least two times the probable wall height below the base of the wall.
6. For *embankments*. The depth of exploration should be at least equal to the height of the embankment and should ideally penetrate all soft soils if stability is to be investigated. If settlements are critical then soil may be significantly stressed to depths below the bottom of the embankment equal to the embankment width.

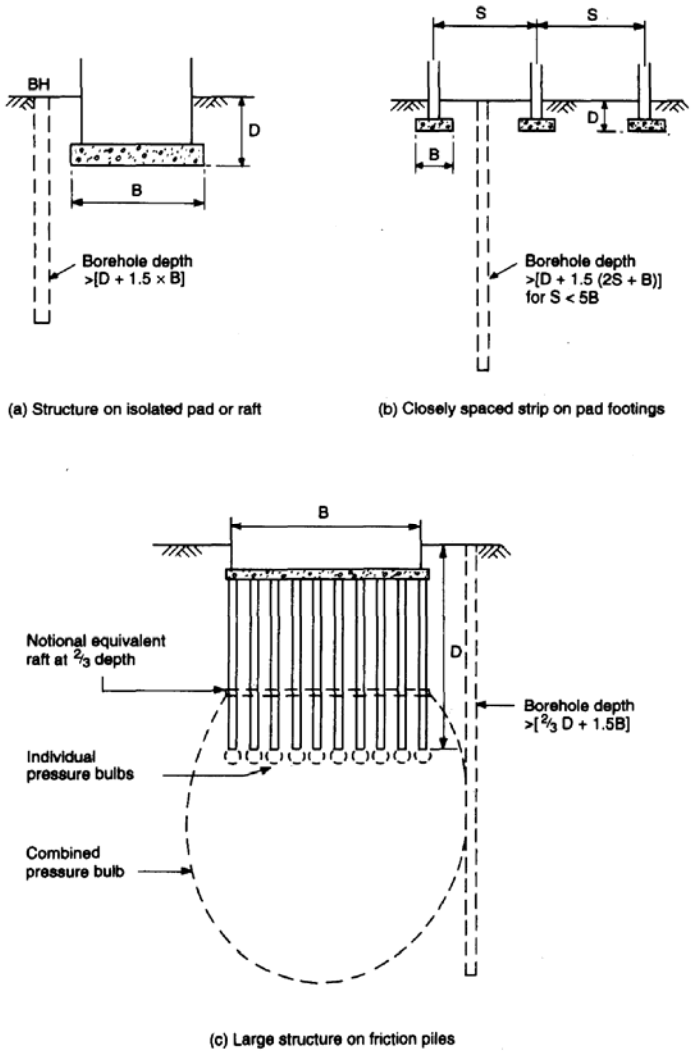


Fig. 1.5 Necessary borehole depths for foundations.

Because many investigations are carried out to determine the type of foundations that must be used, all borings should be carried to a suitable bearing strata, and a reasonable proportion of the holes should be planned on the assumption that piling will have to be used.

Sampling, laboratory testing and in situ testing requirements

As will be seen in the Chapters 6, 7 and 9, which deal with sampling disturbance, sampling techniques, and *in situ* testing, most available sampling and *in situ* testing techniques are imperfect, and often represent a compromise. The normal sampling and *in situ* testing routines in use in the UK, represent the results of just such a compromise. They result from the fact that stiff clays, stoney glacial

tills and gravelly alluvium are so often found in the UK, and that prices for ground investigation are relatively low. In routine ground investigations samples are taken or *in situ* tests made only every 1.5 m down boreholes, and only about 25% of the soil at every borehole location is sampled, however imperfectly. Even in the most intensely investigated site, it is unlikely that more than one part in 1000000 of the volume of ground affected by construction will be sampled.

The sampling routine should be aimed at:

1. providing sufficient samples to classify the soil into broad soil groups, on the basis of particle size and compressibility;
2. assessing the variability of the soil;
3. providing soil specimens of suitable quality for strength and compressibility testing; and
4. providing specimens of soil and groundwater for chemical testing.

Soil and rock are not normally found in pockets, each of a distinct type, but often grades gradually from one soil type (for example, sand) to another (for example, clay). It is therefore necessary artificially to divide the available soil and rock samples into groups, each of which is expected to have similar engineering behaviour. Engineering soil and rock description (Chapter 2), and index tests and classification tests are used for this purpose (Chapter 8).

Geotechnical parameters are obtained by testing specimens which have been selected to be representative of each of the soil groups defined by soil description, and classification and index testing. Where soil grouping cannot be carried out, perhaps because of time or financial constraints, it is often found to be necessary to carry out much larger numbers of the more time-consuming and sophisticated tests required for determining geotechnical design parameters. Therefore this is a false economy.

Thus, if 450 mm long samples are to be taken every 1.0 to 1.5 m down the borehole in cohesive soils, every test specimen should be subjected to determinations of water content and plasticity. Where an undrained shear strength profile is required, tests should be made on every specimen of the appropriate diameter in the depth range required for the profile. For proposed spread foundations, embarkments and temporary works cuttings, these depths should not be less than the height of the cut or fill, or the width of the foundation. If soil conditions are unfavourable, piles may be required; in anticipation of this, shear strengths should then be determined to much greater depths.

Large numbers of undrained triaxial strength tests are required in order to establish a shear strength — depth profile in firm to hard clays, because of the scatter in their results which is induced by fissuring. In the past, it has often been assumed that much smaller numbers of effective strength test results will be needed, because fissuring effects are less important. It now appears that this is not the case. Fissures appear to have little effect on small-strain stiffness, but unfortunately give rise to a large scatter in effective strength parameters (c’ and p’) even when 100mm diameter specimens are used. Current UK practice tends to underestimate the need for a sufficient number of effective stress tests; when long-term slope or retaining wall stability problems must be analysed, at least five sets of tests, each with three specimens, should be made on each soil type. Compressibility tests, normally by oedometer compression, will be required from every specimen within the probable depths of soil to be significantly stressed. Clearly, soil is normally variable, and when a two-stage investigation (a variation survey followed by detailed exploration) is not carried out, the only logical course is to test more extensively those specimens that are obtained.

In the UK *in situ* testing is carried out when:

1. good quality sampling is impossible (for example, in granular soils, in fractured rock masses, in very soft or sensitive clays, or in stoney soils);
2. the parameter required cannot be obtained from laboratory tests (for example, *in situ* horizontal stress);

3. when *in situ* tests are cheap and quick, relative to the process of sampling and laboratory testing (for example, the use of the SPT in London clay, to determine undrained shear strength); and most importantly,
4. for profiling and classification of soils (for example, with the cone test, or with dynamic penetration tests).

The most commonly used test is the Standard Penetration Test (SF1') (Chapter 9), which is routinely used at 1.5 m intervals within boreholes in granular soils, stoney soils, and weak rock. Other common *in situ* tests include the field vane (used only in soft and very soft cohesive soils), the plate test (used in granular soils and fractured weak rocks), and permeability tests (used in most ground, to determine the coefficient of permeability).

Marsland (1986) has stated that:

> the choice of test methods and procedures is one of the most important decisions to be made during the planning and progress of a site investigation. Even the most carefully executed tests are of little value if they are not appropriate. In assessing the suitability of a particular test it is necessary to balance the design requirements, the combined accuracy of the test and associated correlations, and possible differences between test and full-scale behaviour.

The primary decision will be whether to test in the laboratory or *in situ*. Table 1.4 gives the relative merits of these options.

Table 1.4 Relative merits of *in situ* and laboratory testing

In situ testing	Laboratory testing
Advantages	
Test results can be obtained during the course of the investigation, much earlier than laboratory test results	Tests are carried out in a well-regulated environment.
Appropriate methods may be able to test large volumes of ground, ensuring that the effects of large particle sizes and discontinuities are fully represented.	Stress and strain levels are controlled, as are drainage boundaries and strain rates.
Estimates of *in situ* horizontal stress can be obtained.	Effective strength testing is straightforward.
	The effect of stress path and history can be examined.
	Drained bulk modulus can be determined.
Disadvantages	
Drainage boundaries are not controlled, so that it cannot definitely be known whether loading tests are fully undrained.	Testing cannot be used whenever samples of sufficient quality and size are unobtainable, for example, in granular soils, fractured weak rock, stoney clays.
Stress paths and/or strain levels are often poorly controlled.	Test results are only available some time after the completion of fieldwork.
Tests to determine effective stress strength parameters cannot be made, because of the expense and inconvenience of a long test period.	
Pore pressures cannot be measured in the tested volume, so that effective stresses are unknown.	

The ground investigation planner requires a detailed and up-to-date knowledge of both laboratory and *in situ* testing, if the best choices are to be made. Table 1.5 gives a summary of the current situation in

the UK — but this will rapidly become out of date. Whatever is used depends upon the soil and rock encountered, upon the need (profiling, classification, parameter determination), and upon the sophistication of geotechnical design that is anticipated.

Table 1.5 Common uses of *in situ* and laboratory tests

Purpose	Suitable laboratory test	Suitable *in situ* test
Profiling	Moisture content	Cone test
	Particle size distribution	Dynamic penetration test
	Plasticity (Atterberg limits)	Geophysical down-hole
	Undrained strength	logging
Classification	Particle size distribution	Cone
	Plasticity (Atterberg limits)	
Parameter determination:		
Undrained strength, cu	Undrained triaxial	SPT
		Cone
		Vane
Peak effective strength, c' φ'	Effective strength triaxial	
Residual strength, c_r' φ_r'	Shear box	
	Ring shear	
Compressibility	Oedometer	Self-boring pressuremeter
	Triaxial, with small strain measurement	Plate test
	Triaxial consolidation	
Permeability	Triaxial permeability	*In situ* permeability tests
Chemical characteristics	pH	Geophysical resistivity
	Sulphate content	

Geophysics

Geophysical methods (Chapter 4) may be used for:

1. geological investigation, for example in determining the thickness of soft, superficial deposits, and the depth to rock, and in establishing weathering profiles, usually to provide cross-sections;
2. resource assessment, for example the location of aquifers, the delineation of saline intrusion, the exploration of the extent of sand and gravel deposits, and rock for aggregate;
3. detecting critical buried features, such as voids (mineshafts, natural cavities, adits, pipelines) and buried artefacts (old foundations, wrecks at sea, etc.); and
4. determining engineering parameters, such as dynamic elastic moduli, and soil corrosivity.

In some instances (for example, the determination of small-strain stiffness) they may be used in the same way as other *in situ* tests, but generally they are used as a supplement to direct methods of investigation, carried out by boreholes and trial pitting.

The planning of geophysical surveys has been described in detail by Darrocott and McCann (1986). They note that clients have often voiced their disappointment with the results of geophysical site investigation, and note that in their experience the failure of the techniques can usually be attributed to one or more of the following problems:

1. inadequate or bad planning of the survey;
2. incorrect choice or specification of the technique;

3. the use of insufficiently experienced personnel to conduct the survey.

Geophysical surveys should be planned as an integral part of the site investigation. The desk-study information must be available so that the most effective techniques are used, and (as with direct methods of investigation, such as boring and trial pitting) the 'targets' of each part of a geophysical survey must be clearly understood. Table 1.6 shows how a geophysical survey should be planned. This is discussed in more detail in Chapter 4.

Table 1.6 Stages of a geophysical survey as part of a ground investigation

Stage	Action	Details
I	Preliminary meeting between geophysicist and geotechnical specialist	Determine: (a) the precise result(s) expected (the 'targets'); (b) whether geophysical methods can be expected to achieve (a); (c) which technique(s) are likely to be successful; (d) consider cost-effectiveness of geophysics relative to other techniques.
II	Carry out desk study	Determine: (a) ground conditions; (b) groundwater conditions; (c) sources of background interference.
III	Plan geophysical survey	Determine: (a) which techniques are likely to be successful, given the ground conditions, the targets',and the background interference; (b) probability of success with each technique; (c) cost-effectiveness of geophysics relative to other techniques; (d) if geophysics appears possible, chose equipment and plan layout for chosen techniques, and identify suitable personnel.
IV	Carry out geophysical trials	This will only be possible in unusual circumstances.
V	Main geophysical survey	
VI	On-site interpretation	The plan for the geophysical survey (for example, the layout of instruments) may need revision in the light of early data, to improve results
VII	Correlation boreholes	The borehole programme should include holes to allow checking and 'calibration' of the geophysics. If possible these data should be made available to the geophysical survey team during their field work.
VIII	Final interpretation	Final interpretation should be made jointly by experienced geophysicists and geotechnical engineers, drawing together all the data, including that from direct investigation methods.
IX	Reporting	Reporting should include raw data, in electronic form, as well as filtered, processed and interpreted results.
X	Feedback	The success of the work, as found during construction, should be conveyed to the geophysical team.

Specification

As noted in Table 1.2, it is necessary to define, in one way or another, the minimum standards of the work to be carried out during the ground investigation. This is particularly important for all elements of work that are to be procured on the basis of competitive tender, since the specification document is central to the prices offered by contractors when bidding. The principal features of the specification contract documents in common use in the UK are given below.

1. *Entry, access and reinstatement*. Whilst the engineer is responsible for arranging access, the contractor must give sufficient notice of entry. Only agreed access routes to the site of the boreholes can be used, and avoidable damage must be made good by the contractor at his own expense. The contractor must include in his rates for stripping topsoil in the area of the borehole, and for making good damage in the area of the borehole and along the access route. Unavoidable damage to crops and hedges or fences is normally paid for by the client.

2. *Services*. Services are to be located by hand digging a pit 1.5 m deep, where it is thought that service pipes, cables or ducts may be present in the area of a borehole.
   Precautions should be taken to protect field personnel from safety hazards, such as underground electrical cables and gas pipes. Engineers involved in ground investigation should recognize that they are responsible for the safety of those working for them. Public utility companies (gas, electricity, telephone, water, etc.) must be contacted to ensure that, as far as possible, risks to health and safety are properly identified before drilling is started.

3. *Trial pits*. The contractor should excavate trial pits by hand or machine in order that soil can be examined *in situ* and samples taken. The plan area of any such excavation should not normally be less than 2m2. Pits should be kept free from water, where encountered, by pumping. The contractor should supply, fix and remove, on completion, sufficient support to the side of the pits to protect anyone entering the hole. Topsoil should be stripped from the pit area before the start of work and should be stockpiled separately until completion. At the end of work, the pit should be filled with compacted spoil, any surplus being heaped proud over the site and covered with the topsoil. Where pits must be left open overnight the contractor must provide temporary fencing around the excavation.

4. *Boring and drilling*. For light percussion boring the minimum borehole diameter is normally 150 mm, but the contractor is responsible for starting the hole at a sufficiently large size to allow him to complete the hole to the required depth. If he fails to do this, the contractor is responsible for reboring the hole at his own expense.
   Claycutters should not be used in soft alluvial soils, where they may cause significant disturbance ahead of the hole.
   In some specifications the weight of the claycutter (see Chapter 6) has been limited as shown in Table 1.7.
   Shells used for boring in granular soils must not be tight fitting if this causes the soil to blow into the base of the hole. Under these conditions the borehole must be kept full of water at all times, and the shell should have a diameter not more than 90% of that of the inside of the borehole casing.

Table 1.7 Specifications for claycutter

Diameter of boring (mm)	Maximum weight of claycutter and sinker bar (kg)
150	150
200	180

In a document produced by the Association of Ground Investigation Specialists in 1979, it was specified that a shell diameter at least 25mm less than that of the casing should be used (AGIS 1979), and in the current British Standard for the SPT it is a requirement that the outside diameter of the drilling tools should not exceed 90% of the inside diameter of the casing.

The addition of water to borings is variously specified, with some documents preventing the addition of water except in ‘dry granular soils and stiff clays’. In one document the limit for addition of water to clays is fixed by testing the ‘immediate undrained cohesive strength’ with a ‘small field penetrometer’. Water can only be added to the borehole if the strength exceeds 140 kN/m2. Ideally, water should not be added to boreholes when drilling in clays above

groundwater level, whatever their consistency. Once the groundwater table is reached, then rapid drilling in stiff fatty clays may not allow time for swelling to take place. If this action is adopted, the first 1.0 m of the day's drilling should not be sampled as it will have had time to swell as a result of stress relief. In all soils below the water table, the borehole should be kept full of water or drilling mud in order to reduce the effects of stress relief. In very soft or soft soils, this is also necessary to prevent failure of the soil up into the cased borehole. When casing is used, it should never be advanced ahead of the borehole. The bottom of the casing should preferably be kept within 150mm of the bottom of the hole at all times in order to prevent excess loosening of surrounding soil, or the formation of voids.

In soft ground, both light percussion boring and auger boring are normally acceptable in the UK. Washboring or jetting is not permitted.

Rotary core drilling may be carried out by open-holing through soft materials, or by drilling ahead of a soft ground boring which has already been made. Clauses are normally included to point out that the material to be cored may be friable or soft, or may contain mixtures of hard rock with interlayered soft materials. The contractor is normally responsible for selecting equipment which will satisfy the other requirements of the specification (for example, recovery, diameter, etc.). Some specifications require the use of hydraulic feed rigs, which are in almost exclusive use in the UK. The introduction of top drive rigs, however, has the advantage that larger runs can be made without rechucking.

The Bill of Quantities is often arranged so that rotary core payment can be made for open-holing and for the recovery of core. In this case it seems sensible to specify that the contractor shall use the necessary equipment, feed pressures and rates, and run lengths so that 100% recovery in any run can be achieved. Where less than 80% recovery is obtained, payment should be at the rate for open-holing for that run length.

Drilling equipment should in general conform to BS 4019, although sampler barrels other than the double-tube ball-bearing swivel type with knock-on spring core catcher box and face discharge bit should not be discouraged since these may give good results. The minimum core diameter and the depth to which it is to be used should be specified, since the cost of deep larger diameter holes will increase significantly when small highly mobile rigs are in general use. The minimum size in sound rock should be N (or 76mm metric), with H (or 101 mm) used in soft or highly weathered rocks, and P (or 116 mm) used in drift such as stiff clays and glacial till.

The maximum run length should be 3m, but where recovery is reduced to less than 80% the length of the next run is often specified as 1 m. If blocking of the flush ports or loss of flush return is detected at any stage, the barrel and core must be removed from the hole immediately. Clear water is the normally specified flush-fluid in the UK, with bentonite mud sometimes being specified in glacial drift and compressed air being used in soft rocks where water flush causes serious deterioration of the core.

There has been a trend in the UK, in recent years, towards the use of rotary coring to obtain samples of heavily overconsolidated clays, and for this purpose bentonite or polymer muds are sometimes specified.

All boreholes and drillholes should be backfilled and compacted in such a way that subsequent settlement of the backfill is avoided. Under artesian groundwater conditions, special sealing devices may be required.

5. *Sampling*. The contractor is commonly required to take disturbed samples, open-drive samples, piston-drive samples and rotary core from boreholes and drillholes. All samples from

soft ground borings or trial pit excavations should be clearly labelled with the following information:

(i) contract name and reference number;
(ii) reference number of hole;
(iii) reference number of sample;
(iv) date of sampling;
(v) depth of top and bottom of sample below ground level; and
(vi) top (if undisturbed).

In addition to labelling the outside of the sample tube, a similar label, but additionally marked 'top' should be placed inside the top of the tube. All labelling should be protected from the effects of damp and water.

Small disturbed samples are normally specified at the top of each stratum, from between undisturbed samples in fine-grained soils, and from the cutting shoes of all thick-walled open-drive samplers. They should contain not less than 1kg of soil which should, as far as possible, fill an airtight container. Large disturbed samples are normally taken from the test section of borehole used for the SPT (cone) test in gravels and other materials containing coarse particles. Their minimum weight should be 25 kg, although larger samples may be required for specific testing requirements.

Thick-walled open-drive 'undisturbed' samples are standard in firm to very stiff clays in the UK. Most specifications make reference to the British Standard for Site Investigation, and in addition some specify minimum sample tube length (450mm), maximum area ratio (about 25%), inside diameter (100mm) and cutting edge taper ($\nless 20°$). The cutting edge should be sharp and free from burrs. The sample tube and cutting shoe should be free from rust, pitting or burring. The use of oil on the inside of the tube should be limited to the minimum practicable.

Thick-walled open-drive sample tubes should either be jacked into the ground or driven from ground level using a standard penetration test automatic trip hammer. Before lowering the sampler tube down the hole, the bottom of the boring should be cleaned of loose materials. Under extreme circumstances, the use of hand-rotated augering is specified for the 600mm of boring above the sample depth. In order to improve recovery, specifications sometimes require either sampler rotation (if practicable) or a waiting period in order to increase adhesion between the soil and the inside of the sampler tube. Thick-walled open-drive samplers must have a ball valve fitted in the sampler head to prevent the build up of pressure over the sample during the sample drive. This should be kept clean at all times. Flap-type core catchers inserted between the cutting shoe and sample tube are normally only permitted when all else fails. Over-driving should normally be avoided.

The undisturbed sample should be pulled slowly from the soil and brought to the top of the hole. After removing the cutting shoe and the head, disturbed material from the top of the sample should be removed and sufficient soil taken from the base of the tube to allow a 10mm thick wax seal to be placed. The sample should immediately be sealed with at least three brush-coated layers of low melting point microcystalline wax. Following this, an oversize metal foil disc is sometimes specified, which is then covered with further wax. The ends of the tubes should be filled with a damp packing material (sawdust, newspaper or rags), and metal or plastic caps applied. The sample tube should be labelled immediately. In the UK, thick-walled open-drive samples are normally specified at a minimum of one every 1.0m for the first 5 m of drilling, and 1.5 m thereafter.

Piston-drive sampling is normally only vaguely specified, usually for very soft or sensitive soils: 'Equipment shall be of a pattern approved by the Engineer'. If piston samples are required then the equipment should be of the fixed piston type, and samples should be taken

continuously or at 1 m intervals. Clearly, much more stringent specifications are required for sampling sensitive soils, and therefore the following points should be included.

Piston samples should be of the fixed piston type, with an area ratio compatible with their cutting edge angle (see the ISSMFE recommendations (1965) in Chapter 6). The maximum cutting edge angle should be 7°. In alluvial soils the minimum diameter should be 100mm, with a minimum length of 450 mm. The maximum inside clearance should be 0.75—1 .00%, although in very soft and sensitive soils there will be no necessity to include any inside clearance. Where possible, piston samplers should be of a design using short sectional liners made of an inert substance such as plastic or impregnated paper. They may be pushed to the desired sample depth or used from the base of a borehole. During the sample drive the inner (piston) rod must be securely fastened at ground surface so that no downward movement is possible. After sampling, the sampler should be rotated before being carefully brought to the surface. The liners should be removed, immediately labelled, and sealed with wax and push-on caps.

When undisturbed sampling is attempted but no recovery results, the borehole should be cleaned out to the full depth to which the sampler has penetrated, and a fresh attempt to sample should be made immediately. The disturbed soil removed from the borehole should be saved as a large disturbed sample. In some specifications reduced payment is made to the contractor for undisturbed sampling attempts which give samples of less than 100mm length, or if the sample is of no use, provided the contractor is not at fault. When full recovery is not achieved the actual sample length and reason for partial recovery must be recorded.

Rotary core should not be removed from the core-barrel by suspending it from a winch rope and hammering the inner barrel. Corebarrels should be held horizontally whilst cores are extruded using a coreplug by applying a constant pressure, and the cores should leave the barrel and travel on a transparent polythene sheet placed on a rigid plastic receiving channel of approximately the same diameter as the core. After extrusion the core should be sealed in the plastic sheet with waterproof adhesive tape, and bound to the rigid plastic receiving channel. It should then be placed in a corebox such as shown in Fig. 1.6. Wooden spacer blocks should indicate the top and bottom levels of each run.

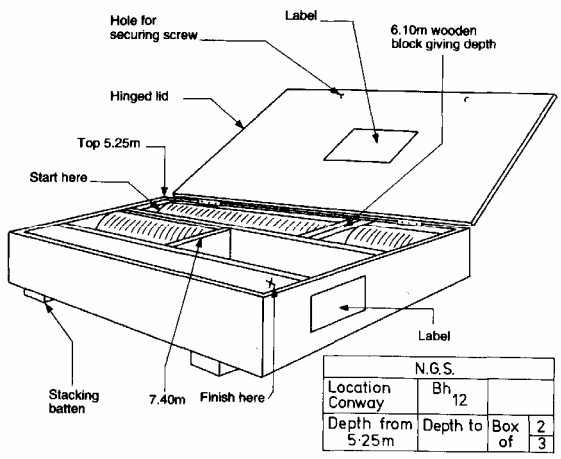


Fig. 1.6 General layout for a corebox (from Redford 1981).

Alternatively, a clear rigid plastic liner (in the UK, sometimes sold under the brand name ‘Coreline’) may be used as a third liner within the corebarrel. This reduces frictional forces between the inner barrel and the core, and in addition allows the core to be withdrawn from the inner barrel whilst it is held in the horizontal position. When a clear rigid plastic liner is used, end caps and plastic tape can be used to protect the core, and coreboxes need not be so carefully made.

In general, it is common to see considerable detail relating to boring and drilling in specification documents, simply because there are currently no national or international standards available for guidance. This is regrettable because, as will be seen in later chapters, drilling technique can have an enormous impact on the quality of samples and *in situ* tests. The advice given in Chapter 7 can be used to make improvements to current specification documents.

In the USA the following American Society for Testing and Materials (ASTM) standards are available for site investigation and sampling:

ASTM D420—87: *Standard guide for investigating and sampling soil and rock*,
ASTM D1452—80: *Soil investigation and sampling by auger borings*,
ASTM D1587—83: *Thin-walled tube sampling*,
ASTM D3550—84: *Standard practice for ring-lined barrel sampling of soils*,
ASTM D2113—83 (reapproved 1987) *Standard practice for diamond core drilling for site investigation*,
ASTM D4220—83: *Standard practices for preserving and transporting soil samples*. In the UK the provisions given in BS 5930 (which in any case describes itself as a code of practice, rather than a standard) for drilling and sampling generally are not suitable for inclusion in a contract specification (see, for example, Clayton (1986) for criticisms).

6. *Groundwater*. The groundwater regime is often not very well determined by ground investigation. Since pore water pressure is usually a very important factor in any engineering calculation, any seepages or inflows into the borehole should be closely monitored. Each time that groundwater is detected, the depth of entry should be measured and the speed of inflow described. Boring should be suspended and groundwater levels observed in an attempt to determine the static groundwater level. Some specifications allow for standing time (i.e. unproductive time) while the groundwater stabilizes in the borehole. Others require that the driller should only suspend work for a maximum of 20 mm. At the end of this period, if the water level is still rising, its depth is to be recorded and drilling recommenced.

   Each groundwater inflow should be sampled. Where water has previously been added for boring purposes, it should be bailed out before sampling. The sample should not be less than 11.

7. *Storage, handling and transporting of soil samples*. All samples and cores should be protected at all times from the adverse effects of weather. They should, as soon as practicable, be placed in a sample store with a humid atmosphere and a temperature between 7 and 18°C. Samples should be handled carefully at all times and should be transported to a soils laboratory for testing within two weeks of sampling.

8. *In situ testing*. BS 1377:1991 and ASTM Part D18 (dealing with Soils and Rocks) give specifications for the most common *in situ* tests, including the SPT, the cone test, vane testing and permeability testing. In addition, German standards and ISSMFE (International Society for Soil Mechanics and Foundation Engineering) Reference Test Procedures are available to cover other forms of testing (for example dynamic penetration testing). Details of these are given in Table 1.8. It is normal in the UK simply to state at the start of a Specification that all the ground investigation work is to be carried out to the British Standards for ‘Site Investigation’ (currently BS 5930:1981) and ‘Testing of Soils for Civil Engineering Purposes’ (currently BS 1377: 1991). It is a much better practice to refer specifically within the Specification to the clauses of required standard dealing with the particular test. British Standards are normally complex, and to avoid omission, specific points to be noted and adhered to by the ground investigation contractor should be highlighted within the Specification document.

Table 1.8 Standards available for *in situ* testing

Test	British Standard	American Standard
Density tests (sand replacement, water replacement, core cutter, balloon, and nuclear methods)	BS 1377:part 9:1990, clause 2	ASTM D1556—82 ASTM D2937—83 ASTM D2167—84 ASTM D2922—91
Apparent resistivity	BS 1377:part 9:1990, clause 5.1	ASTM G57—78 (re-approved 1984)
In situ redox potential	BS 1377:part 9:1990, clause 5.2	
In situ California bearing ratio	BS 1377:part 9:1990, clause 4.3	ASTM D4429—84
Standard penetration test	BS 1377:part 9:1990, clause 3.3	ASTM D1586—84 ASTM D4633—86 (energy measurement)
Dynamic penetration tests	BS 1377:part 9:1990, clause 3.2	
Cone penetration test	BS 1377:part 9:1990, clause 3.1	ASTM D3441—86
Vane test	BS 1377:part 9:1990, clause 4.4	ASTM D2573—72 (re-approved 1978)
Plate loading tests	BS 1377:part 9:1990, clauses 4.1,4.2	ASTM D1194—72 (re-approved 1978) ASTM D4395—84
Pressuremeter test		ASTM D4719—87

9. *Journals*. The information required to form the driller's daily report must be recorded as drilling proceeds. At the end of each day's drilling, the drilling foreman of each rig must prepare a report incorporating the following information:

   (i) job name and location;
   (ii) contractor's name;
   (iii) exploratory hole reference number;
   (iv) depth of drilling at the start and end of the shift;
   (v) type of drilling rig;
   (vi) diameters and depths of all casing;
   (vii) depth of each stratum change;
   (viii) groundwater records;
   (ix) brief description of each soil type; and
   (x) type, diameter and upper and lower depths of each sample, drill run, or *in situ* test;

   for boreholes:

   (xi) locations where water was added to the boring;
   (xii) depths when chiselling was required; and
   (xiii) details of instruments installed;

   for drillholes:

   (xiv) orientation of the drillhole;
   (xv) type and diameter of barrel, and bit; and
   (xvi) flush type, and notes on flush return and loss of return.

These records, produced on standard sheets (Fig. 1.7), should be submitted to the site engineer without fail before the start of the next day's drilling.

Fig. 1.7 Layout for rotary drilling daily report (from Redford 1981).

| NUTTALL GEOTECHNICAL SERVICES LTD. Old Bath Road, Colnbrook, Slough. SL30NJ Colnbrook 3565 | ROTARY DRILLING - FOREMAN'S DAILY REPORT | | |

Table 1.9 Standards available for laboratory testing of soils

Test	British Standard	American Standard
Classification tests		
Moisture content	BS 1377:part 2:1990, clause 3	ASTM D2216—91 ASTM D4643—87
Atterberg limits	BS 1377:part 2:1990, clauses 4,5	ASTM D4318—84
Density	BS 1377:part 2:1990, clause 7	
Specific gravity	BS 1377:part 2:1990, clause 8	ASTM D854—92
Particle size distribution	BS 1377:part 2:1990, clause 9	ASTM D422—63 (re-approved 1972) ASTM D2217—85
Pinhole dispersion test		ASTM D4647—87
Chemical tests		
Organic matter content	BS 1377:part 3:1990, clause 3	
Loss on ignition	BS 1377:part 3:1990, clause 4	ASTM D2974—87
Sulphate content	BS 1377:part 3:1990, clause 5	
Carbonate content	BS 1377:part 3:1990, clause 6	ASTM D4373—84
Chloride content	BS 1377:part 3:1990, clause 7	
pH	BS 1377:part 3:1990, clause 9	ASTM G51—77 (re-approved 1984)
Resistivity	BS 1377:part 3:1990, clause 10	
Redox potential	BS 1377:part 3:1990, clause 11	
Compaction tests		
Proctor/2.5kg rammer	BS 1377:part 4:1990, clause 3.3	ASTM D698—91
Heavy/4.5kg rammer	BS 1377:part 4:1990, clause 3.5	ASTM D1557—91
Vibrating hammer	BS 1377:part 4:1990, clause 3.7	
Strength tests		
California bearing ratio	BS 1377:part 4:1990, clause 7	ASTM D1883—92
Undrained triaxial shear strength	BS 1377:part 7: 1990, clauses 8,9	ASTM D2850—87
Effective strength from the consolidated-undrained triaxial compression test with pore pressure measurement	BS 1377:part 8:1990, clause 7	
Effective strength from the consolidated-drained triaxial compression test with volume change measurement	BS 1377:part 8:1990, clause 8	
Residual strength by direct shear testing in the shear box	BS 1377:part 7:1990, clause 5	ASTM D3080—90
Residual strength using the Bromhead ring shear apparatus	BS 1377:part 7:1990, clause 6	
Compressibility tests		
One-dimensional compressibility in the oedometer	BS 1377:part 5:1990, clauses 3,4	ASTM D2435—90
Isotropic consolidation in the triaxial apparatus	BS 1377:part 8:1990, clause 6	
Permeability tests		
In the constant-head apparatus	BS 1377:part 5:1990, clause 5	ASTM D2434—68 (re-approved 1974)

Table 1.10 Suggested methods for laboratory testing of rocks; ISRM Commission on Testing Methods (formerly The Commission for the Standardization of Laboratory and Field Tests)

Test	Reference in International Journal of Rock Mechanics Mining Science and Geomechanics Abstracts
Description	
Petrographic description	1978, **15**, (2), 41—46
Description of discontinuities	1978, **15**, (6), 319—368
Index tests	
Water content, porosity, density, absorption-related properties, swelling and slake durability	1979, **16**, (2), 141—156
Point load strength	1985, **22**, (2). 51—60
Hardness and abrasiveness	1978, **15**, (3), 89—98
Sound velocity	1978, **15**, (2), 53—58
Mechanical properties	
Uniaxial compressive strength and deformability	1979, **16**, (2), 135— 140
Strength in triaxial compression	1978, **15**, (2), 47—52 revised 1983, **20**, (6), 283—290
Tensile strength	1978, **15**, (3), 99—104
Fracture toughness	1988, **25**, (2), 71—96
Laboratory testing of argillaceous swelling rocks	1989, **26**, (5), 415—426
Large-scale sampling and triaxial testing of jointed rock	1989, **26**, (5), 427—434

Cost considerations

Most published and unpublished opinions on the methods of control and finance of site investigations in the UK express the need for more time and money (Williams and Mettam 1971; Rowe 1972), flexibility of procedure (Green 1968), and adequate liaison between geotechnical and structural design teams (Bridge and Elliott 1967).

Site investigation in the UK traditionally has been carried out by specialist geotechnical contractors. These contractors vary considerably. They may be very experienced organizations controlled by qualified engineers and geologists, and supported by extensive facilities for air photograph interpretation, geotechnical laboratory testing, and computer studies: often, however, they may be organizations with limited assets, limited plant, limited engineering knowledge — and limited liability!

In recent years, the financial restrictions on site investigation seem to have become tighter, but in 1972 Rowe delivered the following arguments in favour of spending more on site investigation.

1. It is known that more claims by piling contractors arise due to poorly or inaccurately known ground conditions than to any other cause (Tomlinson and Meigh 1971).
2. Site investigation costs are very low compared with the cost of earthworks or foundation construction, and even smaller as a proportion of the total capital cost of the works, can be seen in Table 1.11.

These figures represent a decline in expenditure since the 1940s since Harding (1949) reckoned the cost of site investigations for ‘fair-sized works’ to be usually about 1 to 2% of the cost of the main work.

The influence of incorrect site investigation data on the final cost of a project is difficult to assess but can be very large. Rowe cites examples of a case where the omission or inclusion of sand drains could make a difference of 2 to 5% of total project cost, and where unnecessary foundation treatment added 5% to the cost of the works. These figures are certainly not representative of the upper end of the spectrum, as claims for unforeseen ground conditions can easily amount to 10% of contract value.

Table 1.11 Site investigation costs (from Rowe (1972))

Type of work	% of capital cost of works	% of earthworks and foundation cost
Earth dams	0.89—3.30	1.14—5.20
Embankments	0.12—0.19	0.16—0.20
Docks	0.23—0.50	0.42—1.67
Bridges	0.12—0.50	0.26—1.30
Buildings	0.05—0.22	0.50—2.00
Roads	0.20—1.55	(1.60)?—5.67
Railways	0.60—2.00	3.5
Overall mean	0.7	1.5

As noted earlier, Tyrrell *et al.* (1983) found, in an analysis of ten selected highway contracts, that additional expenditure rose to an average of 35% of the tender value. Of this about one-half could be attributed to geotechnical matters. On this basis it is easy to argue for an increase in expenditure on site investigation. But it has proved difficult to establish that increased expenditure on site investigation leads to reductions in construction cost over-runs. What is required is that all expenditure on ground investigation sitework and testing, which typically amounts to 60—70% of the total cost of a site investigation, should be carefully targeted at giving information required for particular and well-defined geotechnical problems. This will lead to reductions in expenditure in some cases, and increases in others.

PROCUREMENT

In the UK it has been widely considered that procurement, in its broadest sense, is the key to obtaining a good site investigation at a reasonable price. Investigations carried out for the Construction Industry Research and Information Association (CIRIA) have been reported by Uff and Clayton (1986, 1991). Many of their more general recommendations are incorporated into the earlier parts of this chapter; only those dealing with the detailed mechanisms of procurement are considered below.

The way in which ground investigation work is organized has been described briefly above, under ‘Implementation’ and ‘Planning’. A number of different organizational models are used worldwide, as can be seen from the examples given in Fig. 1.8. In essence, a good system of procurement will ensure that key elements of work are carried out properly, by competent personnel. In Fig. 1.8, examples A, B and C are satisfactory; D and E omit significant parts of the investigation process, and are bad. It is essential that:

1. desk study, air photo interpretation and a walk-over survey are carried out;
2. the ground investigation is properly designed, taking into account the probable ground conditions and the proposed construction;
3. the required type and standards of ground investigation field and laboratory work are properly defined;
4. during the ground investigation, standards are enforced by competent supervision; and
5. as ground investigation proceeds, the ground conditions are reassessed in the light of information emerging from the work, and that work is rescheduled if necessary.

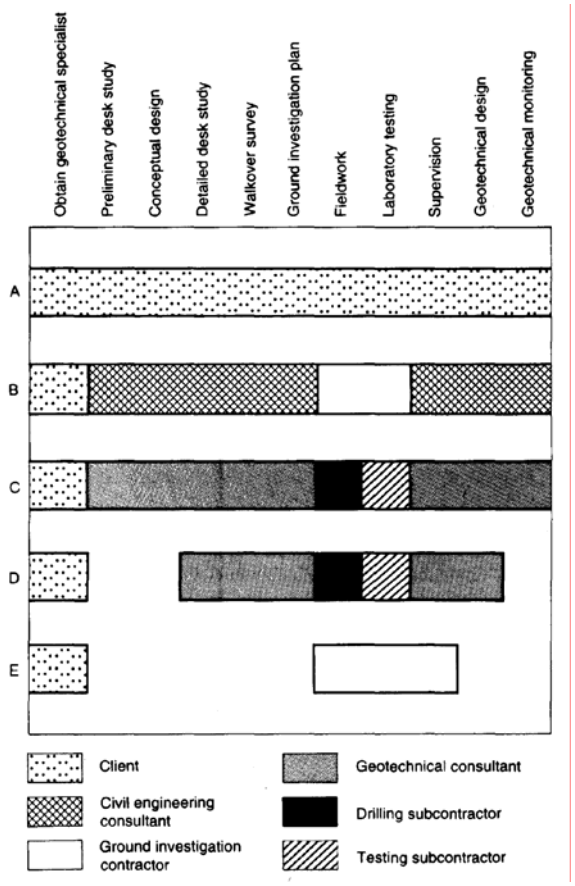


Fig. 1.8 Examples of division of site investigation work.

Procurement methods will often concentrate upon obtaining minimum prices, without considering how well the required quality of ground investigation work can be defined. This is a serious mistake, since many of the activities involved cannot very readily be checked. For example, a good quality standard penetration test requires attention not only to the test equipment and the method used for the test itself, but also to the method of boring, and the water levels within the borehole, both before and during the test. The end product is a series of numbers, the validity of which can be known only if all these matters have been observed, reported, and considered.

Therefore, it is suggested that the procurement system should aim:

1. to ensure that a competent geotechnical adviser is retained by the promoter/ developer at an early stage during the conceptualization of the project, in order to guide the project;
2. as far as possible, to give overall responsibility for all geotechnical matters to a single individual or company; and
3. to select geotechnical advisers and contractors on the basis of their resources (staff, equipment, etc.), and experience with similar forms of construction and ground conditions, and not primarily on the basis of their fee level or unit rates.

In the UK, two systems of procurement of site investigation are in common use, as detailed in *CIRIA Special Publication* SP45 (Uff and Clayton 1986).

System 1: Use of a geotechnical adviser with the separate employment of a contractor for physical work, testing, and reporting as required

In this system the desk study, the planning and supervision of any fieldwork (such as boring, drilling, trial pitting or *in situ* testing) and laboratory testing work that may be necessary is carried out by the

geotechnical adviser. He will often be a member of a firm of civil engineering consultants but may also be a specialist geotechnical consultant.

This system is widely used on large civil engineering projects. The geotechnical adviser will normally be employed by the developer under the Association of Consulting Engineers Conditions of Engagement, while the specialist ground investigation contractor will be chosen by competitive tender and will work under ICE Conditions of Contract. Two versions of ICE contract are in use; the *ICE 5th Edition* and the *ICE Conditions of Contract for Ground Investigation*. When using this system it is important that the developer or his advisers should check that the chosen geotechnical adviser has sufficient geotechnical skill to carry out the desk study, plan and supervise the ground investigation and interpret its results. It is possible to make use of the contractor's engineering skills only after the tendering process. Therefore the skills of the geotechnical adviser are extremely important.

The geotechnical adviser is expected to carry out a thorough desk study and plan an investigation appropriate to the needs of the developer. This is then used to prepare a specification and bill of quantities which, together with the conditions of contract, form the basis of the tender for the field and laboratory work to be carried out by a specialist contractor. Generally between three and four companies should be selected by the geotechnical adviser to tender for the field and laboratory work, on the basis of their previous experience of this type of work, the skills of their staff and the amount and quality of their equipment. The lowest submitted tender price is generally accepted but the contract is subject to remeasurement as the work proceeds. The final cost to the developer of the entire ground investigation will be the sum of the final contract price after measurement and the professional fees paid to the consulting engineer.

This system has been found to work well provided that:

1. an adviser with a sufficient number of skilled geotechnical staff is engaged;
2. a thorough desk study, made by the geotechnical adviser, is used as the basis for the planning of any programme of drilling and testing;
3. not more than four specialist contractors arc asked to tender and the selection of these companies is rationally and thoroughly carried out; and
4. proper levels of supervision are provided by the geotechnical adviser in the control of field and laboratory work. Supervision is the key to the successful use of System 1.

In certain cases it may be advantageous for parts of the work to be done by the contractor on a dayworks basis. Under System 1, the work to be carried out by the contractor must be closely defined before the contract is let and must be paid for at fixed rates independent of the time taken to carry it out. If the work is particularly important to the success of the investigation, if it is very complex, or if the geotechnical adviser needs to be able to vary the work as it proceeds, dayworks payments may be helpful. For example, dayworks could be used to pay for plate loading tests, for drilling and boring in key zones, or for time spent in investigating groundwater conditions. It is also possible to pay a specialist contractor to carry out the reporting of an investigation; this is better done on an hourly basis rather than by lump sums.

System 1 has the advantage of using forms of contract that are well known in the civil engineering construction field and it can be used to demonstrate cost-accountability through the tendering process. This is the most commonly used form of procurement for larger ground investigations and is therefore well understood. Its difficulties lie in the complexity of its contractual arrangements, the need to ensure that sufficient expertise and supervision are provided by the geotechnical adviser and the division of responsibility for the satisfactory outcome of the investigation between the geotechnical adviser and the contractor. It has frequently been said that the method of competitive tendering commonly associated with this system, and the consequent low prices paid to contractors for investigation work, is a major cause of low-quality investigation. This problem, however, is a consequence of too large tender lists prepared without detailed selection of tenderers. It is not necessarily a result of using the system.

System 2: Package deal contract, with desk study, planning and execution of field and laboratory work, and reporting, being carried out by one company or a consortium

No formal conditions of contract exist for this system, although draft documents have been proposed in *CIRIA Special Publication 45*. Despite the lack of published conditions of contract, versions of this system are in common use to obtain ground investigations for low-rise building development. The system is also used for large site investigation contracts carried out abroad, for example in the Middle East.

In this system the developer selects up to three specialist ground investigation companies on the basis of past experience, reputation, and published information relating to specialists in the field. Information on companies and individuals is available from:

- the British Geotechnical Society;
- the Association of Ground Investigation Specialists;
- the Institution of Civil Engineers; and
- the Geological Society.

The companies selected may be either 'contractors' or 'consultants' according to the British Geotechnical Society's Directory, but they should have sufficient qualified and experienced staff to be able to carry out the proposed size of investigation. On the basis of a preliminary desk study, the companies offer to carry out a complete site investigation, including desk study, air photograph interpretation, design and execution of ground investigation and reporting, either for a lump sum or on the basis of measurement of work agreed as the investigation progresses. The specialist company that carries out the work is expected to supervise its own drilling and testing and will be liable under the 1982 Supply of Goods and Services Act both for the quality of work and for any recommendations that are made in the report of the investigation.

The advantages of System 2 to a developer are that a lump sum contract can be negotiated; this is obviously important when carrying out financial forecasting. A further advantage is that the responsibility for ground investigation is not divided between two parties, as in System 1. Because of the cost to the tenderers of preliminary desk studies, it is unlikely that lump sum contracts can be used for very large civil engineering projects, but this type of procurement will certainly be more suited than System 1 to many low-rise building developments, because of its relatively simple contract documentation and its flexibility.

An advantage of this system is that the leading design professional (who might typically be an architect in the case of a low-rise building development) is not necessarily required to have geotechnical skill and experience of ground investigation techniques. If he does not possess such skill, however, it becomes extremely important that care is taken in the selection of ground investigation specialists who are suitable for the complexity of work to be carried out. A possible disadvantage of System 2 is the lack of well-tried and proven contract documentation. However, this does not appear to have prevented the successful use of this method of procurement in recent years. To overcome this it is suggested that the contract documents used are those given in the appendices to *CIRIA Special Publication 45*.

EXECUTION

Supervision

A good site investigation is made in the field. Engineering excellence, sophisticated laboratory

techniques and the use of powerful computational methods cannot ever be expected to make any contribution to a site investigation performed by bad drillers without engineering supervision. Since this often occurs, it is hardly surprising that the value of site investigation is sometimes questioned by engineers not familiar with its techniques.

Supervision of any site investigation requires an engineer who is familiar with:

1. the techniques of investigation; and
2. the objectives of the particular investigation.

This engineer is the key person in ensuring that the best use is made of the expenditure on site investigation, and to this end he must spend a very large part of his time on site during the investigation. While on site, the supervising engineer should:

1. closely watch the drilling and sampling techniques, to ensure that disturbance of soil is minimized; and the techniques and equipment comply with the specification;
2. frequently check the records of borings provided by the drillers for authenticity and accuracy;
3. carry out sample description and prepare engineering logs, except on small investigations where it will be more economical to transport samples to a laboratory for description;
4. liaise with the structural design engineer, so that the investigation can be modified as a result of its initial findings;
5. ensure that driller's and engineer's borehole records, and the samples are despatched to the soil laboratory at frequent intervals;
6. provide conditions of storage for the samples on site which 'will not lead to their deterioration; and
7. check the adequacy of sample sealing by the rig foremen.

Quite clearly, it will be difficult for one man to supervise more than one drilling rig satisfactorily, and for this reason drilling technicians have sometimes been used. A drilling technician will be assigned to one rig, and will be responsible for all the technical aspects of that rig's work. He would normally, for example, prepare the records of drilling, instruct the driller at which level to take samples, and carry out *in situ* testing and the installation of instrumentation.

In the majority of cases, both in the UK and overseas, drilling technicians are not used. Site investigations are usually carried out by drillers who do not understand the mechanisms of disturbance of soil samples, who are not informed of the objectives of the individual investigation, and who are often motivated solely by productivity bonuses. Under these conditions, the supervising engineer is the only force available in the struggle to produce a sound investigation. To be effective, the supervising engineer must understand the practical aspects of drilling.

The key points in checking the effectiveness of a site investigation are as follows.

1. *Avoid excessive disturbance*. Look for damaged cutting shoes, rusty, rough or dirty sample barrels, or badly designed samplers. Check the depth of casings to ensure that these never penetrate beneath the bottom of the borehole. Try to assess the amount of displacement occurring beneath power augers, and prevent their use if necessary.
2. *Check for water*. Ensure that adequate water levels are maintained when drilling in granular soils or soft alluvium beneath the water table. The addition of water in small quantities should be kept to a minimum, since this allows swelling without going any way towards replacing total stress levels. Make sure the driller stops drilling when groundwater is met.
3. *Check depths*. The depths of samples can be found approximately by noting the number of rods placed on the sampling tool as it is lowered down the hole, and the amount of 'stick-up' of the last rod at the top of the hole. This type of approach is often used by drillers, but is not always satisfactory. Immediately before any sample is taken or *in situ* test performed the depth

of the bottom of the hole should be measured, using a weighted tape. If this depth is different from the last depth of the drilling tools then either the sides of the hole are collapsing, or soil is piping or heaving into the base. Open-drive sampling should not then be used.

4. *Look for faulty equipment*. On-site maintenance may lead to SPT hammers becoming non-standard, for example owing to threading snapping and the central stem being shortened, giving a short drop. When working overseas with subcontract rigs the weight of the SPT hammer should also be measured. Other problems which often occur are: (i) the blocking of vents in sampler heads; and (ii) the jamming of inner barrels in double tube swivel-type corebarrels.
5. *Examine driller's records regularly*. The driller should be aware that the engineer is seeking high quality workmanship. One of the easiest ways of improving site investigation is to demand that up to the moment records are kept by the driller as drilling proceeds. These should then be checked several times a day when the engineer visits the borehole. Any problems encountered by the driller can then be discussed, and decisions taken.

Safety

Safety should be of major concern during the fieldwork and laboratory testing phases of ground investigation. Potential hazards include:

1. incapacity as a result of prolonged exposure to bad working conditions (for example, deafness as a result of exposure to high levels of noise);
2. injury or death as a result of misuse of plant and equipment (for example, using frayed winch ropes, not setting up drilling equipment in a stable configuration, etc.);
3. injury or death as a result of contact with overhead electricity cables (particularly by contact with drilling rig masts, but also with cranes during transporting);
4. injury or death as a result of excavation through services (electricity, gas, water, etc.), during boring, drilling or pitting;
5. injury or death as a result of explosions of gases emanating from the ground (for example methane from landfill);
6. injury or death as a result of collapse of trenches on to personnel carrying out logging or sampling;
7. damage to health as a result of contact with contaminated ground, or in the laboratory, working with contaminated samples;
8. poisoning, as a result of inhaling or ingesting toxic gases or substances such as asbestos, cyanides, etc;
9. damage to health, or death, as a result of radiation; and
10. damage to health, or death, as a result of contact with animal carcases or sewage (leading, for example, to anthrax or Weil's disease).

Whilst many of these risks are associated with the investigation of contaminated land, a very large proportion may be present during any site investigation. Under the UK 'Health and Safety at Work etc.' Act, all persons involved with an investigation have a responsibility to see that safe working practices are adopted. This includes the promoter of the development, who may have knowledge of previous site use, the consulting engineer, who must ensure that sufficient resources are devoted to a safety assessment before field and testing work is specified, and the specialist ground investigation contractor, who must enforce safe working practices during the ground investigation.

Engineers and geologists will be particularly responsible, since they will be directly in control of those most exposed to risk. The construction industry has a poor safety record, and there is always the temptation to reduce costs by taking short cuts with safety. This must be prevented. To help in the drive for greater safety in ground investigation the British Drilling Association have recently published two reports:

Code of Safe Drilling Practice, British Drilling Association, Brentwood, Essex, UK (March 1992).

Guidance Notes for the Safe Drilling of Landfills and Contaminated Land, British Drilling Association, Brentwood, Essex, UK (March 1992).

It is recommended that all professionals involved in ground investigation should study both these and the literature to which they refer, before carrying out fieldwork.

Quality assurance

Quality assurance is 'All those planned and systematic actions necessary to provide adequate confidence that a product or service will satisfy given requirements of quality'. In other words, quality assurance concerns the management of an organization to meet agreed quality objectives. In itself it does not guarantee that a service is of the necessary quality for a given job, but attempts to meet predetermined standards by approaching the work in a systematic manner. In this sense it simply represents good management practice.

In the UK, quality systems are now being implemented in the ground investigation industry. They are standardized internationally (ISO 9001—1987), in Europe by CEN (EN 29001—1987), and in the UK (BS 5750:1987) under identical documents.

Quality systems comprise several levels of activity (Fig. 1.9):

1. *Quality policy*. The overall quality intentions and direction of an organization as regards quality, as formally expressed by top management.
2. *Quality management*. That aspect of the overall management function that determines and implements the quality policy.
3. *Quality system*. The organizational structure, responsibilities, procedures, processes and resources for implementing quality management.
4. *Quality control*. The operational techniques and activities that are used to fulfil requirements for quality.

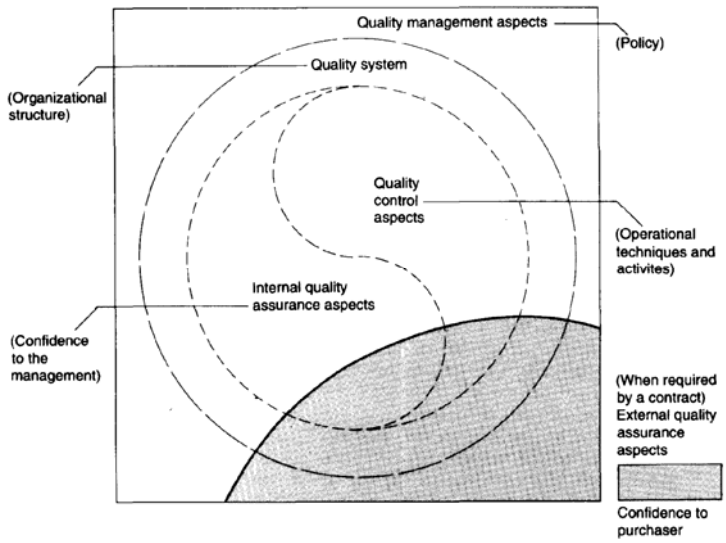


Fig. 1.9 Relationships of quality concepts (BS 5750: part 0: section 0.1: 1987).

In the UK, ground investigation industry quality assurance is being applied at two levels. First, 'internal quality assurance', which aims to provide the management of an organization with the

confidence that the intended quality is being achieved, is being implemented. Under BS 5750, quality systems can be audited by a third party 'certification' body, such as Lloyds or the British Standards Institution. Since ground investigation generally has a rather short duration, it is sensible that at the outset it is the supplier who leads the quality process — it has been found that attempts to impose 'external quality assurance', i.e. activities which aim to provide confidence that the supplier's quality system will provide a product that will satisfy the client's stated quality requirements, are difficult to set up in the absence of legislation, for such small and diverse projects.

Secondly, laboratory testing services are becoming subject to third-party accreditation by the National Measurement Accreditation Service (NAMAS). This represents less of a problem, in theory, because British Standards provide tight specifications for most aspects of the more commonly used tests. At the time of writing (1992) the UK's Department of Transport has stated that all work carried out on UK highway investigation after April 1993 must be carried out in NAMAS accredited laboratories. Presently there is only one major geotechnical laboratory accredited for a wide range of soil tests, although it is expected that several will be able to offer an accredited service by the end of 1992.

In procuring the services of geotechnical specialists, whether consultants, contractors, or specialist sub-contractors, it is recommended that, other things being equal, those who offer a certified quality management system, or an accredited laboratory or field testing service should be favoured.

Chapter 2

Description and classification of soils and rocks

INTRODUCTION

From an engineering viewpoint, the ground beneath a site can conveniently be divided into the categories shown in Table 2.1, which are based upon generalizations of its expected behaviour in construction works.

These broad generalizations are, of course, limited in accuracy. But they give the geotechnical engineer a good basis on which to consider, at the start of a project, both the likely construction problems and the methods of investigation which might be used. In practice, it is found that the ground varies continuously beneath a site, and it is not often possible to find sharp transitions from one type of material to another. This then, calls for more refined, systematic, description and classification of soils and rocks.

The history of soil classification and description has been described by Child (1986). Early attempts to divide soil into different categories used laboratory testing, typically particle size distribution. Casagrande (1947) considered that much of this 'textural soil classification' was unreliable, because it did not always reflect the effects of silt and clay on the engineering behaviour of the mass. On the basis of his experience he considered that only the Airfield Classification System, which was based on the results of particle size analysis and plasticity (Atterberg limit) tests (see Chapter 8), was suitable for an assessment of soil for airfield pavement design. It was noted that, with experience, soil might be classified on the basis of visual/manual description alone; Casagrande therefore suggested what he termed 'descriptive soil classification'.

American practice therefore developed in two directions. Soil Classification, based on Casagrande's Airfield Classification System, became standardized (ASTM D2487— 92). As originally proposed, the system is based solely on particle size distribution and plasticity tests, and soils are designated by letters alone, depending upon which group they fall into. At the same time, a soil description system has been developed, based upon visual examination and simple land tests (ASTM D2488—69).

Table 2.1 Categories of ground beneath a site

Material type	Strength	Compressibility	Permeability
Rock	Very high	Very low	Medium to high
Granular soil	High	Low	High
Cohesive soil	Low	High	Very low
Organic soil	Very low	Very high	High
Made ground	Medium to very low	Medium to very high	Low to high

Early British practice was summarized in Cooling *et al.*'s discussion of Casagrande's 1947 paper. Almost without change, this appeared in the first British Code of Practice on Site Investigations, Code of Practice No. 1 (1950). It was republished as Table 1 of CP 2001:1957. Soil description was based on estimated mass engineering behaviour, and used simple visual description and hand tests. A classification system based on Casagrande's work was included for roads and airfields work, that is to assess the behaviour of materials during compaction and under pavements. Subsequent development

occurred primarily through work by Akroyd (1957), Dumbleton (1968, 1981), Dumbleton and Nixon (1973), and the Engineering Group Working Party on 'The Preparation of Maps and Plans in Terms of Engineering Geology' (Geological Society of London 1972). BS 5930:1981 contained two parts, a soil description based on mass behaviour, and the British Soil Classification System (BSCS) based on the work of Dumbleton (1968, 1981).

SOIL AND ROCK DESCRIPTION

> Soil and rock description is to a certain degree subjective. In order to minimise the subjective element a systematic examination should be carried out using a standard terminology, whether the material be in a natural exposure, trial pit face or samples recovered from a borehole.
>
> The use of a standardised scheme of description ensures that:
>
> (i) all factors are considered and examined in logical sequence
> (ii) no essential information is omitted
> (iii) no matter who describes the sample, the same basic description is given using all terms in an identical way
> (iv) the description conveys an accurate mental image to the readers
> (v) any potential user can quickly extract the relevant information.
>
> *Norbury et al., 1986*

The engineering description of the ground conditions beneath a site is a progressive exercise which at each step involves further departure from strictly factual description, and thus an increased interpretative element. Three steps are involved.

1. The description of individual samples from a borehole, each sample being described in isolation and in completely factual terms, noting any disturbance or obvious loss of material caused by sampling. Any two geologists or engineers with sufficient and comparable experience should produce almost identical descriptions.
2. The combination of these individual descriptions to form a stratum description on the borehole log. In so doing, the engineer or geologist will take into account the information on the ground conditions, depths to strata changes, groundwater levels, field and laboratory test results, etc., given on the driller's daily record sheets. Interpretation is necessary, and so, as Norbury *et al*. have stated 'strictly, there is no such thing as a "factual" borehole log'.
3. The drawing together of individual borehole, trial pit and exposure records, to arrive at an assessment of the mass properties of the various strata, their geometric distribution, and their variability, in a summary in the text of the ground investigation report.

In this process the skills of soil and rock description, coupled with experience, are paramount.

SOIL DESCRIPTION

The description of soil is currently covered, in the UK, by BS 5930:1981. At the time of writing BS 5930 is under revision. The system of soil description given below therefore does not follow completely the code, but takes into account both current good practice, and changes which have been proposed.

Samples must be described in a routine way, with each element of the description having a fixed position within the overall description:

a) consistency or relative density;
b) fabric or fissuring;

c) colour;
d) subsidiary constituents;
e) angularity or grading of principal soil type;
f) PRINCIPAL SOIL TYPE (in capitals);
g) more detailed comments on constituents or fabric;
h) (geological origin, if known) (in brackets); and
i) soil classification symbols (optional).

Descriptions should be simple, since very detailed comments on all aspects of a soil lead to confusion. Some examples are given below:

Very stiff fissured dark grey CLAY (London clay)
(a) (b) (c) (f) (h)

Loose brown very sandy subangular coarse GRAVEL with pockets of soft grey clay
(a) (c) (d) (e) (f) (g)

Firm laminated brown SILT and CLAY
(a) (b) (c) (f) (f)

Soil types

In routine soil description, the material being considered is first placed into one of the *principal soil types* in Table 2.2.

Table 2.2 Principal soil types

Soil type	Description
CLAY	Cohesive soil
BOULDERS	Granular soils
COBBLES	Granular soils
GRAVEL	Granular soils
SAND	Granular soils
SILT	Granular soils
PEAT	Organic soil
MADE GROUND	Man-made soils and other materials

Most soils will be composed of a variety of different particle sizes, some of which may be cohesive. Whilst *classification* is concerned only to determine the proportion by weight of each constituent, *description* is carried out to ascertain probable engineering behaviour. In this sense, we adhere to the proposals of CP 2001 (1957) and reject BS 5930:1981, following Norbury *et al*. (1986) (Table 2.3).

Table 2.3 Comparison of CP 2001 and BS 5930

CP 2001	BS 5930
Soils possessing cohesion and plasticity are described as fine soils, although the majority of the soil by weight may be coarse or very coarse soil. It is not possible to give a percentage of clay and/or silt above which they become the principal component, since the mass behaviour depends on the mineralogy of the soil particles. The description is based on engineering judgement.	Soils with more than 35% clay and/or silt are described as either clay or silt. Soils with less than 35% are described in terms of coarse or very coarse soils, irrespective of whether they have cohesion and plasticity. The description is therefore based on the particle size distribution, but the division between silt and clay is strictly on the Atterberg limits. These factors can be difficult to assess visually for some materials and laboratory tests are required to confirm descriptions.

The fundamental difference between these two proposals is that under the BS 5930:1981 proposals, many 'clays' i.e. materials of low strength, very low permeability and high compressibility, must be termed silts, sands or gravels. In practice the addition of about only 12% by weight of clay is required to make a well-graded granular material perform in engineering works as a cohesive soil. Therefore the stance of BS 5930:1981 is considered untenable.

The first stage of the description process is the identification of the principal soil type, on the basis of the expected behaviour of the soil mass. In soils where the granular fraction dominates behaviour (termed '*granular soils*'), the principal soil type is identified on the basis of a particle size.

As an aid to visual identification, it should be noted that coarse silt represents the normal limit of resolution of individual grains with the unaided eye. The principal soil type is the (single) component of the soil (i.e. boulders, cobbles, gravel, sand or silt) which is thought to represent (in a coarse soil) the greatest proportion by weight (Table 2.4). Where two types are thought to be equal, two components may be given (for example, sand and gravel, boulders and cobbles). BS 5930:1981 rightly noted that the properties of very coarse soils (i.e. boulders and cobbles) cannot be reliably estimated from boreholes-trial pits or exposures must be used.

Table 2.4 Identification of principal soil type

Principal soil type	Particle size (mm)		Particle size (mm)
Boulders	>200		
Cobbles	60—200		
		coarse	20—60
Gravel	2—60	medium	6—20
		fine	2—6
		coarse	0.6—2
Sand	0.06—2	medium	0.2—0.6
		fine	0.06—0.2
Silt	<0.06		

In soils where the cohesive fraction dominates the engineering behaviour (termed 'cohesive soils'), the soil is described as *clay*. BS 5930:1981 differentiates between silts and clays on the basis of their position relative to (i.e. above or below) Casagrande's 'A' line — see later. Since some pure clay minerals plot below the A-line (see Dumbleton and West (1966), for example) and some silts plot above the A-line (Clayton 1983) this approach is not considered reasonable. Silts are fine-grained non-cohesive soils — as such they have relatively good effective strength, compressibility and drainage properties, from an engineering point of view. The separate identification of silts and clays can therefore most easily be made on the basis of dry strength. Silts have very low to low dry strengths, whilst clays have medium to very high dry strengths (see, for example, ASTM D2488).

Peat is the name given to soil which is primarily composed of plant matter, usually found decomposing in mires and fens. The ASTM sub-committee on peat and organic soils have concluded that an organic soil should not be called a peat unless its organic content is 75% or more. Any organic content should be recorded during sample description, since even a small proportion can lead to large increases in compressibility and decreases in the strength of an otherwise good material.

Topsoil is the thin layer of aerated organic matter, close to ground surface, which supports living vegetation.

Hobbs (1986) has suggested that a full description of a peat should include details of colour, degree of decomposition, wetness, main constituents (e.g. fibre type), mineral soil content, smell, chemistry, tensile strength, and plasticity. Such complex description is clearly outside the scope of description that can be justified for normal ground investigation, and indeed will not normally yield useful

correlations with geotechnical behaviour. The proposals of Burwash and Weisner (1984, 1987) would seem to be adequate for most purposes:

1. determine organic content, to confirm that the material is a peat (>75% organic matter);
2. describe degree of humification in accordance with the von Post method (Table 2.5); and
3. where possible, give basic fibre details.

Examples of description would be:

- black amorphous PEAT (H_6);
- brown non-woody fine fibrous PEAT (H_2).

It is extremely important to attempt, during the description of near-surface soils, to identify made ground. Such material may be:

- compressible;
- highly variable;
- chemically contaminated.

Made ground is ground filled by man's activities, rather than as a result of geomorphological processes. Made ground may comprise (for example) compacted granular fill, in which case it may be extremely difficult to detect from the description of an isolated sample. At the other extreme, it may result from the tipping of household waste. In either case, sample description can only hope to identify made ground by searching for man-made artefacts, such as fragments of brick, clinker, tile, glass, etc., and at the other end of the scale, concrete, cars and parts of machinery, paper and plastics.

Table 2.5 Assessment of degree of humification (after von Post (1922))

Degree of humification	Decomposition	Plant structure	Content of amorphous material	Material extruded on squeezing (passing between fingers)	Nature of residue
H1	None	Easily identified	None	Clear, colourless water	
H2	Insignificant	Easily identified	None	Yellowish water	
H3	Very slight	Still identifiable	Slight	Brown, muddy water; no peat	Not pasty
H4	Slight	Not easily identified	Some	Dark brown, muddy water; no peat	Somewhat pasty
H5	Moderate	Recognizable, but vague	Considerable	Muddy water and some peat	Strongly pasty
H6	Moderately strong	Indistinct (more distinct after squeezing)	Considerable	About one-third of peat squeezed out; water dark brown	
H7	Strong	Faintly recognizable	High	About one-half of peat squeezed out; any water very dark brown	Fibres and roots more resistant to decomposition
H8	Very strong	Very indistinct	High	About two-thirds of peat squeezed out; also some pasty water	
H9	Nearly complete	Almost not recognizable		Nearly all the peat squeezed out as a fairly uniform paste	
H10	Complete	Not discernible		All the peat passes between the fingers; no free water visible	

As with very coarse soils, made ground is best described in an exposure or a trial pit. But in this case there will be much greater safety considerations. Made ground can often produce poisonous gas (as a result of decomposition of organic material), will contain sharp materials, likely to injure (glass, metals, etc.), and may give rise to instability in the sides of trial pits. It is not advisable to allow work to be carried out from within trial pits in made ground unless proper support, breathing apparatus, and full protective clothing are available.

There are no formal systems in use for the description of made ground. Where the made ground resembles natural soil, then normal soil descriptions can be used, but with additional comments added to inform the reader as to how the material has been identified as made ground (for example, 'scattered brick and tile fragments'). In all cases the following should be noted:

- organic matter, and its degree of decomposition;
- smell;
- striking colours;
- signs of heat (combustion);
- presence or absence of large objects (concrete blocks, cars, fridges, etc.)
- voids, hollow objects;
- other compressible materials; and
- anything by which the made ground may be dated (for example, product labels, old newspapers).

Secondary constituents

Where soils are composed of a variety of different constituents, a simple scheme is required to allow them to be identified. Norbury *et al.* (1986) have criticized the complexity of the proposals in BS 5930, and there is certainly considerable evidence that they have not been applied in practice. It is suggested, following Norbury *et al*., that the scheme of Table 2.6 should be adopted.

Table 2.6 Identification of soils

Principal soil type	Approx. % of secondary constituent (by weight)	Terminology used	
		Before principal constituent	After principal constituent
Granular soil types (boulders, cobbles, gravel, sand, silt)	0	—	—
	<5	Slightly (sandy*)	With a little (sand*)
	5—20	— (sandy*)	With some (sand*)
	20—40	Very (sandy*)	With much sand
	about 50	(SILT*) and	(SAND*)
Cohesive soil types (clay)	0	—	—
	<35	Slightly (sandy*)	With a little (sand*)
	35—65	— (sandy*)	With some (sand*)
	>65	Very (sandy*)	With much (sand*)

* Use appropriate secondary soil type (e.g. gravel, sand, silt or clay). The description silty CLAY' is not used, and where the sample contains silt or clay the above terms are used to indicate an estimated degree of cohesiveness.

In practice it is very difficult to estimate the secondary constituents of soils by eye and by feel, and particularly so in cohesive soils. Therefore it is important to check descriptions to ensure that they reflect the estimated properties of the different materials, and that the implied total percentage does not exceed 100%. It should be borne in mind that the description of the presence of even small quantities of fines is helpful in granular soil (because the permeability of granular soil is dominated by its fine

fraction), but is not particularly helpful in cohesive soil, where the description of fabric (i.e. the spatial distribution of different particle sizes) is more critical, since this has a large effect on mass strength and permeability. Not more than two secondary constituents should be used, of which only one should appear before the principal soil type. It should be remembered that the method of sampling can change the proportion of different soil types.

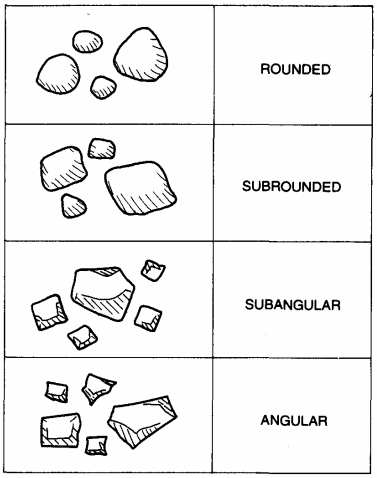
Examples are:

- Slightly silty SAND;
- Silty SAND and GRAVEL with a little clay;
- Very sandy CLAY.

Grading and particle shape

Coarse granular materials, such as sands and gravels have particles of sufficient size to allow a visual assessment of their shape, angularity and grading. Only extremes of shape, such as flat or equidimensional particles should be noted. Angularity is defined in BS 812 and ASTM D2488, and should be given only for boulders, cobbles, gravels and coarse sands. Since roundness depends largely on the method and distance of transportation of the material from its original bedrock, angularity can be useful not only in assessing sands and gravels for use as aggregate, but also in determining the origins of a material and the weathering processes which have brought it to its present state. Figure 2.1 illustrates angularity.

Fig. 2.1 Angularity of coarse soil particles.



Because of their limited accuracy, visual estimates of grading should be restricted to extremes of grading in the coarser soil fractions. The terms ‘uniform’ (i.e. containing a restricted range of particle size) or ‘well-graded’ (i.e. containing a wide range of particle sizes) can be used immediately before the main constituent. For the purposes of classification of soil for earthworks in the UK, the boundary between ‘well-graded’ and ‘uniform’ may be taken at a value of the coefficient of uniformity (D60/D10) equal to 10.

Density and strength

The relative density of granular soils is assessed on the basis of field testing. If field tests are not carried out, then the density description should not be used.

Simple field tests, suitable for application in trial pits, are given in BS 5930: 1981:

Loose	Can be excavated with a spade: 50mm wooden peg can easily be driven
Dense	Requires pick for excavation: 50mm wooden peg hard to drive
Slightly	Visual examination: pick removes soil in lumps which can be cemented abraded.

In boreholes, SPT results are routinely used to provide an estimate of density. Traditionally in the UK Terzaghi and Peck's (1948) classification for sand has been used, regardless of the granular soil type as shown in Table 2.7.

Table 2.7 Classification for sand

SPT N(blows/300 mm)	Relative density
0—4	Very loose
4—10	Loose
10—30	Medium dense
30—50	Dense
>50	Very dense

However, different components have used different systems. In some the density descriptor (for example, medium dense) used on a borehole log was obtained simply by averaging the SPT in a given stratum, and looking up the appropriate term in Table 2.7. In others, following Gibbs and Holtz (1957) work on the effect of overburden presence on the SPT 'N' value, 'N' values were corrected to a common overburden pressure before the descriptor was chosen. These two procedures will produce quite different results either at very shallow depth or at very great depths.

Following Skempton (1986) and Clayton (1992) the procedure proposed is as follows.

1. 'N' values should be corrected for effective overburden pressure and energy (see Chapter 10), to give $(N_1)_{60}$
2. In coarse granular soils, it should be noted that the N value will be high for the *in situ* relative density. There is now increasing evidence that coarse (i.e. gravel) particles significantly increase penetration resistance, and the classification given below, which was derived for sands, will overestimate the density of gravels.
3. The density descriptor should be selected from Table 2.8.

Table 2.8 Selection of density descriptor

$(N_1)_{60}$ (blows/300 mm)	Density descriptor
0—3	Very loose
3—8	Loose
8—25	Medium dense
25—42	Dense
42—58	Very dense

During sample description, simple hand tests are used to describe the consistency of cohesive soils. Consistency is the estimated undrained shear strength of the intact blocks of a soil. Large diameter triaxial tests carried out on fissured clays will normally give much lower values of undrained shear strength because of the weakening effect of the fissures. Three simple tests are commonly used to determine consistency. These use the hand and fingers, a pocket penetrometer or a hand vane tester (Fig. 2.2). The vane test will normally give undrained shear strengths in engineering units, but most pocket penetrometers are calibrated in terms of unconfined compressive strength; this should be halved to find the undrained shear strength. All tests used to find consistency are carried out on small

samples at a rate much faster than is used in laboratory strength testing. For these reasons their results should not normally be used in engineering calculations. Some penetrometers have in-built (empirical) corrections, which are intended to correct for size and rate effects. These corrections must be removed by calculation before the strength descriptor is selected from the list below.

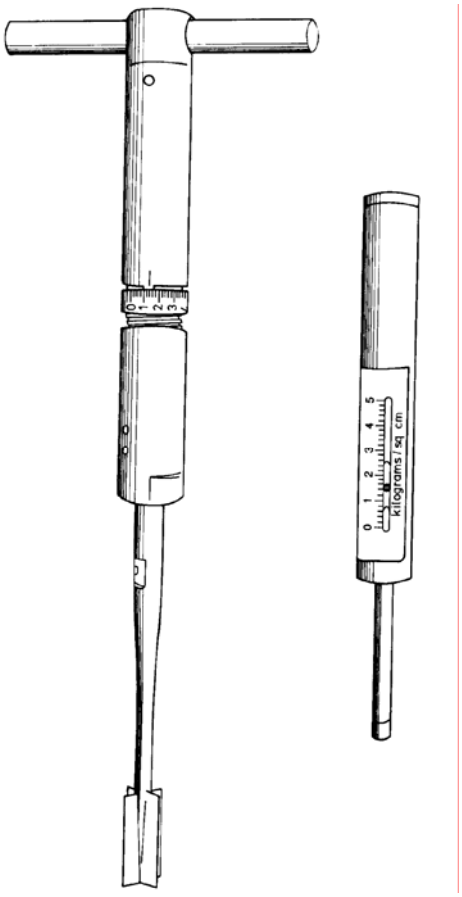


Fig. 2.2 Hand vane and pocket penetrometer.

Where strength measurements cannot be made, the consistency of a soil can be determined with reasonable accuracy using hand and fingers, as follows:

1. Very soft Exudes between fingers when squeezed in hand
2. Soft Moulded by light finger pressure
3. Firm Can be moulded by strong finger pressure
4. Stiff Cannot be moulded by fingers Can be indented by thumb
5. Very stiff Can be indented by thumb nail
6. Hard Cannot be indented by thumb nail (not included in BS 5930:1981).

Where a sample straddles a boundary, this can be indicated as, for example, 'firm to stiff'. On the engineer's borehole record, where a gradual or sharp change in consistency occurs within a clay formation, it is common to use a description such as 'firm, becoming stiff from 12 m, etc.'. Alternatively, if the formation has been fully described before the depth has been reached at which a change in consistency occurs, one can put '...becoming stiff at 12 m' opposite the relevant depth. Consistency is related directly to strength as shown in Table 2.9.

The blows necessary to drive open-drive sampler tubes such as the SPT split spoon or U100 into the ground are sometimes used in estimating consistency. Opinions on the use of the SPT N value to determine undrained shear strength vary. De Mello (1969) shows that correlations between strength and N value are generally poor, but Stroud (1974) has found good correlations for heavily overconsolidated soils, mainly in the UK (Fig. 9.3). There seems little point in attempting to correlate

the blows required to drive a U100 sampler with soil strength, because the weight of the hammer and ancillary equipment can vary considerably. Some drilling companies use a jarring link and sinker bar at the base of the hole, while others rod up to the top of the hole and drive the tube with an SPT trip hammer.

Table 2.9 Relationship between consistency and undrained shear strength of intact soil

Consistency	Undrained shear strength			
	p.s.i.	p.s.f.	t.s.f.	kN/m^2
Very soft	<2.5	<375	<0.20	<20
Soft	2.5—5.0	375—750	0.20—0.40	20—40
Firm	5—10	750—1500	0.40—0.75	40—75
Stiff	10—20	1500—3000	0.75—1.50	75—150
Very stiff	20—40	3000—6000	1.50—3.00	150—300
Hard	>40	>6000	>3.00	<300

Fabric and structure

Fabric can be defined as the arrangement of different particle size groups within a soil, whereas structure relates to the arrangement of individual particles within a particular size group. Fabric is of considerable importance in determining the mass behaviour of soil, as Rowe (1968a,b, 1971, 1972) has shown, and every effort should be made to assess its form during engineering sample description. Structure is normally difficult to detect either with the naked eye or a hand lens.

Fabric may exist in the form of silt filled fissures, laminations or varves consisting of very thin successive layers or gradations of clays, silts and sands, pockets and lenses of granular material which have a limited lateral extent within a mass of clay, or layers or seams which have a reasonable thickness and considerable lateral extent. The only satisfactory way to look for fabric is to cut halfway into an undisturbed sample along its length, and then pull it into two equal semi-cylindrical parts (e.g. Fig. 7.4). In this way the uncut part of the flat face will not be smeared, and coarse fabric will be immediately obvious. The presence of fine silt layers or dustings in clays can be detected by leaving one-half of the sample exposed. The silt, which dries much faster than the clay, will show up as lighter. Fissures should be examined to see whether they contain silt or sand sized material, and will therefore speed consolidation. The spacing of these features is of obvious relevance to the choice of sample and test specimen size. BS 5930:1981 defines descriptive terms for the spacing of bedding and other discontinuities.

Colour

Colour changes may indicate the extent of weathering, or changes of strata. The absolute colour of a soil is rarely of enormous importance, which is perhaps fortunate as many people are colour-blind and also because colour is highly subjective. The colour seen by one person will depend on the type of light source, the background, the size of the object and the colours that have been seen immediately before. Therefore, if colour is to be recorded objectively certain precautions should be taken. When describing soils in a laboratory, daylight fluorescent tubes should be used in order to try to obtain the same colours as will have been seen in the field. Colours should be compared with a standard chart specifically designed for this purpose. One such, based on the Munsell colour classification system is the Geological Society of America's 'Rock-colour Chart'.

SOIL CLASSIFICATION

Soil classification systems are set up to allow the expected properties of the soil in a given situation to

be conveyed in a shorthand form. Geotechnical soil classification systems are often termed 'textural classification', probably because of their agricultural origin ('textural' referred to the appearance of the ground several weeks after ploughing (Smart 1986) — grouping of soils into 'light', 'medium' and 'heavy' corresponds to the need for one, one-and-a-half and two horses respectively to plough one furrow). In current usage, soil classification systems are primarily aimed at road and airfield applications.

The British Soil Classification System appeared first in BS 5930:1981. A corrected version was subsequently published by Dumbleton (1981), see Table 2.10. The US Soil Classification is standardized in ASTM D2487—69. In both systems the soil is classified on the basis of particle size distribution and plasticity (Atterberg limit) tests (see Chapter 8 for further information), and described in terms of Group Symbols (such as 0, for gravel). Where required, these symbols can be applied to engineering borehole records.

Soil classification works well for granular soils, but is less satisfactory for cohesive soils. In fact, the soils are divided into 'coarse' and 'fine' based solely on the percentage by weight passing a given sieve size (35% finer than 0.06mm in the BSCS). The same soil 'mix' will behave differently, if clay is present, depending upon the type of clay mineral (e.g. kaolin, illite, montmarillonite). Also, most soil classification systems do not take into account the differences that occur when the same clay fraction is mixed with granular soils of different gradings (see, for example, Smart, 1986). When soils are classified as 'fine soils' then the division between silt and clay is made by reference to Casagrande's A-line (Fig. 8.4). 'Clays' are said to plot above the A-line and silts below. Yet crushed upper chalk (demonstrably a silt-sized material, and 98.5% calcium carbonate) plots above the A-line, while pure kaolin (and kaolin—silt mixtures) plots below the A-line. Smart (1986) has rightly concluded that 'samples plotting below the A-line should not automatically be called silt, nor should samples above it be called clay'. But whatever their faults, soil classification systems must be applied to the letter when they are used.

Note I The name of the soil group should always be given when describing soils, supplemented, if required, by the group symbol, although for some additional applications (e.g. longitudinal sections) it may be convenient to use the group symbol alone.
Note 2. The group symbol or sub-group symbol should be placed in brackets if laboratory methods have not been used for identification, e.g. (GC).
Note 3. The designation FINE SOIL or FINES, F, may be used in place of SILT, M, or CLAY, C, when it is not possible or not required to distinguish between them.
Note 4. GRAVELLY if more than 50% of coarse material is of gravel size. SANDY if more than 50% of coarse material is of sand size.
Note 5. SILT (M-SOIL), M, is material plotting below the A-line, and has a restricted plastic range in relation to its liquid limit, and relatively low cohesion. Fine soils of this type include clean silt-sized materials and rock flour, micaceous and diatomaceous soils, pumice, and volcanic soils, and soils containing halloysite. The alternative term 'M-soil' avoids confusion with materials of predominantly silt size, which form only a part of the group. Organic soils also usually plot below the A-line on the plasticity chart, when they are designated ORGANIC SILT, MO.
Note 6. CLAY. C. is material plotting above the A-line, and is fully plastic in relation to its liquid limit.

Table 2.10 The British Soil Classification System (Dumbleton 1981)

Soil groups (see note 1)			Subgroups and laboratory identification				
GRAVEL and SAND may be qualified Sandy GRAVEL and Gravelly SAND, etc. where appropriate		Group symbol (see notes 2 & 3)	Subgroup symbol (see note 2)	Fines (% less than 0.06 mm)	Liquid limit %	Name
COARSE SOILS less than 35% of the material is finer than 0.06 mm	GRAVELS More than 50% of coarse material is of gravel size (coarser than 2mm)					
	Slightly silty or clayey GRAVEL	G	GW	0 to 5		Well graded GRAVEL
			GP			Poorly graded/Uniform/Gap graded GRAVEL
	Silty GRAVEL	G-F	G-M	GWM GPM	5 to 15		Well graded/Poorly graded silty GRAVEL
	Clayey GRAVEL		G-C	GWC GPC			Well graded/Poorly graded clayey GRAVEL
	Very silty GRAVEL	GF	GM	GML, etc	15 to 35		Very silty GRAVEL; subdivide as for GC
	Very clayey GRAVEL		GC	GCL		<35	Very clayey GRAVEL (clay of low,
				GCI		35 to 50	intermediate,
				GCH		50 to 70	high,
				GCV		70 to 90	very high,
				GCE		>90	extremely high plasticity)
	SANDS More than 50% of coarse material is of sand size (finer than 2mm)						
	Slightly silty or clayey SAND	S	SW	SW	0 to 5		Well graded SAND
			SP	SPu SPg			Poorly graded/Uniform/Gap graded SAND
	Silty SAND	S-F	S-M	SWM SPM	5 to 15		Well graded/Poorly graded silty SAND
	Clayey SAND		S-C	SWC SPC			Well graded/Poorly graded clayey SAND
	Very silty SAND	SF	SM	SML, etc	15 to 35		Very silty SAND, subdivided as for SC
	Very clayey SAND		SC	SCL		<35	Very clayey SAND (clay of low,
				SCI		35 to 50	intermediate,
				SCH		50 to 70	high,
				SCV		70 to 90	very high
				SCE		>90	extremely high plasticity)

Table 2.10 *continue* (Dumbleton 1981)

Soil groups (see note 1)			Subgroups and laboratory identification				
GRAVEL and SAND may be qualified Sandy GRAVEL and Gravelly SAND, etc. where appropriate			Group symbol (see notes 2 & 3)	Subgroup symbol (see note 2)	Fines (% less than 0.06 mm)	Liquid limit %	Name
FINE SOILS more than 35% of the material is finer than 0.06 mm	Gravelly or sandy SILTS & CLAYS 35% to 65% fines	Gravelly SILT	FG	MG	MLG, etc			Gravelly SILT: subdivide as for CG
		Gravelly CLAY (see note 4)		CG	CLG	35 to 65	<35	Gravelly CLAY of low plasticity
					CIG		35 to 50	of intermediate plasticity
					CHG		50 to 70	of high plasticity
					CVG		70 to 90	of very high plasticity
					CEG		>90	of extremely high plasticity
		Sandy SILT (see note 4)	FS	MS	MLS, etc	35 to 65		Sandy SILT; subdivide as for CG
		Sandy CLAY		CD	CLS, etc			Sandy CLAY; subdivide as for CG
	SILTS & CLAYS 65% to 100% fines	SILT (M-SOIL)	F	M	ML, etc	65 to 100		SILT: subdivide as for CG
		CLAY (see note 5 & 6)		C	CL		<35	CLAY of low plasticity
					CI		35 to 50	of intermediate plasticity
					CH		50 to 70	of high plasticity
					CV		70 to 90	of very high plasticity
					CE		>90	of extremely high plasticity
ORGANIC SOILS		Descriptive letter 'O' suffixed to any group or sub-group symbol.					Organic matter suspected to be a significant constituent. Example MHO. Organic SILT of high plasticity.	
PEAT		Pt	Peat soils consist predominantly of plant remains which may be fibrous or amorphous.					

ROCK DESCRIPTION

An engineering description of rock should, in theory, embody those features which are significant in influencing its engineering performance. Most rocks are cut by discontinuities which characteristically have little or no tensile strength. The engineering performance (strength, compressibility, permeability and durability) of any mass of rock containing such fractures will be significantly influenced by their presence. The description of these fractures is clearly an important aspect of rock description in general. A complete rock description is commonly divided into three parts:

1. a description of the rock material (or intact rock):
   the term rock material here refers to rock that has no through-going fractures significantly reducing its tensile strength;
2. a description of the discontinuities; and
3. a description of the rock mass:
   the term rock mass here refers to the rock material and the discontinuities. information from (1) and (2) are therefore combined to provided an overall description of the rock mass.

Ideally, the best means of obtaining a comprehensive description of the rock mass is by careful examination of large exposures. In many site investigations on rock, however, it is not always possible to gain access to such large exposures and hence the rock mass description has to be derived mainly from borehole information. Boreholes provide a reasonable means for examining the rock material but do not permit a comprehensive description of the discontinuities.

In soil description, the soil name is derived from the particle size or organic constituent which is present in sufficient proportions to have the most significant influence on the engineering behaviour. Hence the soil name conveys valuable information on engineering performance. The systematic engineering description of soils has evolved with the relatively young science of soil mechanics. In contrast to this, a basis for naming rocks was already well established within the science of geology when the need to develop a method for the systematic engineering description of rock arose. Rock names currently used in such descriptions are therefore, by tradition, based on geological names. This can be a source of confusion among engineers with minimal geological training. The names reflect the genesis of the rock and are often based on such factors as grain size, texture, and mineral assemblage. The resultant names seldom bear any relation to engineering performance. For the igneous rocks which comprise only 25% of the Earth's crust there are over 2000 names, reflecting subtle mineralogical differences, usually with little engineering significance. In contrast, sedimentary rocks have fewer names and yet they are more abundant and can exhibit a much broader range of engineering properties.

Geological classification of rock

Rocks are divided into three groups based on mode of formation. These groups include the following.

1. *Igneous rocks*. These are formed from the solidification of molten material.
2. *Sedimentary rocks*. These are formed from the accumulation of fragmental rock material and organic material or by chemical precipitation.
3. *Metamorphic rocks*. These are formed by alteration of existing rocks through the action of heat and pressure.

A series of handbooks produced by the Geological Society of London on the field description of igneous rocks (Thorpe and Brown 1985), sedimentary rocks (Tucker 1982) and metamorphic rocks (Fry 1984), provide some useful advice but are aimed primarily at field geologists. Within each genetic class, different classification systems are employed. In many cases a full petrographic analysis is required to classify a rock specimen in geological terms. Such classification systems are too

elaborate for engineering application, and usually provide little or no information of engineering significance. For engineering use the classification systems have been simplified and the number of rock names kept to a minimum. In a full rock description a similar approach to describing soil samples is employed, making use of prefixes and suffixes to describe selected features of the hand specimen and to include estimates of certain engineering characteristics. Before discussing the scheme by which rocks are described, it is necessary to outline how rock types are identified.

The principal criteria used in classifying all types of rock material include:

1. mineral assemblage;
2. texture and fabric,
   texture refers to the mutual relationship between the constitutive crystals/grains; and
3. grain size.

Igneous rocks

Rocks in this broad family are characterized by a crystalline or more rarely, glassy texture with low porosity (usually <2%), unless the rock has been weathered. Generally the strongest rocks are found among this group.

Igneous rocks are formed from solidification of molten material (magma) which may originate in or below the Earth's crust. Magma may solidify within the crust (at depth or near the surface) giving rise to intrusive igneous rocks, or it may pour out on to the Earth's surface before solidifying completely, giving rise to extrusive igneous rocks. The initial chemical composition of the magma together with the rate at which the magma cools determine the mineral assemblage, texture and grain size of the resulting igneous rocks.

The grain size of igneous rock can range from very coarse (equivalent to gravel and cobbles in soil) to fine (equivalent to silt and clay in soil) and is related to the rate of cooling of the parent magma. Coarse-grained rocks are associated with slow cooling rates, and fine-grained rocks with more rapid rates of cooling. If the magma cools very rapidly then there is no time for crystals to develop and amorphous glassy rock is produced. Generally intrusive igneous rocks are crystalline with grain sizes ranging from very coarse- to medium-grained (cobbles to fine sand in soil terms). Extrusive rocks are usually fine-grained (silt to clay in soil terms), crystalline, glassy (or opalescent) or porous.

The principal criteria used in the petrological classification of igneous rocks are as follows.

- Mineral assemblage.
  The main rock forming minerals which occur in igneous rocks include quartz, feldspar, muscovite, biotite and mafic minerals. Classification is based on the relative proportions of quartz, feldspar and mafic minerals. Quartz and feldspar are generally light in colour, whereas the mafic minerals are generally dark. A simple classification of igneous rocks may be based on colour index (i.e. whether the rock is dark or light in colour).
- Grain size.

A simplified classification suitable for engineering use is shown in Table 2.11. In general, most igneous rocks will be placed in the acid or basic category. It is often difficult to identify intermediate igneous rocks in hand specimen or indeed in the mass. It should be pointed out that experienced igneous petrologists will often resort to a simple classification such as that shown in Table 2.11 when making a preliminary description of the rock as seen in the field. A more obscure geological name would normally only be applied after thin sections have been studied in the laboratory.

Table 2.11 Classification of igneous rocks

Field relations	texture	Grain size	Acid	Intermediate	Basic	Ultrabasic
			Light coloured rocks	Light/dark coloured rocks	Dark coloured rocks	Dark coloured rocks
Intrusive	Crystalline	Very coarse grained 60 mm	Rock consists of very large and often well developed crystals of quartz, feldspar, mica and frequently rare minerals. PEGMATITE			
		Coarse grained 2 mm	At least 50% of the rock is coarse grained enough to allow individual minerals to be identified.			Rock is coarse grained and dark in colour (dull green to black) with a granular texture. It contains olivine and augite in abundance but no feldspars.
			Rock is light coloured with an equigranular texture (majority of grains approximately the same size) and contains >20% quartz with feldspar in abundance. GRANITE	Rock may be medium to dark in colour with a more or less equigranular texture and contains <20% quartz with feldspar and hornblende in abundance. DIORITE	Rock is dark coloured and often greenish with abundant plagioclase (about 60%) and augite together with some olivine. The rock usually feels dense. GABBRO	PERIDOTITE
		Medium grained 0.06 mm	At least 50% of the rock is medium grained. Crystal outlines are generally visible with the aid of a hand lens but individual minerals may be difficult to identify.			Rock is greyish green to black with a splintery fracture when broken and generally feels soapy or waxy to the touch. It is often criss-crossed by veins of fibrous minerals and/or banded.
			Rock is similar in appearance to granite but the crystals are generally much smaller. MICRO-GRANITE	Rock is similar in appearance to dionte but the crystals are generally much smaller. MICRO-DIORITE	Rock is dark coloured and often greenish with a granular texture. Individual minerals may be difficult to identify. The rock usually feels dense. DOLERITE	SERPENTINITE
Extrusive	Crystalline/glassy	Fine grained	At least 50% of the rock is fine grained. Outlines of crystals are not usually visible even with the aid of a hand lens. All rocks in this category may be vesicular.			
			Rock is light coloured (often pale reddish brown or pinkish grey) and may be banded. RHYOLITE Rock is light coloured with a very low specific gravity and highly vesicular. PUMICE	Rock is medium to dark in colour (shades of grey, purple, brown, or green) and frequently porphyritic ANDESITE	Rock is black when fresh and becomes red or green when weathered. The rock is often vesicular and/or amygdaloidal. BASALT	
	Glassy	Glassy	Rock is glassy and contains few or no phenocrysts. It is often black in colour and has a characteristic vitreous lustre and conchoidal fracture. OBSIDIAN Rock is glassy and contains few or no phenocrysts. It may be black, brown or grey in colour with a characteristic dull or waxy lustre. PITCHSTONE			

Basalt is probably the most common extrusive igneous rock. It is generally characterized by structures such as flow banding, often with porous zones (pumice) and columnar jointing. Basalts extruded under water often exhibit pillow shaped structures (pillow lava). Of the intrusive rocks, granite and dolerite are perhaps the most ubiquitous.

Some igneous rocks exhibit large crystals embedded in a finer-grained matrix. Such rocks are termed porphyritic and the large crystals are termed phenocrysts. Extrusive igneous rocks often have numerous spherical or ellipsoidal voids (vesicles) scattered throughout or concentrated in layers. These are produced by the inclusion of gas bubbles within the magma as it cools. In some cases, these voids may be filled with minerals. Such mineral filled inclusions are termed amygdales.

An elementary discussion of igneous rocks is given by Blyth and deFreitas (1984), McLean and Gribble (1985) and Goodman (1993), and a more detailed discussion dealing with genesis and petrography is given by Hatch *et al.* (1972).

Sedimentary rocks

Most sedimentary rocks are cemented aggregates of transported fragments derived from pre-existing rocks. Typically these rocks will comprise rock fragments, grains of minerals resistant to weathering (mainly quartz) and minerals derived from the chemical decomposition of pre-existing rocks (clay minerals) bound together with chemical precipitates such as iron oxide and calcium carbonate. Other forms of sedimentary rock include accumulations of organic debris (typically shell fragments or plant remains), fragmental material derived from volcanic eruptions, and minerals that have been chemically precipitated (for example, rock salt, gypsum and some limestones).

The polygenic nature of sedimentary rocks has resulted in the development of a number of classification schemes. However two broad groups can be identified.

1. *Detrital (fragmental or clastic) sediments*:
   - clastic deposits: accumulations of rock or mineral fragments;
   - bioclastic deposits: accumulations of faunal debris (for example, shell, coral or bones); and
   - pyroclastic deposits: accumulations of fragmental material produced by volcanic eruption.
2. *Organic and chemical sediments*:
   - organic deposits: accumulations of dead plants and other biodegradable material; and
   - chemical deposits: accumulations of minerals chemically precipitated from surface water or groundwater.

The fragmental nature of the rocks in group (1) permits them to be classified primarily on the basis of grain size, predominant mineral composition (i.e. greater than 50%) and texture and fabric. Rocks belonging to group (2) are classified on the basis of composition alone. Table 2.12 shows how sedimentary rocks are classified for engineering purposes.

The detrital sediments when deposited may be regarded as soils. With time they are gradually transformed into rock through the action of consolidation, creep and cementation. This process termed diagenesis results in a progressive increase in strength and decrease in compressibility and permeability. The degree of diagenesis is highly variable and results in rocks with porosities ranging from less than 1% to greater than 50%. Sandstones may be weakly cemented such that individual grains may be easily removed by light abrasion or so well cemented that they are much stronger than concrete. Some of the younger mudrocks in the UK are only weakly cemented and are thus considered as engineering soils (for example, London Clay, Gault Clay, Weald Clay). The older and more strongly cemented mudrocks are generally stronger and are less likely to soften and slake when exposed to the elements. However many of these have been subjected to such high overburden stresses that the platey clay mineral particles have become aligned and impart a fissility to the rock resulting in a high degree of anisotropy. Such rocks are referred to as shale.

Table 2.12 Classification of sedimentary rocks

Group Usual structure	Grain size	Composition and texture	Detrital sediments Bedded	
			Quartz, rock fragments, feldspar, and other minerals.	At least 50% of the rock is composed of carbonate minerals (rocks usually react with dilute HCl).
Granular	Coarse-grained 2 mm	Rudaceous	Rock is composed of more or less rounded grains in a finer grained matrix: CONGLOMERATE Rock is composed of angular or sub-angular grains in a finer grained matrix: BRECCIA	CALCI-RUDITE
Granular	Medium-grained 0.06 mm	Arenaceous	Rock is composed of: (i) mainly mineral and rock fragments. SANDSTONE (ii) 95% quartz. The voids between the grains may be empty or filled with chemical cement. QUARTZ SANDSTONE (iii) 75% quartz and rock fragments and up to 25% feldspar (grains commonly angular). The voids may be empty or filled with chemical cement. ARKOSE (iv) 75% quartz and rock fragments together with 15% + fine detrital material. ARGILLACEOUS SANDSTONE	CALC-ARENITE
Smooth	Fine- grained 0.002 mm	Argillaceous	Rock is composed of at least 50% fine-grained particles and feels slightly rough to the touch. SILTSTONE Rock is homogeneous and fine-grained. Feels slightly rough to smooth to the touch. MUDSTONE Rock has same appearance and feels as mudstone but reacts with dilute HCl. CALCAREOUS MUDSTONE	CALCI-SILTITE CHALK (Bioclastic)
Smooth	Very fine-grained	Argillaceous	Rock is composed of at least 50% very fine-grained particles and feels smooth to the touch. CLAYSTONE Rock is finely laminated and or fissile. It may be fine or very fine grained SHALE	CALCI-LUTITE

Some sandstones (particularly quartzite) may appear to be crystalline as a result of being relatively fine grained and well cemented or as a result of recrystallization. In the absence of any evidence of bedding or other sedimentary features it would be easy to mistaken them for igneous rocks. In terms of the engineering behaviour of the rock material such a mistake is not serious. However the relationship between an intrusive igneous rock and the host rock is quite different from that of a sandstone unit within a sequence of other sedimentary rocks and hence such an error may have serious consequences.

Rocks containing at least 50% clay minerals are termed claystone if homogeneous or shale if laminated and fissile. These fine-grained rocks will generally have a smooth surface texture.

Detrital sediments containing mainly carbonate minerals may be subdivided on the basis of grain size. They may have granular or smooth textures depending on the grain size. Those with granular textures are termed calci-rudite, calc-arenite and calci-siltite depending on grain size. Very fine-grained varieties include chalk and calci-lutite.

Table 2.12 *continue*

Group Usual structure			Pyroclastic sediments Bedded	Chemical and organic sediments	
Grain size		Composition and texture		Bedded	Massive/Bedded
			At least 50% of the grains are of fine-grained volcanic material. Rocks often composed of angular mineral or igneous rock fragments in a fine-grained matrix.	Crystalline carbonate rocks depositional texture not recognizable. Fabric is non-elastic.	Depositional textures often not recognizable.
Granular	Coarse-grained 2 mm	Rudaceous	Rock is composed of: (i) Rounded grains in a fine-grained matrix: AGGLOMERATE (ii) Angular grains in a fine-grained matrix: VOLCANIC BRECCIA	Rock is crystalline and composed of carbonate (>90%) reacts violently with HCl. LIMESTONE Rock is crystalline and may show a yellowish colouration and/or the presence of voids. Reacts mildly with cold dilute HCl. Reaction increases by heating the HCl. DOLOMITE LIMESTONE Rock is crystalline and composed of magnesium carbonate (>90%). When small chip of rock is immersed in dilute HCl, these is no immediate reaction, but a slow formation of CO_2 beads on surface of chip; reaction slowly accelerates. Rate of reaction in increased by heating HCl. DOLOMITE	Rock is crystalline, salty to taste and may be scratched with the finger nail. HALITE (rock salt) Rock is crystalline and may be scratched with the finger nail. Grains turn into a chalky white substance when burnt for a few minutes. GYPSUM Rock is crystalline: colourless to white, frequently with a bluish tinge. It is harder than gypsum and has three orthogonal cleavages. ANHYDRITE Rock is black or brownish black and has a low specific gravity (1.8—1.9). It may have a vitreous lustre and conchoidal fracture and/or breaks into pieces that are roughly cuboidal. COAL
	Medium-grained 0.06 mm	Arenaceous	Rock is composed of mainly sand sized angular mineral and rock fragments in a fine-grained matrix. TUFF		
Smooth	Fine- grained 0.002 mm	Argillaceous	Rock is composed of silt sized fragments in a fine- to very fine-grained matrix. Matrix and fragments may not always be distinguished in the hand specimen.		Rock is black or various shades of grey and breaks with a characteristic conchoidal fracture affording sharp cutting edges. The rock cannot be scratched with a penknife. FLINT Rock has similar appearance and hardness as flint but breaks with a more or less flat fracture. CHERT
	Very fine-grained		FINE-GRAINED TUFF VERY FINE-GRAINED TUFF		

Sedimentary rocks containing at least 50% volcanic material although detrital in nature are generally treated separately. These are referred to as pyroclastic sediments and are subdivided on the basis of grain size. It is sometimes difficult to recognize pyroclastic rocks without the aid of a thin section. Medium-grained pyroclastic rocks (tuffs) are generally characterized by angular grains (medium to coarse sand size) which resemble crystals in a fine-grained matrix.

The chemical and organic sediments are subdivided on the basis of mineral or organic content and texture. Rocks formed by chemical precipitation generally have a crystalline texture. Most chemically precipitated rocks are water soluble and weaker than igneous or metamorphic rocks.

Carbonate rocks (greater than 50% carbonate content) with a crystalline texture are termed limestone or dolomite according to their magnesium content. Both the crystalline and fragmental varieties of carbonate rocks can be identified by their reaction with hydrochloric acid (HCl).

An elementary discussion of sedimentary rocks is given by Blyth and deFreitas (1984), McLean and Gribble (1985) and Goodman (1993). For a more detailed discussion, see Greensmith (1978) and Tucker (1982).

Metamorphic rocks

Metamorphic rocks are derived from pre-existing rocks of all types in response to marked changes in temperature or stress or both. An increase in temperature or pressure can cause the formation of new minerals and the partial or complete recrystallization of the parent rock with the development of new textures. Three broad types of metamorphism can be distinguished.

1. *Dynamic metamorphism*. This type of metamorphism generates intense stresses locally, which tend to deform, fracture and pulverize the rock.
2. *Regional metamorphism*. This type of metamorphism affects an extensive area through an increase in pressure and temperature.
3. *Contact metamorphism*. This type of metamorphism results from the heating of the host rock in the vicinity of a body of intruded igneous magma.

In all of these groups it is possible to distinguish various intensities of metamorphism (termed metamorphic grade) based on mineral assemblages. The metamorphic minerals produced, however, depend to a large extent on the chemical composition of the original rock and are often difficult to identify in the hand specimen. Thus mineral composition is not an essential factor in a simple classification of metamorphic rocks. In many cases metamorphic rocks are deformed during recrystallization resulting in the development of characteristic and often conplex fabrics and textures. The most conspicuous fabric exhibited in the hand spec1en or field exposure is a layering (fabric) and preferred orientation of mineral gneiss within each layer (texture). This type of fabric with its characteristic texture is termed foliation or schistosity, and is most common in regional metamorphic rocks.

The complexity of metamorphic rocks is such that there no generally agreed descriptive classification, nor are there agreed definitions of such common metamorphic rock types as schist, gneiss and amphibolite. This is confusing for the geologist and even more so for the engineer.

Many metamorphic rocks retain sufficient of their primary sedimentary or igneous features to be given sedimentary or igneous names. If it is necessary to emphasize that a particular rock has undergone metamorphism, this may be done by adding the prefix '*meta-*' before the appropriate igneous or sedimentary rock name, for example, metabasalt, metaquartzite.

In many areas, however, primary igneous and sedimentary features have been completely destroyed by metamorphism. In others it is not certain whether the boundaries between different compositional types of metamorphic rock represent sedimentary bedding or not. In such cases a metamorphic rock name must be used. Fabric may be used together with grain size to produce a simple aid to naming such rocks as shown in Table 2.13.

An elementary discussion of metamorphic rocks is given by Blyth and deFreitas (1984), McLean and

Gribble (1985) and Goodman (1993), and a more detailed discussion dealing with genesis and petrography is given by Mason (1978).

Table 2.13 Classification of metamorphic rocks

Fabric	Foliated	Massive
Grain size		
Coarse-grained	Rock appears to be a complex intermix of metamorphic schists and gneisses and granular igneous rock. Foliations tend to be irregular and are best seen in field exposure: MIGMATITE	Rock contains randomly orientated mineral grains (fine- to coarse-grained). Foliation, if present, is poorly developed. This rock type is essentially a product of thermal metamorphism associated with igneous intrusions and is generally stronger than the parent rock: HORNFELS
	Rock contains abundant quartz and/or feldspar. Often the rock consists of alternating layers of light coloured quartz and/or feldspar with layers of dark coloured biotite and hornblende. Foliation is often best seen in field exposures: GNEISS	Rock contains more than 50% calcite (reacts violently with dilute HCl), is generally light in colour with a granular texture: MARBLE
	Rock consists mainly of large platey crystals of mica, showing a distinct subparallel or parallel preferred orientation. Foliation is well developed and often undulose:	If the major constituent is dolomite instead of calcite (dolomite does not react immediately with dilute HCl), then the rock is termed a:
2 mm	SCHIST	DOLOMITIC MARBLE
Medium-grained	Rock consists of medium- to fine grained platey, prismatic or needle-like minerals with a preferred orientation. Foliation often slightly nodulose due to isolated larger crystals which give rise to a spotted appearance:	Rock is medium to coarse-grained with a granular texture and is often banded. This rock type is associated with regional metamorphism:
0.06 mm	PHYLLITE	GRANULITE
Fine-grained	Rock consists of very fine grains (individual grains cannot be recognized in hand specimen) with a preferred orientation such that the rock splits easily into thin plates:	Rock consists mainly of quartz (95%) grains which are generally randomly orientated giving rise to a granular texture:
	SLATE	QUARTZITE (METAQUARTZITE)

DESCRIPTION OF ROCK MATERIAL

In the UK various methods have been proposed for the description of intact rock. Table 2.14 compares the principal methods.

Table 2.14 Comparison of the principal methods of description of intact rock

Description	Anon (1970)	Anon (1972)	BS 5930 (1981)
Colour	•	•	•
Grain size	•	•	•
Texture and structure	•	•	•
Weathered state	•	•	•
Alteration state	•	•	
Cementation state	•		
Minor lithological characteristics	•	•	
Mineral type	•		
ROCK NAME	•	•	•
Estimated strength	•	•	•
Other characteristics and properties		•	•

Over the years the number of terms addressed specifically in the description of rock material has reduced in an effort to simplify the description process and to make it more concise. However many

geologists and engineers still make reference to such factors as cementation state, minor lithological characteristics and mineral type where relevant in their rock descriptions.

The following scheme for systematic rock material description is commonly used in practice:

(a) colour;
(b) grain size;
(c) texture fabric and structure;
(d) weathered state and alteration state where relevant;
(e) minor lithological characteristics, including cementation state where relevant;
(f) ROCK NAME (in capitals);
(g) estimated strength of the rock material; and
(h) other terms indicating special engineering characteristics.

An example of this system in use is shown in Table 2.15.

Table 2.15 Example of systematic rock material description

	(i)	(ii)	(iii)
(a)	Light pinkish grey	Light yellowish brown	Light pinkish white
(b)	Coarse-grained	Fine-grained	Medium-grained
(c)	Porphyritic, massive	Thickly bedded	Foliated
(d)	Slightly weathered Slightly kaolinized	Fresh	Fresh
(e)		Weakly cemented Ferruginous	With bands of dark coloured biotite with preferred orientation
(f)	GRANITE	QUARTZ SANDSTONE	GNEISS
(g)	Very strong	Weak	Very strong
(h)	Impermeable except along joints	Porous	

The word order used in both soil and rock description should be consistent and highlight the engineering behaviour of the material. The schemes were developed at different times and by groups of people with different backgrounds. The scheme for describing soil was developed first alongside the science of soil mechanics. It represents a scheme developed by engineers for engineers. The scheme for describing rock in a systematic manner for engineering purposes, however, was developed much later but did not follow the science of rock mechanics. The resulting scheme is biased somewhat towards geology. in mitigation, this probably stems from the difficulties involved in describing a material that can take on so many different forms within an existing framework of geological terms that need to be retained for ease of communication between engineers and geologists. Apart from the potential communication problems there is a strong argument for not using geological names in rock description and classification. indeed Duncan (1969) proposed a scheme based on texture, structure, composition (calcareous or non-calcareous), colour and grain size.

As a result of the different paths taken in the development of the two schemes they are not consistent in the word order used. Table 2.16 shows a comparison between the two schemes illustrated in this book.

Classical models of soil mechanics involve the concepts of initial porosity and its subsequent modification by stress history. The engineering description of soil embodies these concepts and in addition recognizes the undrained strength of low permeability soils. The first term in the description (*consistency and relative density*) is providing information on strength (for cohesive soils) or initial porosity (for cohesionless soils). The second term (*fabric or fissuring*) provides some information on the likely engineering performance of the soil in the mass. The descriptive terms used here should

highlight inhomogeneity, anisotropy and discontinuities such as fissures. The third term (*colour*) is rarely of great importance other than as a means of correlation.

Table 2.16 Comparison between soil and rock descriptions

Soil	Rock
Consistency or relative density	Colour
Fabric or fissuring	Grain size
Colour	Texture, fabric and structure
Subsidiary constituents	Weathered state and alteration state
Angularity or grading of principal soil type	Minor lithological characteristics
PRINCIPAL SOIL TYPE	ROCK NAME
More detailed comments on constituents or fabric	Estimated strength of the rock material
	Other terms indicating special engineering characteristics

In many cases, the soils will contain a mixture of particle sizes or materials which will influence their mechanical properties. Hence it is necessary to attempt to separate the subsidiary components of a soil from the principal component within the description. The name used for the soil in the description relates to the principal soil type and, in general, is based on particle size or the presence of organic material. The soil name thus provides valuable qualitative information concerning the physical and mechanical properties of the material. For example, the term 'sand' is indicative of relatively high permeability (provided the subsidiary component is not clay or silt) and relatively low compressibility, whereas the term 'clay' indicates relatively low permeability and relatively high compressibility. In granular soils (for example, sand or gravel) the soil name may be further refined by describing the shape of the particles (angularity). The object of each term within the soil description serves a specific purpose in attempting to define the engineering performance of the material as far as is possible without subjecting it to laboratory or *in situ* tests. Indeed, in some countries the engineering description of soils are used directly in geotechnical design and no provision is made for mechanical testing. The word order is necessary to provide a systematic framework for description. In summary, the components of the engineering description of soil may be considered as providing the following information:

- strength;
- variability and mass behaviour (e.g. anisotropy); and
- composition (indicative of permeability and compressibility).

For historical reasons the word order used in rock description is different from that used in the description of soil. Not only is this confusing, but the order used fails to highlight adequately the engineering properties in the same manner as for soil description.

Until recently, soil mechanics has considered strength to be of paramount importance and hence the descriptive terms relating to this property appear first in the word order. In rock engineering, rock material strength is of lesser importance than that of the rock mass. However, when describing intact rock the strength of the rock is possibly the most important parameter. In rock description the term relating to strength appears after the rock name. The first term in the word order is reserved for the least important parameter: colour.

It will be appreciated from the discussion on the geological classification of rocks that in many cases the rock name is derived from the texture, composition and grain size. It seems strange that texture and grain size feature so highly within the word order. The rock name may not be so indicative of engineering properties as a soil name is, but it generally is strongly indicative of texture and grain size. In certain cases some qualification of texture and/or grain size may be necessary in the description.

For example a sandstone may be composed predominantly of fine-grained sand particles and hence this should be stated in the description.

Fabric and structure will often play an important role in determining the engineering behaviour of the rock and thus should appear near the top of the word order.

The weathered state of a rock may be difficult or impossible to identify in a hand specimen. However the effects of weathering on the rock material will picked up in the other descriptors such as strength and colour. In many cases the most noticeable effect of weathering at the rock material scale will be discolouration and/or weakening adjacent to discontinuities. The problems involved in the description of weathering are discussed in detail later.

In the authors' opinion the word order should be changed such that it matches as far as possible that used for soil description and highlights better the more important engineering characteristics. The proposed new word order is shown below:

(a) estimated strength;
(b) fabric and structure;
(c) colour;
(d) lithological characteristics including where relevant:
    - texture,
    - grain size,
    - weathered state if known with certainty,
    - alteration state (e.g. kaolinized, hematized),
    - cementation state (e.g. weakly or strongly cemented),
    - type of cement (e.g. calcareous, ferruginous, siliceous),
    - subsidiary minerals (e.g. mica in sandstones),
    - porosity (e.g. porous or highly porous),
    - fossil content (e.g. fossiliferous or shelley);
(e) ROCK NAME;
(f) evidence of weathering (e.g. discolouration adjacent to discontinuities, including degree of penetration or loss of cement); and
(g) other terms indicating special engineering characteristics.

Here is an example of the above word order in use:

Moderately weak, bioturbated, light reddish brown, weakly cemented, calcareous fine SANDSTONE highly friable adjacent to discontinuities to a depth of 10mm.

The terms used in the conventional description of rock material are described below.

Colour

Colour is often the most noticeable feature of a rock but is possibly the most difficult to describe accurately and hence unaided assessments can be most misleading. It is normally associated with mineral composition of the constituent particles or the cementing material (in the case of sedimentary rocks) and hence should not be underrated. The colour of rock should be assessed objectively applying similar precautions to those mentioned in assessing the colour of soils. Wherever possible, colours should be compared with a standard chart, such as the rock colour chart produced by the Geological Society of America (1963) or the Munsell Soil Colour Chart (obtainable from Tintometer Ltd, Waterloo Road, Salisbury, England). Where standard charts are not available the simplified scheme proposed by the 1972 Working Party Report (Geological Society of London 1972) should be used (Table 2.17). For example, a rock colour might be described as dark greenish grey'.

Table 2.17 Rock colour

1	2	3
Light	Pinkish	Pink
Dark	Reddish	Red
	Yellowish	Yellow
	Brownish	Brown
	Olive	Olive
	Greenish	Green
	Bluish	Blue
		White
	Greyish	Grey
		Black

Grain size

Many of the common rock types are classified on the basis of grain size and hence the rock name often has an inherent grain size implication. It is recommended, however, that a descriptive grain size term should be included before the rock name. The same broad grain size ranges that are used for describing soils should be employed for rocks, as shown in Table 2.18.

Table 2.18 Grain size

Term	Particle size (mm)	Equivalent soil grade
Very coarse-grained	>60	Boulders and cobbles
Coarse-grained	2—60	Gravel
Medium-grained	0.06—2	Sand
Fine-grained	0.002—0.06	Silt
Very fine-grained	<0.002	Clay

For grains <0.06mm, grain boundaries are indistinct or below the threshold of visibility of the unaided eye.

Although it is recommended that the broad grain size bands be used (see Table 2.18), it would be more useful if the grain size subdivided further, particularly in the medium grain size (i.e. sand) range. For example, the rock name SANDSTONE implies a medium grain size but this could be qualified more by calling it a FINE SANDSTONE if the dominant grain size was in the fine sand range (0.06—0.2mm). In the above example the grain size qualifier is placed next to the rock name to avoid confusion. This approach is implied in the rock classification table (table 9) in BS 5930:1981. Of course, it can be argued that the inclusion of the grain size in the recommended form aids checking whether the correct rock name has been used.

Texture, fabric and structure

Texture may be defined as the geometrical aspect of the constituent particles or crystals together with the mutual relationship between them. In sedimentary rocks, texture refers to the size, shape and arrangement of the component mineral grains and in igneous and metamorphic rocks it deals with the crystallinity, granularity and the geometric relationships between the constituent minerals. The textural terms used by geologists are often complex and are frequently based on examination of thin sections under a microscope.

The most common textural terms are crystalline, glassy, granular or smooth. In most cases these may be sufficient for engineering use. However, these textures like grain size form the basis on which the rock has been named. The rock name therefore will suggest the texture. However variations in texture may be described using these terms.

The more complicated textural terms can be employed to a limited extent to provide a shorthand for descriptions. For example, an igneous rock may contain coarse- or medium-grained crystals in a finer-grained crystalline matrix. Such a texture could be described as porphyritic. A similar texture in a metamorphic rock would be termed porphyoblastic.

Rock fabric refers to the spatial arrangement and orientation of grains within the rock. In sedimentary rocks, fabric essentially deals with grain to grain relations, grain orientation, cementation and porosity. Fabric in igneous and other crystalline rocks refers to the pattern produced by the shapes and orientations of the crystalline and non-crystalline components of the rock.

In some cases rock fabric may not be recognized without the aid of a microscope. Examples of rocks fabric terms are homogeneous, schistose and lineated.

Structure refers to the larger scale inter-relationship of texture and fabric and is therefore often more noticeable. Common structural terms include foliated, massive, flow-banded, veined and homogeneous. In bedded sedimentary rocks, the individual beds may exhibit laminated, cross-laminated, graded, slump or bioturbated structures. The surfaces of bedding planes may be ripple-marked, sun-cracked or sole-marked.

BS 5930:1981 recommends that the descriptive terms shown in Table 2.19 be used for planar structures, such as bedding and lamination in sedimentary rocks. The terms and definitions used are based on the Engineering Group Working Party Report on the Preparation of Maps and Plans in Terms of Engineering Geology (1972).

Table 2.19 Spacing of planar structures

Term	Spacing
Very thickly bedded	>2m
Thickly bedded	600mm—2m
Medium bedded	200 mm—600 mm
Thinly bedded	60 mm—200 mm
Very thinly bedded	20mm—60mm
Laminated (sedimentary) Closely (metamorphic and igneous)	6rnm—20mm
Thinly laminated (sedimentary) Very closely (metamorphic and igneous)	<6mm

For sedimentary rocks, structures such as bedding may be described as thick beds or thickly bedded (for example thickly bedded limestone).

Weathered state

Rock material displays a wide variation in physical and mechanical properties. Some part of this variation is attributable to the different origins of the three principal groups of rocks, igneous, sedimentary and metamorphic. However, any rock which is brought into the near surface environment will be subject to structural, textural and mineralogical changes resulting from weathering which will affect its engineering properties. In general, rock material may lose strength, become more compressible and its permeability may increase or decrease. It is this near surface environment that yields the most variation in physical and mechanical properties of both rock material and the rock mass. It is also the environment in which most engineering works take place.

Weathering is that process of alteration and breakdown of rock occurring under the direct influence of the hydrosphere and the atmosphere, at or near the Earth's surface. This process results from

adjustment of the rock to the stress, chemical and biological conditions of the near surface environment and comprises physical disintegration (*physical weathering*) and chemical decomposition (*chemical weathering*). Where there has been little or no transport of altered or loosened material a complete weathering profile may be present in which a residual soil grades downwards through weathered rock into unaltered 'fresh' rock.

The processes of physical disintegration and chemical decomposition generally act together in weathering rock. However, weathering is strongly influenced by climate (rainfall and mean temperature), often with the result that one process is predominant. For example, in hot desert regions physical disintegration will be the dominant process, whereas chemical decomposition will dominate in a humid tropical region. However, it should be pointed out that the progress of chemical decomposition usually relies on fractures formed partly as a result of physical disintegration. Similarly, fractures may develop in response to changes in volume and weakening from chemical weathering.

The extremes of weathering are unaltered fresh' rock and residual soil. Intermediate states of weathering are difficult, if not impossible, to identify if the dominant weathering mechanism and the appearance of the end members are not known, particularly in weak rocks. The task is further complicated if the rock mass cannot be observed. In many cases engineers and geologists attempt to describe the weathered state of rock samples obtained from drillholes. A drillhole does not provide a sufficient volume of the rock mass to permit an accurate assessment of the state of weathering. Indeed, the weathered state of the rock obtained from drillholes is based almost entirely upon the condition of the rock material. In cases where physical weathering processes dominate, the rock material will appear relatively fresh even when the fracture state of the rock mass may indicate a high degree of weathering of the rock mass.

The main processes of chemical weathering all depend on the presence of water and may result in the alteration or dissolution of the component minerals grains. In the case of sedimentary rocks, the cement which binds the grains together is also prone to chemical attack. Typically, the chemical decomposition of the rock material starts at discontinuity walls and works inwards towards the centre of the intact blocks. This is often associated with discolouration penetrating the rock from the discontinuity walls. In cases were cement is removed by solution, the rock may be friable adjacent to discontinuities, and the zone of discolouration may be absent. The degree to which the discolouration or the removal of cement has penetrated the rock will indicate the degree of weathering. Of course, in cases where this zone has penetrated to the centre of blocks it may be difficult to determine whether the observed features are a product of weathering, or simply associated with the way in which the rock was formed (i.e. the rock is really fresh). Such problematic cases can only be resolved by making observations of the rock mass to define the overall weathering profile.

The effect of only slight or moderate chemical decomposition will be to influence the shear strength and compressibility of the discontinuities with little effect on the intact rock. Since in most rock masses the discontinuities control the engineering performance, the recognition of the early stages of chemical decomposition is clearly important. When the volume of chemically decomposed rock exceeds that of the fresh rock in intact blocks the rock material properties will be affected. It is likely that when the rock material is in such a highly weathered condition, the discontinuities will not have such a significant effect on the performance of the rock mass as would be the case if the rock material were fresh. This is particularly true with respect to compressibility.

In soluble rocks, chemical weathering may be recognized by the opening of discontinuity apertures and the presence of voids. Only the insoluble material is left behind. If the rock contains little or no insoluble material nothing is left behind except a void. Typically, the dissolution is associated with the passage of water through the discontinuities and hence the process results in an increase in aperture and a reduction in the degree of contact between adjacent discontinuity walls. This will affect the shear strength and compressibility characteristics of the rock mass as well as increasing the mass permeability.

Physical weathering of rock will generally cause the formation of new fractures, together with the opening of existing discontinuities. If the effects of chemical weathering are minimal the rock material will remain relatively fresh. In such cases the weathering may only be recognized from discontinuity spacing and aperture measurements. These measurements form an essential feature of rock mass descriptions. It will be the pattern of variation in spacing and aperture that will indicate the degree and stages of physical weathering. The decrease in discontinuity spacing and the general loosening of intact blocks of rock associated with this weathering process will have a significant influence on the performance of the rock mass.

From the above discussion it can be seen that the main indicators of weathering are as follows.

Chemical weathering

DECOMPOSITION

- Discolouration penetrating into the rock material from discontinuity walls.
- Formation of a zone of noticeably weaker rock (grains easily removed in the case of igneous rocks and granular sedimentary rocks or rock has become softened in the case of some mudrocks) penetrating inwards from discontinuities.

SOLUTION

- Removal of cement adjacent to discontinuity walls (in cemented sedimentary rocks).
- Widening of discontinuity apertures often with evidence of channelling.
- The presence of voids either associated with discontinuities or within the intact rock.

Mechanical weathering

- Formation of new fractures resulting in a reduction in discontinuity spacing and intact block size.
- Widening of discontinuity apertures.
- Loosening of the fracture block system.

The effects of chemical decomposition can generally be identified from intact blocks of rock seen in isolation from the rock mass. The degree of weathering may be determined from either by the amount that the zone of decomposition penetrates the rock material or from the ratio of the volume of rock to that of residual soil. In rock subject to solution or physical weathering the evidence of weathering and hence the determination of the degree of weathering can only be assessed by examination of the rock mass.

Both chemical and physical weathering processes will ultimately produce a residual soil in which the original texture, fabric and structure of the rock is destroyed. When such materials are observed they will be described as soils. They will only be recognized as the ultimate product of weathering through the establishment of a weathering profile within the rock mass.

Weathering normally takes place in a systematic fashion, such that highly weathered rock or residual soil may be found at or near the ground surface and this grades into fresh rock at depth, giving rise to a weathering profile. This simple pattern, however, may not always occur in reality due to local variations in rock type and geological structure. It is possible for weathered rock to pass laterally into unweathered rock and for discrete zones of weathered rock to exist below fresh rock. The depth of weathering is considerably variable ranging from centimetres to over 100 m.

In the schemes for the systematic description of rock material currently in use there is an expectation for the degree of weathering to be described. It is clear from the above discussion that when describing

rock material from a hand specimen or from a stick of core it may be extremely difficult or impossible to make a statement about the degree of weathering. Certain readily identifiable features of weathering such as discolouration can and should be incorporated into the rock description. Other features of weathering such as loss of strength will be incorporated within the overall description without necessarily being attributed to weathering. When the descriptions are brought together in order to study the rock mass the pattern of such features will aid in the identification of a weathering profile.

Attempts have been made at developing classification schemes which allow the degree of weathering to be defined for different lithologies (Anon 1970, Anon 1977, BS 5930 1981). The early schemes (Anon 1970, for instance) were based on the chemical weathering of granitic rocks and represented a hybrid material grade and mass zonal scheme. In 1977, the Working Party of the Engineering Group of the Geological Society on the Description of Rock Masses (Geological Society of London 1977) clearly separated the description of weathering on a rock material scale and on a rock mass scale. This scheme, like the earlier ones, placed great emphasis on the weathering profiles developed on granitic rocks in tropical and sub-tropical environments. Little guidance was given for the description of rock material weathering. BS 5930:1981 provided recommendations for the description of weathering of rock material. The British Standard proposed that weathered rock material may be described or graded using four terms: *decomposed*, *disintegrated*, *fresh* and *discoloured*, but provided no guidance for determining and describing the degree of weathering. Attempts to use these schemes in the description of rock material have met with difficulty. It is the opinion of the authors that any reference to degree of weathering should be omitted from the description unless it is known with some certainty on the basis of experience and knowledge of the typical weathering profile for that rock type. For rocks weathering in conditions where physical disintegration dominates, it is unlikely that the degree of weathering may be determined from examination of rock material alone.

Clear evidence of weathering should be included in the description of rock material. For example, in cases where discontinuity walls are discoloured or weakened these features, together with the distance they have penetrated, should be included.

The Engineering Group of the Geological Society has commissioned a Working Party to study the description and classification of weathered rocks for engineering purposes. The Report of this Working Party (1995) provides a scheme for describing the state of weathering for uniform rock materials which are moderately strong or stronger in the fresh state and which show a clear gradation in engineering properties during weathering. The proposed classification scheme requires the use of appropriate index tests such as the point load test and slaking tests.

The most logical approach to the problem of classifying the degree of weathering is to describe the rock material without attempting to provide a statement on how weathered it may be, apart from commenting on the presence of discolouration, decomposition, voids and softening. Once sufficient descriptive data on the rock material and the rock mass has been acquired to establish the mechanisms and stages of weathering present, a site specific weathering classification can be easily developed to provide a consistent means of describing both the rock material and, more importantly, the rock mass.

Alteration

Alteration refers to those changes in the chemical or mineralogical composition of a rock produced by the action of hydrothermal or other fluids. A common example of this phenomenon in granite rocks is the alteration of feldspars to form kaolinite. This is termed kaolinization. Other common forms of alteration are tourmalinization, mineralization, decalcification, and dolomitization. It is difficult to distinguish between the effects of weathering and alteration in some cases. In general, weathering effects die out at depth whereas alteration effects may be significant at great depth.

The engineering characteristics of the rock material and the rock mass can be drastically changed by alteration. For example, kaolinized granite is usually considerably weaker than unaltered granite.

Dolomitization in limestone is associated with a volume change which results in the formation of cavities.

In most cases, the descriptive terms used for weathering may be used to describe alteration. It should, however, be made clear in the description that the rock material has been subject to alteration in order to make the distinction between weathering and alteration.

Minor lithological characteristics

Minor lithological characteristics refer to the cementation state and cement type together with subordinate particle size and dominant mineral composition in the case of sedimentary rocks. In the case of igneous and metamorphic rocks, it refers to dominant or unusual mineral types. This section of the rock description may be used for noting unusual or interesting lithological features that are thought to be relevant to the engineering behaviour of the rock. Simple terms should be used where possible and defined if there is any likelihood of ambiguity. Terms should be quantified wherever possible.

In cemented rocks, descriptive terms should be used to describe the state of cementation. The scheme shown in Table 2.20 is recommended.

Table 2.20 Rock cementation

Term	Definition
Indurated	Broken only with sharp pick blow, even when soaked; makes hammer ring
Strongly cemented	Cannot be abraded with thumb or broken with hands
Weakly cemented*	Pick removes material in lumps which can be abraded with thumb and broken with hands
Compact*	Requires pick for excavation; 50mm peg hard to drive more than 50—100mm

* These materials may be treated as soil.

The cementation of sedimentary rocks takes place in two ways, first, by the enlargement of mineral grains by deposition of the same mineral on each grain surface in crystallographic continuity with the parent mineral grain, and secondly by the deposition of mineral matter in the pore spaces between grains. The descriptive terms for the common types of cement are shown in Table 2.21.

Table 2.21 Common types of cementing minerals

Composition of cement	Common form of cement	Term	Identification characteristics
Silica	Quartz Chalcedony	Siliceous	Rock normally hard and does not react with dilute HCl
Iron oxide	Limonite Haematite	Ferruginous	Mineral grains often stained brown or yellowish brown
Calcium carbonate	Calcite	Calcareous	Cement will react with dilute HCl

Other common terms used to describe minor lithological characteristics include clayey, marly, silty sandy, cherty shaly, clastic, bioclastic and metamorphosed.

Rock name

Rock names should be technically correct but simple enough for general and field uses. The scheme outlined in Tables 2.11, 2.12 and 2.13 is recommended. These are based on Dearman's scheme of petrographic description (Dearman 1974) which has been adopted (with some light alterations) by the Geological Society of London's Engineering Group Working Party in its report The Description of Rock Masses for Engineering Purposes (1977). Where necessary, a full petrographic analysis can be carried out at a later stage to enable a more petrographically correct rock name to be given.

Estimated strength of rock material

It is only absolutely essential to know the strength of rock material when describing massive rocks with little or no discontinuities, since in rocks which have discontinuities the behaviour of the rock mass is largely governed by these and not the rock material. It is useful, however, to have an assessment of rock material strength in the rock description particularly for assessing the shear strength of discontinuities (Barton 1973).

When describing rock cores or rock exposures, it is normally sufficient to estimate the strength. A scheme for estimating rock material strength based on the modified scheme of Piteau (1970) is recommended by the 1977 Working Party Report (Geological Society of London 1977). This scheme is shown in Table 2.22.

Table 2.22 Rock material strength

Term	Unconfined compressive strength MN/m^2 (MPa)	Field estimation of hardness
Very strong	>100	Very hard rock — more than one blow of geological hammer required to break specimen
Strong	50—100	Hard rock — hand held specimen can be broken with single blow of geological hammer
Moderate strong	12.5—50	Soft rock — 5mm indentations with sharp end of pick
Moderately weak	5.0— 12.5	Too hard to cut by hand into a triaxial specimen
Weak	1.25—5.0	Very soft rock — material crumbles under firm blows with the sharp end of a geological pick
Very weak rock or hard soil	0.60—1.25	Brittle or tough, may be broken in the hand with difficulty
Very stiff	0.30_0.60*	Soil can be indented by the finger nail
Stiff	0.15_0.30*	Soil cannot be moulded in fingers
Firm	0.08_0.15*	Soil can be moulded only by strong pressure of fingers
Soft	0.04_0.08*	Soil easily moulded with fingers
Very soft	<0.04*	Soil exudes between fingers when squeezed in the hand

* The unconfined compressive strengths for soils given above are double the undrained shear strengths.

Where possible, strength should be estimated using a simple field test such as the Schmidt hammer rebound test (Hucka 1965; Deere and Miller 1966; Hendron 1968; Aufmuth 1974; Dearman 1974) or the Point Load Test (Franklin *et al.* 1971; Broch and Franklin 1972; Bieniawski 1973, 1975; Aufmuth 1974; ISRM 1985). Of these two field tests, the point load test is the more reliable.

DESCRIPTION OF DISCONTINUITIES

Most rock masses are fractured, and in many cases these fractures form a distinct pattern of parallel or sub-parallel sets. These fractures, which are known collectively as discontinuities, are recognized by the fact that they have little or no tensile strength. In most cases it is the discontinuities that control the engineering performance of a rock mass and not the rock material. The degree to which the discontinuities control performance is related to the relative scale of the engineering works and the

spacing of the discontinuities in three dimensions. For example, in the case of a shallow foundation on rock, if the discontinuity spacing is significantly greater than the dimensions of the foundation, the performance of the foundation is likely to be relatively unaffected by the discontinuities. If however, the discontinuities have a spacing similar to or less than the foundation dimensions, then the discontinuities will play the dominant role. Another factor which will influence the performance of the rock mass is the orientation of discontinuities relative to the direction of the applied stresses. In the foundation example above, the applied stress is vertical and hence any horizontal or near horizontal discontinuities will have the greatest influence on the settlement characteristics of the foundation. The orientation of discontinuities also play an important role in controlling the stability of rock slopes.

Just as intact rocks are characterized by generic rock names, so discontinuities may be characterized by their generic type. Identification of the generic type is not always easy, even for the trained geologist. However, such a characterization may prove useful in predicting the extent and importance of a particular set of discontinuities. There are many classifications of discontinuities as to form, size and origin. This is indicative of the fact that there is no single classification system that is used in practice (Chernyshev and Dearman 1991). However two principal types of discontinuity may be readily identified:

1. discontinuities characterized by shear displacement; and
2. discontinuities characterized by very little or no shear displacement.

The most common discontinuities of type 1 are faults and shears. Faults may have relative displacements ranging from over a kilometre to less than a metre. They tend to occur in sets, often clustered within a fault zone, but may be widely spaced. Shears exhibit much smaller displacements than faults. Both faults and shears often have surfaces marked by slickensides or contain crushed rock, clay or sandy infill (gouge).

The most common types of discontinuity are characterized by very little or no shear displacement (type 2) and are generally referred to as joints. Most fractures of this type have formed in response to shear and tensile stresses associated with tectonism and hence may be related to geological structures such as folds. However, in igneous rocks fractures result from cooling and in all rock types fractures may be produced by stress relief as overburden is removed by erosion.

Discontinuities often occur in sub-parallel sets within a rock mass. It is these sets and systems that will generally control the engineering performance of the rock mass at shallow depths. These discontinuities generally include joints, cleavages, bedding planes, laminations and some tension cracks. The initial examination of a rock mass may not reveal all of the sets present, and in some cases it may not be possible to identify any sets at all. However, the systematic nature of these discontinuities permits statistical analysis (discussed later) allowing sets if present to be identified. Some discontinuities are unique and hence do not always occur in sets'. In some cases, sets may be found but are very limited in extent. Such discontinuities have to be considered individually. They include faults, shear planes, veins and some tension cracks. Although they may only affect a relatively small proportion of the rock mass they can nevertheless control engineering performance locally.

Since discontinuities play such a significant role in controlling the engineering performance of rock masses, it is essential that they are described carefully and systematically. Those parameters that can be used in some type of analysis should be quantified whenever possible.

For example, in the case of rock slope stability certain quantitative descriptions can be used directly in a preliminary limit equilibrium analysis. The orientation, location, persistence, and shear strength will be used in determining the most likely mechanisms of failure. This information together with joint water pressure and the shear strength of critical discontinuities will permit a preliminary limit equilibrium analysis to be carried out (Hoek and Bray 1981). For the purposes of a preliminary investigation these parameters can probably be estimated with reasonable accuracy from careful description of the discontinuities. Features such as roughness, wall strength, degree of weathering,

type of infilling material and signs of water seepage will provide important additional data for this engineering problem.

For the case of a shallow foundation on rock, orientation, spacing and aperture of discontinuities may be used in estimating the compressibility of the rock mass. In such a case attention should be paid to the orientation of discontinuities relative to the direction of the applied foundation load. Features such as roughness, wall strength and compressibility, degree of contact, degree of weathering and the type of infilling material will provide meaningful additional data for this engineering problem (Matthews 1993).

It will be noted that in the above examples there is a high degree of overlap in terms of those features that are regarded as important. Piteau (1970, 1973) lists the discontinuity properties that have the greatest influence at the design stage as follows:

- orientation;
- size;
- frequency;
- surface geometry;
- genetic type; and
- infill material.

Suggested methods for the description of discontinuities in rock masses are given by the Engineering Group of the Geological Society Working Party Report on the description of rock masses (1977) and the International Society for Rock Mechanics Commission on Standardization of Laboratory and Field Tests (ISRM 1978). The two schemes are compared in Table 2.23.

Table 2.23 Comparison of methods for description of discontinuities

Working Party Report (1977)	ISRM (1978)
Number of discontinuity sets	Orientation
Location and orientation	Spacing
Spacing	Persistence
Aperture	Roughness
Persistence	Wall strength
Infilling	Aperture
Nature of surfaces	Filling
Additional information	Seepage
Weathered and altered state	Number of discontinuity sets Block size

It will be seen from Table 2.23 that the two schemes for the systematic description of discontinuities are very similar. The major differences are in the order in which the features are described and some of the terms used. For example the Working Party Report (1977) suggests that the weathered or altered state of the discontinuities should be described. Essentially this means ‘to what extent has the discontinuity walls undergone deterioration by chemical weathering processes?’ Such processes are likely to bring about a change in strength and compressibility of the discontinuity walls. Instead of attempting to describe weathering the ISRM scheme focuses on the effects of weathering by suggesting that the wall strength be assessed. The ISRM scheme specifically asks for the seepage characteristics of fractures to be noted where possible. This important discontinuity property is included in the Working Party Report under the heading of additional information. The only feature not common to both schemes is the description of block size which forms part of the ISRM scheme.

Both the Working Party Report (1977) and ISRM scheme provide a good general introduction to the qualitative aspects of discontinuity measurement. However, these publications are limited by the fact that they do not incorporate data processing techniques, developed in the 1980s for the elimination of

sampling bias and the quantification of discontinuity characteristics. A comprehensive treatment of these aspects of discontinuity description may be found in Priest (1993).

Orientation

The orientation of discontinuities in a rock mass is of paramount importance to design in rock engineering.

The orientation of a discontinuity in space is described by dip direction (azimuth, three digits) measured clockwise in degrees from true north and by the dip of the line of steepest declination in the plane of the discontinuity measured in degrees from the horizontal (two digits), (for example: dip direction/dip (025°/52°). These measurements are normally made by means of a magnetic compass and clinometer device fitted with a spirit level (ISRM 1978). The majority of discontinuity surfaces are irregular, resulting in a significant amount of scatter of measurements being made over a small area. To reduce this scatter it is recommended that a 200mm diameter aluminium measuring plate be placed on the discontinuity surface before any measurement is made. In many cases, there may not be enough of the discontinuity surface exposed to allow the use of such a plate. If the exposure cannot be enlarged then a smaller plate must be used. A suitable combined compass and clinometer (geological compass) to which measuring plates can be attached is shown in Hoek and Bray (1981). Combined compass clinometers are available from a number of sources and range in price from about £30 to over £100. The most common type of geological compass is the Silva compass (type 15T). Some of the more expensive compass clinometers suffer from problems with the damping of the compass needle. Too much damping can lead to errors in dip direction measurements, whereas too little can make such measurements very time consuming. The process of making discontinuity orientation measurements is made more efficient by the use of a digital compass in combination with a digital clinometer. An electronic geological compass has been developed by F.W. Breithaupt and Son (Germany) which incorporates these features enabling the orientation of discontinuities to be measured and stored simply by placing the lid of the device on the discontinuity. The device has a resolution of 10 in both dip direction and dip, and is capable of storing up to 4000 measurements together with comments. These data may be transferred to a personal computer in the office for processing.

Many compasses have the capacity to correct for differences between magnetic north and true north. It is recommended that this adjustment is always set to zero; corrections can be made later during processing or plotting (Priest 1993). It should also be noted that compass needles balanced for magnetic inclination in the northern hemisphere will be severely out of balance in the southern hemisphere. Furthermore, electronic compasses set up for use in the northern hemisphere should not be used in the southern hemisphere.

Through careful use of the conventional geological compass and practice it is possible achieve a resolution of less than 30 seconds in dip and dip direction on readily accessible discontinuities (Priest 1993). However, Ewan and West (1981) conclude that different operators measuring the orientation of the same feature have a maximum error of ± 10° for dip direction and ±5° for dip angle.

Spacing

Discontinuity spacing is a fundamental measure of the degree of fracturing of a rock mass and hence it forms one of the principal parameters in the engineering classification of rock masses. In particular, for tunnelling this property has been used in the classification for support requirements (Barton *et al*., 1974; Bieniawski 1976) and for foundation settlement predictions on rock (Ward *et al.* 1968). The spacing of adjacent discontinuities largely controls the size of individual blocks of intact rock. In exceptional cases, a close spacing may change the mode of failure of the rock mass from translational

to circular. In such cases where the joints are extremely closely spaced the rock mass will tend to behave like a granular soil and joint orientation is likely to be of little consequence.

Discontinuity spacing may be considered as the distance between one discontinuity and another. More specifically ISRM (1978) defines discontinuity spacing as the perpendicular distance between adjacent discontinuities. It is easier when collecting spacing data in the field to adopt the former more general definition. For example, a random sample of discontinuity spacing values may be obtained from a linear scanline survey (described later). Such a survey provides a list of the distances along the scanline to the points where it is intersected by the discontinuities which have been sampled. Subtraction of consecutive intersection distances provides the discontinuity spacing data. Perpendicular discontinuity spacing data may be determined during data processing in the office. However it is more meaningful if such spacings are determined for discontinuities of the same type (e.g. the same discontinuity set).

Priest (1993) defines three different types of discontinuity spacings.

1. *Total spacing*: The spacing between a pair of immediately adjacent discontinuities, measured along a line of general, but specified, location and orientation.
2. *Set spacing*: The spacing between a pair of immediately adjacent discontinuities from a particular discontinuity set, measured along a line of any specified location and orientation.
3. *Normal set spacing*: The set spacing when measured along a line that is parallel to the mean normal to the set.

The mean and range of spacings between discontinuities for each set should be measured and recorded. Ideally these measurements should be made along three mutually perpendicular axes in order to allow for sampling bias. Where discontinuity sets are readily identifiable in the field the normal set spacing of each set may be recorded in terms of the maximum, minimum and modal (most frequent) or mean spacing. A comprehensive treatment of the statistical analysis of discontinuity spacing and frequency is given by Priest (1993).

Descriptive terms for discontinuity spacing given by the Engineering Group Working Party Report on the Description of Rock Masses (Geological Society of London 1977), ISRM (1978) and BS 5930:1981. These are compared in Fig. 2.3. Typically, these descriptive terms will be applied in the field to normal set spacings.

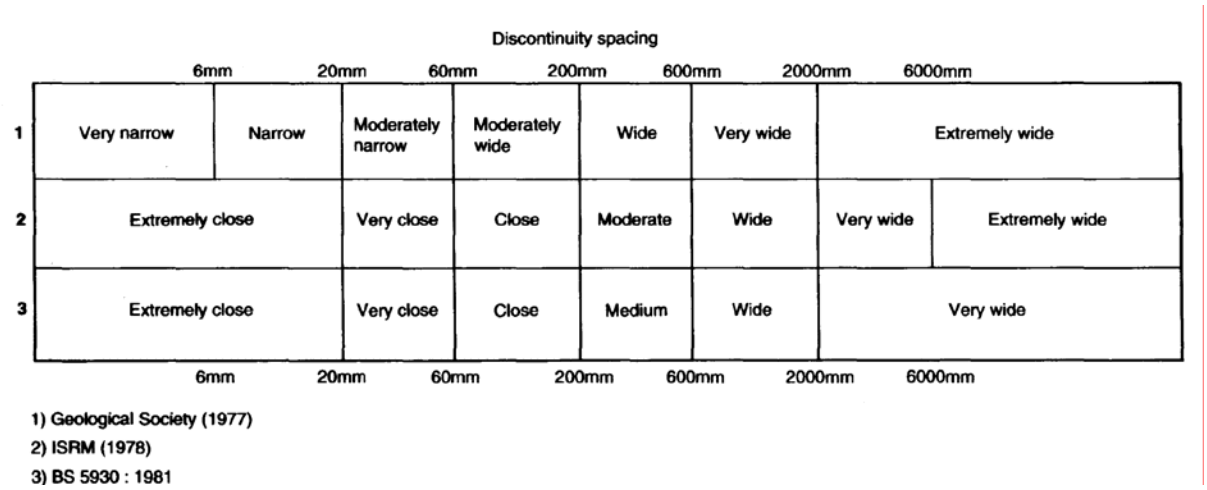


Fig. 2.3 Discontinuity spacing classification schemes: (1) Geological Society (1977): (2) ISRM (1978); (3) BS 5930: 1981.

It will be seen from Fig. 2.3 that the descriptive terms used in each document are similar but refer to different spacing classes. The Geological Society classification gives more emphasis to the more

closely spaced fractures, whereas the ISRM classification places most emphasis on the widely spaced fractures. The British standard follows closely that recommended by the ISRM but omits the extremely wide category.

Clearly whichever classification one chooses to use it is important to state the source of classification or give the definitions of the descriptive terms used.

Persistence

Persistence refers to the discontinuity trace length as observed in an exposure. It is one of the most important factors in discontinuity description, but unfortunately it is one of the most difficult to quantify. One of the common problems that arises is the measurement of the persistence of major joints which are continuous beyond the confines of the rock exposure. It is recommended that the maximum trace length should be measured, and comment made on the data sheet to indicate whether the total trace length is visible and whether the discontinuity terminates in solid rock or against another discontinuity. Clearly, persistence is very much scale dependent and any measurements of persistence should be accompanied by the dimensions of the exposure from which the measurements were made.

It is helpful when collecting discontinuity persistence data in the field to set up a simple classification scheme based on trace length and type of termination. The trace length used in such a classification will vary from exposure to exposure according to the extent of the exposed rock in each case. Therefore it will be necessary either to define the categories each time or express the trace lengths as a percentage of the maximum possible trace length.

Matherson (1983) considers persistence to be a fundamental feature in quantifying the relative importance of discontinuities in a rock mass. BS 5930:1981 does not place sufficient emphasis on the observation and measurement persistence.

Wall roughness

The wall roughness of a discontinuity is a potentially important component of its shear strength, particularly in the case of undisplaced and interlocked features. In terms of shear strength, the importance of wall roughness as aperture, or infilling thickness or the degree of displacement increases. In cases where adjacent walls are not fully interlocked or mated the wall roughness will directly influence the degree of contact which in turn effect the compressibility of the discontinuity.

In general, the roughness of a discontinuity can be characterized by the following.

1. *Waviness*. First order wall asperities which appear as undulations of the plane and would be unlikely to shear off during movement. This will affect the initial direction of shear displacement relative to the mean discontinuity plane.
2. *Roughness*. Second order asperities of the plane which, because they are sufficiently small, may be sheared off during movement. If the wall strength is sufficiently high to prevent damage these second order asperities will result in dilatant shear behaviour. In general this unevenness affects the shear strength that would normally be measured in a laboratory or medium scale *in situ* shear test.
3. *Condition of the walls*. Description of rock material forming discontinuity faces. Special attention should be given to weak zones in walls produced by weathering or alteration.

Waviness may be measured by means of a standard tape or rule placed on the exposed discontinuity surface in a direction normal to the trend of the waves. The orientation of the tape, together with the mean wave length and maximum amplitude should be recorded. In some cases it may be necessary to

assess the waviness in three dimensions in which case a compass and disc clinometer are recommended.

Roughness may be assessed by profiling the discontinuity surface. Short profiles (<150mm) can be measured using a profiling tool which is obtainable in most DIY stores. Longer profiles may be measured using a 2 m rule as described by ISRM (1978).

A number of quantitative techniques for measuring waviness and roughness are described in detail by ISRM (1978). No guidance on the measurement of wall roughness is given in BS 5930:1981.

Quantitative techniques for assessing roughness can be time consuming, and for preliminary rock mass surveys a qualitative assessment making use of simple descriptive terms should be employed. ISRM (1978) recommends a nine point visual classification shown in Table 2.24 which is based on two scales of observation: small scale (several centimetres); and intermediate scale (several metres).

Table 2.24 Roughness categories

Category	Degree of roughness
I	Rough (or irregular), stepped
II	Smooth, stepped
III	Slickensided, stepped
IV	Rough (or irregular), undulating
V	Smooth undulating
VI	Slickensided, undulating
VII	Rough (or irregular), planar
VIII	Smooth, planar
IX	Slickensided, planar

The term 'slickensided' should only be used if there is clear evidence of previous shear displacement along the discontinuity.

It should be pointed out that such a scheme for describing roughness is meaningful only when the direction of irregularities in the surface is in the least favourable direction to resist sliding. It is also necessary to specify the trend of the lineation on the surface of the discontinuity in relation to the direction of shearing. Furthermore, it is recommended that at each site where this scheme is used, typical examples of each category should be identified and photographed to maintain uniformity of assessment and hence make the scheme more objective.

Wall strength

Wall strength refers to the equivalent compression strength of the adjacent walls of a discontinuity. This may be lower than the intact strength of the rock owing to weathering or alteration of the walls. The relatively thin 'skin' of wall rock that affects shear strength and compressibility can be tested by means of simple index tests. The apparent uniaxial compressive strength can be estimated from Schmidt hammer tests (Barton 1973) and from scratch and geological hammer tests, since the latter have been roughly calibrated against a large body of data. It is recommended that such tests be carried out on freshly broken rock surfaces such that the estimated wall strength may be directly compared with that of the intact rock. It is likely that the intact strength may be measured in the laboratory as part of the investigation and this will provide a means of calibrating these somewhat crude field measurements. The descriptive terms used for intact strength discussed earlier may be applied to the description of wall strength.

Wall strength may also be assessed in terms of weathering grade. ISRM (1978) recommends a set of descriptive terms for both the discontinuities and the rock mass as a whole. The same scheme has been adopted by BS 5930:1981 for the general description of weathering (or alteration) of the rock material and the rock mass.

Aperture

Aperture is the perpendicular distance separating the adjacent rock walls of an open discontinuity, in which the intervening space is air or water filled. Discontinuities that have been filled (for example, with clay) also come under this category if the filling material has been washed out locally.

Large apertures may result from shear displacement of discontinuities having a high degree of roughness and waviness, from tensile opening resulting from stress relief, from outwash ‘and dissolution. Steep or vertical discontinuities that have opened in tension as a result of valley formation or glacial retreat may have extremely wide apertures measurable in tens of centimetres.

In most sub-surface rock masses, apertures may be closed (i.e. <0.5mm). Unless discontinuities are exceptionally smooth and planar it will not be of great significance to shear strength that a ‘closed’ feature is 0.1mm wide or 1.0 mm wide. Such a range of widths however may have a greater significance with respect to the compressibility of the rock mass. Where compressibility is concerned it is important to describe the degree of contact across adjacent rock walls in addition to the aperture observations.

Large apertures may be measured with a tape of suitable length. The measurement of small apertures may require a feeler gauge. Details of measurement techniques may be found in ISRM (1978). The descriptive terms recommended by ISRM (1978) are given in Table 2.25.

Table 2.25 Discontinuity apertures

Aperture	Description	Features
<0.1 mm	Very tight	
0.1—0.25 mm	Tight	Closed
0.25—0.5 mm	Partly open	
0.5—2.5 mm	Open	
2.5—10 mm	Moderately open	Gapped
>10 mm	Wide	
1—10 cm	Very wide	
10—100 cm	Extremely wide	Open
>1 m	Cavernous	

Filling

Filling refers to material that separates the adjacent rock walls of a discontinuity and that is usually weaker than the parent rock. Typical filling materials are sand, silt, clay, breccia, gouge and mylonite. Filling may include thin mineral coatings and healed discontinuities. Mineral coatings such as chlorite can result in a significant reduction in shearing resistance of discontinuities (Hencher and Richards 1989). In general, if the filling is weaker and more compressible than the parent rock its presence may have a significant effect on the engineering performance of the rock mass. The drainage characteristics of the filling material will not only affect the hydraulic conductivity of the rock mass but also the long- and short-term mechanical behaviour of the discontinuities since the infill may behave as a soil.

ISRM (1978) suggests that the principal factors affecting the physical behaviour of infilled discontinuities are as follows:

1. mineralogy of filling material;
2. grading or particle size;
3. overconsolidation ratio (OCR);
4. water content and permeability;
5. previous shear displacement;

6. wall roughness;
7. width of infill; and
8. fracturing or crushing of wall rock.

If the thickness of the infill exceeds the maximum amplitude of the roughness the properties of the infill will control the mechanical behaviour of the discontinuity. Clearly the wall roughness and the thickness or width of infill must be recorded in the field. An engineering description of the infill material should be made in the field and suitable samples taken for laboratory tests. The infill should be carefully inspected in the field to see whether there is any evidence of previous movement (for example, slickensides) since this is likely to reduce the shearing resistance of the fracture significantly.

Seepage

Water seepage through rock masses results mainly from flow through discontinuities ('secondary permeability') unless the rock material is sufficiently permeable such that it accounts for a significant proportion of the flow. Generally it should be noted whether a discontinuity is dry, damp or wet or has water flowing continuously from it. In the latter case the rate of flow should be estimated. Of course such observations are dependent upon the position of the water table and the prevailing weather conditions. It is important to note whether flow is associated with a particular set of discontinuities. ISRM (1978) gives a set of descriptive terms that may be applied to seepage observations for filled and unfilled discontinuities.

Number of sets

The appearance of the rock mass together with its mechanical behaviour will be strongly influenced by the number of sets of discontinuities that intersect one another. The appearance of the rock mass is affected since the number of sets tends to control the degree of overbreak in excavations. The number of sets also affects the degree to which the rock mass can deform without failure of intact rock. In tunnelling, three or more sets will generally result in a three-dimensional block structure.

A number of sets may be identified by direct observation of the exposure. However the total number of sets present in the rock mass is normally determined from a statistical analysis of the discontinuity orientation data (Matherson 1983; Priest 1985; 1993).

The number of joint sets comprising the intersecting joint system. The rock mass may be further subdivided by individual discontinuities such as faults.

Block size

Block size is an important indicator of rock mass behaviour. Rock masses comprising relatively large blocks tend to be less deformable than those with small blocks. In the case of underground excavations such rock masses generally develop favourable arching and interlocking. In the case of slopes, a small block size may result in ravelling or circular failure brought about by the rock mass behaving like a granular soil.

The block size is determined from the discontinuity spacing, number of sets and persistence. The number of sets and the orientation of discontinuities will determine the shape of the resulting blocks. However, since natural fractures are seldom consistently parallel regular geometric shapes such as cubes, rhombohedrons and tetrahedrons rarely occur.

BS 5930:1981 recommends the descriptive terms for block size and shape given in Table 2.26. A more quantitative approach to block size description is given by ISRM (1978).

Table 2.26 Block size and shape

First term (size)	Maximum dimension
Very large	>2 m
Large	600 mm—2 m
Medium	200 mm—600 mm
Small	60 mm—200 mm
Very small	<60 mm
Second term (shape)	**Nature of block**
Blocky	Equidimensional
Tabular	One dimension considerably smaller than the other two
Columnar	One dimension considerably larger than the other two

Other descriptive terms which give an impression of the block size and shape include:

Massive	Few fractures or very wide spacing
Irregular	Wide variations of block size and shape
Crushed	Heavily jointed to give medium gravel size lumps of rock.

METHODS OF COLLECTING DISCONTINUITY DATA

The method used in collecting discontinuity data will depend largely upon the engineering application and the degree of access to the rock mass. In some cases, for example tunnelling, discontinuity data are required at some depth below the ground surface. Surface exposures may be available but may not be representative of the rock mass at the depth of interest, owing to weathering agencies (for example, stress relief causing reduction in joint spacing and an increasing in aperture). Access to the rock mass at the depth of interest can only be achieved through the use of trial adits, shafts or drillholes. Trial adits and shafts are expensive and hence in the majority of cases the preferred method of access is by drillholes. In other cases, for example shallow foundations and rock slope design, the necessary rock mass information may be obtained from surface exposures if available. Drillhole information may be used to supplement the data obtained from surface exposures. Where surface exposures are not available, or are considered to be unrepresentative of the rock, mass drillholes alone may be the only source of data. Table 2.27 shows how the quality of discontinuity data is affected by the type of access to the rock mass and the type of survey method used.

It may be seen from Table 2.27 that access to the rock mass via a drillhole suffers from a number of disadvantages. First, a drillhole permits only a small volume of the rock mass to be viewed such that the persistence of discontinuities cannot be adequately assessed. Secondly the orientation of the core must be known before any fracture orientation measurements can be made. Also, drillholes are prone to directional biasing of discontinuity data unless they are drilled with different orientations. For example, if only vertical drillholes are employed any vertical or near vertical sets of discontinuities may be missed altogether or a false impression may be given with respect to the frequency of these fractures. The only way to overcome this problem is to drill inclined holes at a number of different orientations. Terzaghi (1965) and Priest (1993) discuss methods of dealing with directional biasing and Priest (1985) discusses methods of analysing orientated core.

It is impossible to measure aperture from drillhole core since it is inevitable that the sticks of core will have moved relative to one another during and after sampling. The only way of measuring aperture in this case is by inspection of the drillhole wall. This is achieved using a borehole impression packer or a borehole television camera. The borehole impression packer has been used with success in hard rocks such as granite. In weak rocks however there is a tendency for the borehole wall to be eroded by the cuttings as they are brought to the surface. This is particularly so in the chalk where small

fragments of flint can be very effective in eroding the drillhole wall, making the interpretation of the impression packer data very difficult.

Table 2.27 Quality of information from different types of discontinuity survey and access to the rock mass (based on Geological Society of London Working Party Report on the Description of Rock Masses (1977))

Type of information	Direct measurement (surface exposure, trial adit or shaft)	Surface photography	Drillhole core	Orientated drillhole core	Drillhole camera	Drillhole impression packer	Geophysics acoustic methods
Location	Good	Good	Good	Good	Good	Good	Medium
Type of discontinuity	Good	Medium	Good	Good	Good	Poor	Poor
Description of rock material	Good	Poor	Good	Good	Poor	None	None
Orientation: dip	Good	Medium	Medium/Poor	Good	Poor	Good	Poor
Orientation: dip direction	Good	Medium	Poor	Medium	Medium	Medium	Poor
Spacing	Good	Good	Medium	Medium	Medium	Medium	Poor
Persistence	Good	Good	Poor	Poor	Poor	Poor	Poor
Wall roughness: waviness	Good	Medium/Poor	Poor	Poor	Poor	Poor	Poor
Wall roughness: roughness	Good	Medium/Poor	Medium	Medium	Poor	Poor	Poor
Wall strength	Good	None	Medium	Medium	None	None	None
Aperture	Good	Poor	Poor	Poor	Medium	Medium	Medium/Poor
Infill: nature	Good	Poor	Medium	Medium	Poor	Poor	None
Infill: thickness	Good	Poor	Medium/Poor	Medium/Poor	Medium	Poor	Poor
Seepage	Good	Medium	None	None	Medium	None	Poor
Number of sets	Good	Good/Medium	Poor	Medium	Medium	Medium	None
Block size	Good	Good/Medium	Poor	Medium	Medium	Medium	Poor

Ranking: Good feature measured reliability;
Medium feature measured but not easily and often with poor reliability;
Poor feature difficult to measure, often measurement is inferred;
None impossible to identify feature.

Discontinuity infill may be washed out or contaminated by the drilling fluid such that it becomes difficult to assess its thickness or properties adequately. Mineral coatings on joint walls may be observed in core samples. However, it is impossible to assess the degree of coverage from such a small sample.

Adits, shafts and surface exposures can offer a much larger expanse of rock for examination and permit direct observation and measurement of discontinuities. However, they can be just as prone to directional biasing as the drillhole if only a single orientation of adit or exposed face is available. These forms of access clearly have distinct advantages over the drillhole, but the trial adit or shaft may prove too expensive and the surface exposure may be unsuitable due to inadequate size. Despite the obvious disadvantages of the drillhole, the fracture state of the rock mass may be determined in sufficient detail to permit classification of the rock mass. Where surface exposures have revealed one or more discontinuity sets drillholes and drillhole core may be used effectively to check whether these persist at depth.

DISCONTINUITY SURVEYS

The rock mass contains a considerable amount of geometrical information which must be collected, filtered and interpreted. Clearly an irregular highly fractured rock face presents a somewhat daunting challenge to anyone who wishes to quantify the rock structure or discontinuity network in an unbiased

manner. It is important, therefore, to ensure that measurement systems are based upon objective but flexible sampling strategies linked to rigorous data analysis based upon the principles of geometrical probability and statistics (Priest 1993). Typically between 1000 and 2000 discontinuities should be sampled to provide an adequate characterization of a site (Priest and Hudson 1976). This number is generally made up from samples between 150 and 350 discontinuities taken at between 5 and 15 sample locations chosen to represent the main zones based on geological structure and lithology. In some cases the extent of the site or the exposed rock makes such large numbers of measurements impractical or impossible. In such cases the minimum sample of 200 discontinuities should be taken.

The method of collecting discontinuity data will vary according to the type of access to the rock mass. The two broad sampling strategies that can be adopted involve either the logging of drillhole core or the examination of an exposed rock face.

When using drillholes, a detailed fracture log of the core is required together with an inspection of the drillhole wall. In the case of exposed rock faces above or below ground the most widely used sampling methods include scanline sampling (ISRM 1978; Priest and Hudson 1981; Priest 1993) and window sampling (PahI 1981; Priest 1993).

Fracture logging of drillhole core

A common problem in logging fractures in core samples or in a rock face is identification of artificial fractures resulting from the drilling process or by the creation of the face (blasting and stress relief). These fractures are normally excluded from the log, unless a conscious decision is made to the contrary which should be clearly stated on the log. A degree of judgement is therefore required. Artificial and natural fractures can often be distinguished from each other by observing the freshness, brightness, staining and erosion of the fracture surface. For example, in the chalk natural fractures often exhibit manganese spots or dendritic patterns and relatively smooth surfaces, whereas artificial fractures are clean and rough.

Every natural fracture which cuts the core should be described in the following manner.

1. The position of the fracture in the core sample should be recorded. A pictorial log of the fractures cutting the drillhole may be made from this information.
2. The angle the fracture makes with the core axis should be noted. Where the orientation of the core is known the dip and dip direction of the fracture should be determined.
3. The roughness of the fracture surfaces, should be noted.
4. If any infill is present, its thickness and nature should be described.
5. The presence of any mineral coatings should be noted.
6. Where possible or practical the compressive strength of the fracture surface should be determined using a Schmidt hammer. A qualitative assessment of the wall strength may be made by indenting the wall and the side of the core using the point of a knife, pick or other sharp implement.

The average spacing of discontinuity sets identified from the fracture log may be determined from the recorded positions of the relevant fractures. The general fracture state of the rock mass may be assessed from the determination of the total and solid core recovery, the fracture index, Rock Quality Designation (RQD) and Lithological Quality Designation (LQD). These parameters are defined in the section on Logging Rock Cores.

Scanline sampling

Although many different techniques have been described for sampling discontinuities in rock

exposures (Muller 1959; Pacher 1959; Da Silveria *et al.* 1966; Knill 1971) the line or scanline approach is preferred (Piteau 1970; Broadbent and Rippere 1970) on the basis that it is indiscriminate (all discontinuities whether large or small should be recorded) and provides more detail on discontinuity spacing (Priest and Hudson 1976, 1981) and attitude than other methods. However there is currently no universally accepted standard for scanline sampling.

In practice, a scanline survey is carried out by fixing a measuring tape to the rock face by short lengths of wire attached to masonry nails hammered into the rock. The nails should be spaced at approximately 3 m intervals along the tape which must be kept as taut and as straight as possible. The face orientation and the scanline orientation should be recorded along with other information such as the location, date, and the name of the surveyor. Where practicable the face and scanline, including a scale and appropriate label, should be photographed before commencing the sampling process. In cases where the face is irregular it will be necessary to take photographs from several viewpoints. A simple way to provide a scale is to attach clearly visible markers at 1 m intervals along the length of the scanline. Care should be taken to minimize distortion of the face on the photographs.

Once the scanline is established the surveyor works systematically along the tape recording the position and condition of every discontinuity that intersects it. The features that are commonly recorded include the following.

1. *Intersection distance*. This is the distance in metres (rounded to the nearest cm) along the scanline to the intersection point with the discontinuity. Where the face is irregular it will be necessary to project the plane of fractures not in contact with the tape on to the tape such that the position of such fractures can be accurately recorded. In highly irregular faces this method can lead to significant errors in the determination of joint spacing. Ideally a clean, approximately planar rock face should be selected for scanline sampling.
2. *Orientation*. This is the dip direction and dip of the discontinuities.
3. *Semi-trace length*. This is the distance from the intersection point on the scanline to the end of the discontinuity trace. The distance may be measured directly, estimated by eye or scaled from a photograph of the rock face, when it becomes available. There will be two semi-trace lengths associated with each discontinuity: one above and one below for a horizontal scanline; one to the left and one to the right for an inclined or vertical scanline.
4. *Termination*. It is helpful to record the nature of the termination of each semi- trace. The scheme recommended by ISRM (1978) has proved to be adequate:
   ‘I’ or 1 Discontinuity trace terminates in intact rock material
   ‘A’ or 2 Discontinuity trace terminates at another discontinuity
   ‘O’ or 3 Termination obscured or trace extends beyond the limits of the exposure.
5. *Roughness*. A profile of the discontinuity surface roughness may be made in the manner described earlier or the Joint Roughness Coefficient (JRC) (Barton 1973) may be estimated visually.
6. *Curvature*. This refers to surface irregularities with a wavelength greater than about 100 mm. This can be determined by measuring offsets at 100mm intervals along a straight base line, then digitizing and quantifying the resulting profile (ISRM 1978). This can be assessed visually using the classification scheme given in Table 2.28.

Other features such as type of discontinuity, nature of infill, aperture, water flow, slickensides are generally reported in a comments column on the logging sheets.

Further scanlines should be set up on a second rock face, approximately at right angles to the first, to minimize the orientation sampling bias. Errors may arise from sampling a single line since those discontinuity sets which have an orientation similar to that of the face and those whose traces are nearly parallel to the scanline are likely to be missed in the survey as a result of directional or orientation sampling bias. Corrections have been devised to compensate for directional bias (Terzaghi

1965; Robertson 1970) but these will not aid the identification of joints sets which intersect the scanline at low angles (<10°).

Table 2.28 Roughness categories

Category	Degree of roughness
1	Polished
2	Slickensided
3	Smooth
4	Rough
5	Defined ridges
6	Small steps
7	Very rough

The length of the scanline should be at least fifty times the mean discontinuity spacing (Priest and Hudson 1976) in order to estimate the frequency of discontinuities to a reasonable degree of precision. Priest (1993) recommends that the scanline should contain between 150 and 350 discontinuities, of which about 50% should have at least one end visible. These criteria have implications on the minimum size of exposed rock face that can be representatively sampled. Often a compromise must be sought between these criteria owing to size of face or restrictions in the access to parts of the face.

West (1979) investigated the reproducibility of measuring joint frequency in the Lower Chalk. The position along a measuring tape of all joints intersected by the scanline were recorded by six observers. Very small fractures (less than about 0.1 m in length), shattered rock exhibiting fine crazing and rubble or screen zones were disregarded. Different observers produced different joint frequency diagrams, indicating the subjective nature of the method of measurement used. The element of judgement required in deciding which fractures to ignore is considered to be the primary cause of the differences in the records. Ewan *et al*. (1981) carried out a similar study in limestone, sandstone and mudstone exposed in the Kielder Aqueduct Tunnels. They found that the number of joints recorded could vary by up to a factor of four for different observers measuring the same scanline. However, in the case of this study, the 10 m scanline lengths used were typically less than the minimum recommended by Priest and Hudson (1976) (i.e. 50 times the mean joint spacing) which may have contributed to the variation in joint count between observers. Clearly it is necessary to reduce the subjective element of joint measurement by measuring all fractures that cross the scanline. However, this may prove to be too time consuming and indeed unrepresentative fractures such as those induced by the excavation method would be included. The subjectivity cannot therefore be eliminated, only minimized.

Window sampling

Window sampling provides an area based alternative to the linear sampling techniques outlined above which reduces the sampling biases for discontinuity orientation and size. The preliminaries and measurement techniques are essentially the same as for scanline sampling except that all discontinuity traces which across a defined area of the rock face are measured.

The sampling window may be defined by setting up a rectangle of measuring tapes pinned to the rock face. In order to minimize sampling bias effects it is recommended that the window should be as large as possible, with each side of a length such that it intersects between 30 and 100 discontinuities (Priest 1993). Where possible, two windows of similar size should be set up on mutually perpendicular faces.

In general, areal sampling provides a poor framework within which to collect orientation, frequency and surface geometry data for individual discontinuities. The window is likely to contain a large number of relatively small discontinuities, making it difficult to keep track of which discontinuities

have been measured. The process of window sampling is generally more laborious than scanline sampling when attempting to apply the same rigorous sampling regime.

Face sampling

At the preliminary stage of an investigation it is often necessary simply to establish the number of discontinuity sets, the average orientation of each set and the relative importance of each set. Matherson (1983) suggests that for simple rock slope stability assessments at the preliminary site investigation phase it is sufficient to observe persistence, aperture and infilling in addition to orientation. In such cases it may be expedient to adopt a sampling strategy that is less rigorous than those outlined above.

Face sampling involves recording the orientation, persistence, roughness, aperture, filling and seepage of a representative number of discontinuities in the exposed face. A suitable geological compass should be used for the orientation measurements and a suitable classification or coding system should be used for the other parameters. The size of the sample must be large enough to ensure statistical reliability. A minimum of 200 measurements per face is recommended by the Engineering Group Working Party Report, *The Description of Rock Masses for Engineering Purposes* (Geological Society of London 1977) whereas a minimum of 150 measurements is recommended by ISRM (1978). Such large sample sizes may preclude the measurement of all the parameters listed above for each fracture. An evaluation of the importance of each discontinuity however should be made.

Face sampling is generally non-systematic and tends to concentrate on discontinuity orientation which is adequate for preliminary investigations, where the information should be kept simple. Discontinuity spacing cannot be readily determined unless sets are recognizable in the face or scanline sampling is carried out. The data from the face survey may be easily analysed by hand and can be used to provide a rapid assessment of stability of rock cuts. It should be pointed out that the data from a face survey alone are insufficient for rigorous stability analysis.

The grouping of data collected from a very large exposure or from a number of different locations may obscure discontinuity patterns. Ideally the area studied should be divided into units, domains or structural regions (Piteau 1973). Data from each face should be assessed separately. If it can be established that each face shows a similar discontinuity pattern it may then be possible to group the data from a number of faces. If different rock types are present in the survey area the discontinuities observed in each type should be placed in separate groups. Collection of data in domains allows individual or grouped assessment. This is likely to be of major importance where alignment, design or rock type change along a proposed route.

PRESENTATION OF DISCONTINUITY DATA

Discontinuity data should be presented in a form that allows ease of assimilation and is amenable to rapid assessment. Discontinuities may be shown on maps, scale drawings of exposures or block diagrams which may be used to indicate the spatial distribution and interrelationships of these features. Such methods, although very useful, do not allow the quantitative assessment of orientation and spacing which are perhaps the most important aspects of discontinuities within a rock mass. In many cases, it is not possible to recognize all the discontinuity sets or assess the dominant orientation of some discontinuity sets in the field. Such factors may only be assessed by statistical analysis of discontinuity orientation data.

Discontinuity orientation data are easier to visualize and analyse if presented graphically. The simplest form is a rose diagram, which shows the frequency of fractures as a function of their dip direction, but says nothing about their dip magnitude. It is therefore difficult to identify joint sets using this approach. The most commonly used method of presenting orientation data is the hemispherical

projection. This method allows the distribution of dip and dip direction to be examined simultaneously and provides a rapid visual assessment of the data as well as being readily amenable to statistical analysis. Although this method is used extensively by geologists, it is little understood by engineers, since it bears no recognizable relationship to more conventional engineering drawing methods. The basis of the method and its classic geological applications are described by Phillips (1971). Rock engineering applications are described in detail by Goodman (1976), Hoek and Brown (1980), Priest (1980, 1985, 1993), Hoek and Bray (1981) and Matherson (1983).

Hemispherical projection methods

The principle of hemispherical projection methods is that the orientation of a line in three-dimensional space is uniquely represented by a point within a two-dimensional area. Hemispherical projections may be divided into the following two main types.

1. *The equal-angle projection*. This projection accurately preserves the angular relationships between features. The data are plotted on an equatorial equal- angle net (Fig. 2.4a).
2. *The equal-area projection*. This projection preserves the spatial distribution of features. The data are plotted on an equatorial equal-area net (Fig. 2.4b).

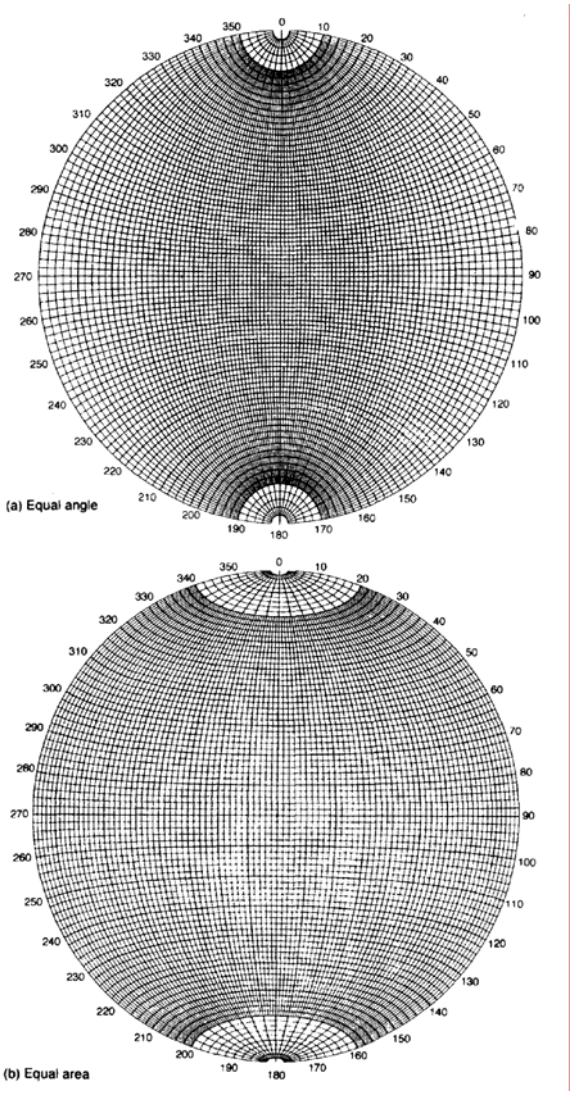


Fig. 2.4 Examples of equitorial nets for plotting discontinuity orientation data: (a) equal angle; (b) equal area.

It is important, when presenting orientation data for statistical analysis, that both the angular relationships and the spatial distribution of the discontinuities are accurately represented. Clearly this is not possible using a single hemispherical projection. Furthermore, not all hemispherical projections are suitable for statistical analysis. The equal-area projection permits the assessment of statistical distributions whilst still permitting planes and lines to be plotted, but with reduced accuracy, and hence is used for presenting discontinuity data in rock engineering. The principles of the equal- area hemispherical projection are shown in Fig. 2.5. This type of projection, like other types of hemispherical projection, is based upon a reference sphere that is free to move in space, but not free to rotate. Thus, all the discontinuities within an exposure can be represented in the same sphere in terms of orientation, independent of their position in space. Each discontinuity will cut the sphere in a similar manner to that shown in Fig. 2.5. In the equal-area projection the trace of the discontinuity on the lower hemisphere can be projected on to a planar surface directly below. The projected trace appears as a great circle. The distance from the centre of the projection to the great circle along the direction of dip is related to the dip of the discontinuity by the expression:

$$AX = 2R\cos\frac{90+\theta}{2} \tag{2.1}$$

where R radius of the reference sphere, and θ dip of discontinuity. When the plane is horizontal (i.e. θ = 0°), AX= 2Rcos45° = R√2. This means that the radius of the resultant projection is larger, by a factor √2, than the radius of the reference hemisphere. The point X is, therefore transferred to point X', a distance OX' from the centre of the plane of projection, by letting OX` = AX/√2. Hence:

$$OX' = R\sqrt{2}\cos\frac{90+\theta}{2} \tag{2.2}$$

Priest (1985) described the process of equal-area projection as like 'peeling the skin' off the lower reference hemisphere, flattening it out and then shrinking it to a circle of radius R.

Discontinuities may be represented as great circles projected from either the lower or upper hemisphere giving rise to lower and upper hemisphere projections respectively. Lower hemisphere projections are most commonly used for assessing discontinuity data. In practice, the great circle representing a discontinuity is plotted directly on to a Lambert or Schmidt net. Examples of these nets are given in Hoek and Bray (1981) and Priest (1985). If 200 measurements of dip and dip direction are made for each locality as recommended, the data are not going to be clearly presented as 200 great circles.

Furthermore, the assessment of statistical distribution is not possible using great circles. Every plane, however, can be represented on the hemispherical projection by a discrete point which is the projection of a line that intersects the plane of the discontinuity at right angles (Fig. 2.5). This line is termed a pole and is unique for each plane. Projections of poles will always be offset from the corresponding great circle by the radius of the reference hemisphere, along the diagonal representing the direction of dip.

An example of a hemispherical projection of poles representing discontinuities intersecting a number of scanlines is shown in Fig. 2.6. The poles shown in Fig. 2.6 have been classified according to persistence using symbols of different sizes. A cluster of large circles would indicate a set of persistent joints in Fig. 2.6. A single persistent discontinuity, however, can be just as important in rock engineering terms as a cluster of less persistent discontinuities. It is therefore useful to indicate persistence of fractures on the hemispherical projection. The manner in which this has been done in Fig. 2.6 does not permit an accurate statistical analysis of the data directly from this plot. For statistical analysis of poles they must be plotted as points.

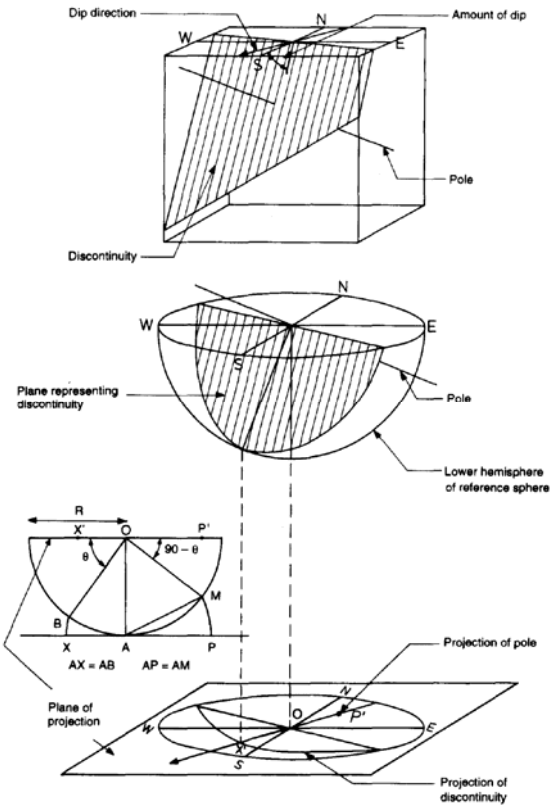


Fig. 2.5 Principles of equal area stereographic projection.

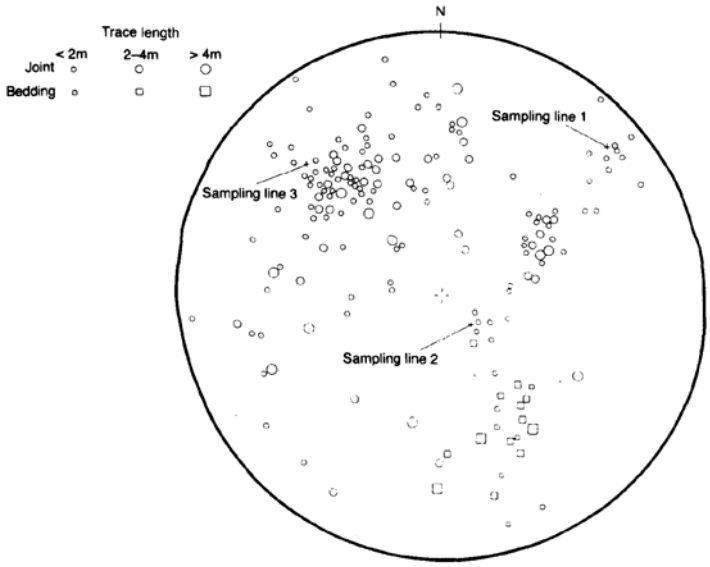


Fig. 2.6 Example of a lower-hemisphere plot of discontinuity normals (poles) classified according to persistence (after Priest 1985).

Statistical analysis of orientation data is carried out to identify sets of parallel or nearly parallel discontinuities. If the rock mass is dominated by a systematic planar fracture pattern then the predominant orientation of each discontinuity set may be identified with relative ease by observing the clusters of poles on the lower hemisphere projection. However in many cases the interpretation of the data is complicated by the following factors:

1. the discontinuities are not planar;

2. the rock mass is cut by randomly orientated discontinuities in addition to those that occur in sets; and
3. the degree of parallelism within a given set may be relatively low.

These factors will result in poles being less clustered. In such cases discontinuity sets, if present in the rock mass. may only be identified from a statistical analysis. This involves placing a sampling window over the data, to generate a matrix of moving average values, representing the variation in the concentration of poles over the projection. The moving average values may be contoured at some appropriate interval to aid interpretation. Figure 2.7 shows the contours of pole concentration determined in this way. Details of various sampling methods are given in Hoek and Bray (1981), Matherson (1983) and Priest (1993). The most commonly used technique is the counting circle (Hoek and Bray 1981; Priest, 1993). This makes use of a circle with a diameter such that the area of the counting circle represents 1% of the plane of projection. The circle is placed on the plane of projection and the number of poles falling within its perimeter are counted and recorded on the plane of projection at the position of the centre of the counting circle as a percentage of the total population of poles. The counting circle is moved over the plane of projection either in a random manner (floating circle) or using a rectangular grid. When the counting circle is close to the edge of the net, any part of the circle that extends beyond the perimeter must re-enter at a diametrically opposite point. Thus the contours of pole concentrations crossing the perimeter of the net should be symmetrical about the diameter. The counting circle method can be most time consuming. A more rapid system of sampling is provided by using a Dimitrijevic Counting Net (Dimitrijevic and Petrovc 1965). The use of this net is described by Matherson (1983). There are a number of computer programs available to perform the statistical analysis. These should be used with caution since it is not always clear what sampling method is being used.

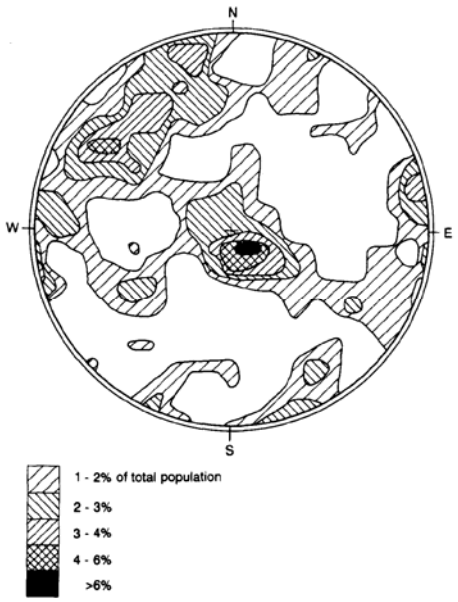


Fig. 2.7 Typical polar concentrations for an exposure of chalk.

Contoured lower hemispherical projections provide a rapid assessment of rock slope stability assuming the shear strength of the discontinuities are purely frictional (Markland 1972; Hoek and Bray 1981). Such assessments do not provide a factor of safety, they simply allow potential failure mechanisms to be identified. Hoek and Brown (1980) and Priest (1985) describe the use of hemispherical projections in the assessment of the stability of underground excavations.

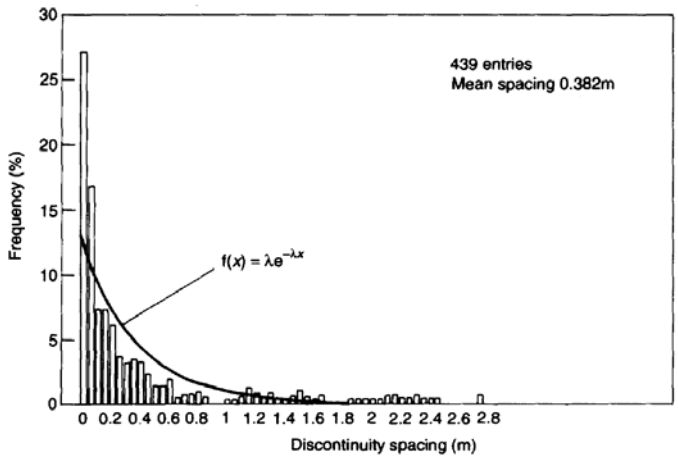
Histograms

Discontinuity spacing data are best presented in the form of histograms. Histograms may be produced

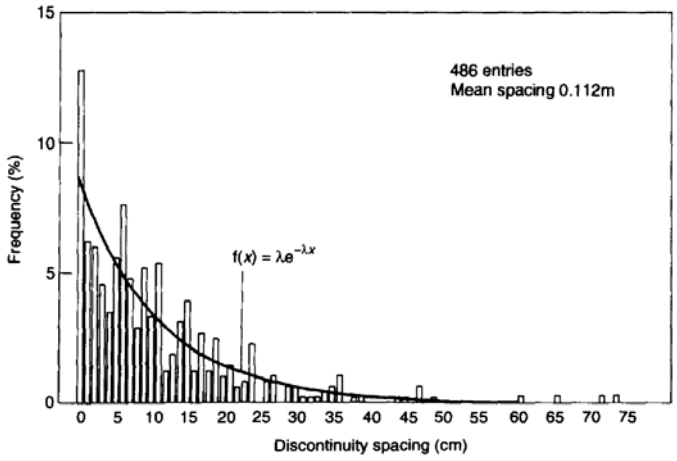
for individual sets of discontinuities or for all discontinuities intersecting a scanline. If the discontinuities in a particular set exhibit a regular spacing they will give rise to a normal distribution and a mean spacing may be easily determined. However, in many cases fractures are clustered or randomly spaced giving rise to a negative exponential distribution (Priest and Hudson 1976). Examples of joint frequency distributions measured from scanlines in sandstone and mudstone are given in Fig. 2.8. the histograms show a close agreement with the negative exponential distribution expressed as:

$$f(x) = \lambda e^{-\lambda x} \tag{2.3}$$

where λ = the mean discontinuity frequency per metre. By fitting a negative exponential distribution to the spacing data the mean spacing may be determined from 1/ λ.



(a) Sandstone 3 : Discontinuity spacings



(b) Mudstone 3 : Discontinuity spacings

Fig. 2.8 Histograms of discontinuity spacings (after Yenn 1992).

Priest and Hudson (1976) established the following relationship between Rock Quality Designation (RQD) and the mean discontinuity frequency per metre (λ):

$$RQD = 100e^{-0.1\lambda}(0.1\lambda + 1) \tag{2.4}$$

where *RQD* is a parameter normally derived from drilicore (Deere 1964) and is commonly used in the

classification of rock masses. The definition of RQD and its application to rock mass classification is discussed later.

DESCRIPTION OF ROCK MASSES

The rock material descriptions, the location of changes in lithology and the description of discontinuities can be brought together to form an overall description of the rock mass in engineering terms. The key elements of a rock mass description are as follows.

1. *Lithology*. This includes the rock types present and any variations in rock material properties within each lithological unit. A rock mass therefore may be divided up into zones on the basis of lithology or changes in intact material properties.
2. *Structure*. This includes large and small scale geological structures such as bedding, folding, faulting and intrusive bodies of igneous origin. A rock mass therefore may be divided up into zones on the basis of structure. It is likely that such a zonation will be similar to that based on lithology for certain structures.
3. *Weathering and alteration*. The processes of weathering or alteration are likely to bring about changes in the mechanical properties of the rock material and the rock mass within any given lithological unit. Weathering is likely to affect rocks near the ground surface, although it should be remembered that in certain cases the depth to which weathering extends may be more than l00m. Alteration may affect rocks at any depth. The rock mass may be divided into zones based on the degree of weathering or alteration.
4. *Discontinuities*. The discontinuities cutting the rock mass may be associated with a number of processes such as deposition, cooling, tectonism and weathering. The pattern of discontinuities commonly varies from place to place within a rock mass as a result the interaction of one or more of these processes and the rock material. The rock mass can therefore be zoned on the basis of discontinuity pattern (orientation, spacing and persistence will be dominant factors contributing to discontinuity patterns). Other attributes of discontinuities such as wall roughness and aperture may also be employed in this exercise.
5. *Engineering application*. Any engineering grade classification is likely to be performed for a particular engineering application. Different applications place emphasis on different attributes of the rock material and the discontinuities.

A rock mass description will involve dividing the rock mass into units of similar expected engineering behaviour. The parameters used in this exercise will be taken from (1), (2), (3) and (4) above and controlled by the engineering application. An example of such a method of rock mass description is given in Fig. 2.9. Thus, features are arranged into groups on the basis of their relationship with the application. Such a grouping forms the basis of engineering grade classification. An example of such a engineering grade classification is given for the chalk at Mundford, Norfolk by Ward *et al*. (1968). The application in this case was the assessment of rock mass compressibility under foundation loading. Ward *et al.* assumed that the compressibility of the rock mass depended primarily upon the following factors:

1. the presence or absence of structure;
2. the spacing of discontinuities;
3. the orientation of discontinuities;
4. the aperture of discontinuities; and
5. the hardness of intact chalk.

The classification of the Mundford chalk was based on these factors. The definition of the divisions used in this classification is shown in Table 2.29.

In areas not affected by intense tectonism the chalk is generally characterized by sub-horizontal and sub-vertical sets of discontinuities. In such cases it will be the subhorizontal discontinuities that will

most influence the compressibility of the rock mass when subject to applied vertical stress changes. Factors (1), (2) and (4) are generally controlled by weathering processes in the chalk. At the Mundford site, a bed of low porosity high strength chalk (a hardground) was present within the rock mass. The intact chalk within this hardground had different mechanical properties to the chalk above and below it, and hence it was considered to be of importance in controlling the mass compressibility when within the zone of influence of the loaded area. Thus lithology played a part in this classification.

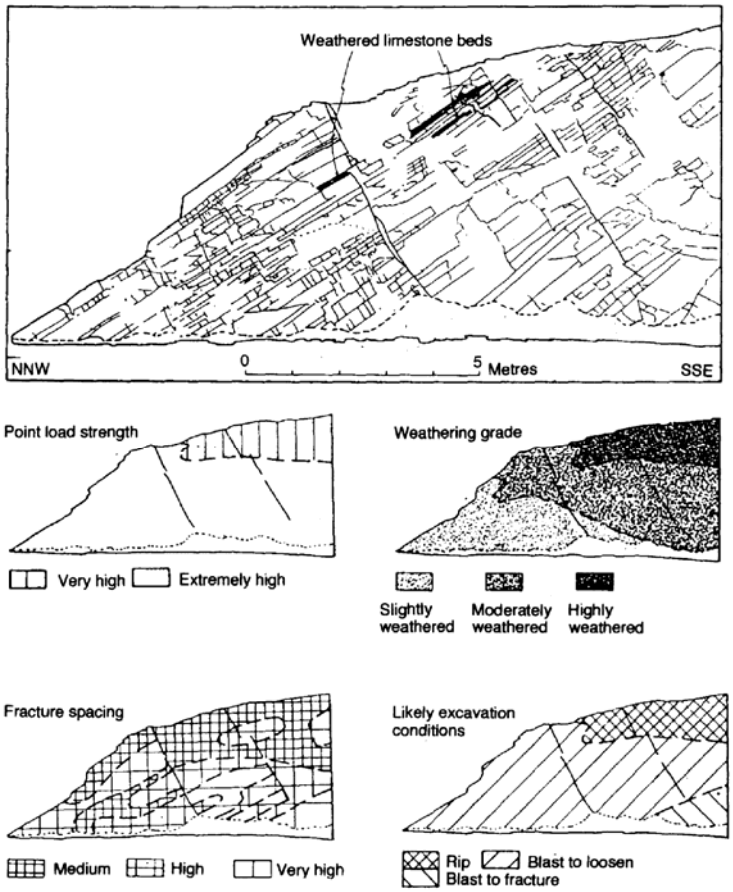


Fig. 2.9 Geotechnical properties and engineering appraisal of a quarry face comprising chert and limestone (after Fookes *et al.* (1971)).

Whilst structure, discontinuity spacing and aperture were defined in a way that could be determined with relative ease in the field, the other parameters were not. The lack of definition of some components may stem from the fact that these, like many engineering grade classifications, are site specific. This classification and its subsequent extensions have been used by geotechnical engineers indiscriminately over much of the chalk outcrop in the UK.

In order to make such a classification more applicable to the whole of the chalk outcrop a greater understanding of the mechanisms which control the mass compressibility of this material is needed. In an attempt to achieve a more generally applicable assessment of chalk mass compressibility based on visual assessment, Matthews (1993) proposed a more simplified classification that reflects our current state of knowledge. This classification is shown in Table 2.30.

Rock mass classifications are commonly used in rock engineering as an aid to design. Most of these classifications such as the geomechanics classification (Rock Mass Rating System (RMR), Bieniawski (1973)) and the Q-System (Barton *et al.* 1974) were developed primarily for underground excavation engineering. Some of these classifications such as the RMR system have been extended for use in rock foundation and slope engineering.

Table 2.29 Visual classification of the chalk (after Matthews *et al.* (1990))

Grade	Original description (Ward *et al*. (1968), I-V, Wakeling (1970), VI)	SPT N	Identification factors			
			Normally used in practice		Not normally considered	
			Structure	Jointing/ particle sizes	Hardness/strength	Weathering
VI	Extremely soft structureless chalk, containing small lumps of intact chalk.	<8	Bedding and jointing absent	Behaviour dominated by chalk fines	Extremely soft	
V	Structureless melange. Unweathered and 8—15 partially weathered angular chalk blocks and fragments set in a matrix of deeply weathered remoulded chalk. Bedding and jointing are absent.	8-15		Behaviour dominated by intact lumps		Deeply weathered
IV	Friable to rubbly chalk. Unweathered or partially weathered chalk with bedding and jointing present. Joints and small fractures closely spaced, ranging from 10mm apart to about 60mm apart. Joints commonly open up to 20mm and infilled with weathered debris and small unweathered chalk fragments.	15-20	Bedding and jointing present	Joints: 10-60mm spacing <20mm aperture with infill debris	Friable to rubbly	Unweathered or partially weathered
III	Rubbly to blocky chalk. Unweathered medium to hard chalk with joints 60mm to 200mm apart. Joints open up to 3 mm, sometimes with secondary staining and fragmentary infillings	20-25		Joints: 60-200mm spacing <3mm aperture possible infill	Rubbly to blocky	Unweathered , sometimes with secondary staining on joints
II	Medium hard chalk with widely spaced closed joints. Joints more than 200 mm apart. When dug out for examination purposes this material does not pull away along the joint faces but fractures irregularly.	25-35		Joints: >200mm spacing 0mm aperture	Medium hard	Unweathered
I	Hard, brittle chalk with widely spaced closed joints. Details as for Grade II but here the chalk is harder.	>35		As II	Hard	Unweathered

Table 2.30 Assessment of mass compressibility of chalk based on visual assessment (after Matthews (1993))

Grade	Description	Compressibility characteristics
A	Structured chalk: discontinuities more than 200mm apart, and closed.	Very low compressibility. E_i=1000-10000 MPa q_y>l000kPa Intact dry density significantly affects compressibility.
B(i)	Structured chalk: discontinuities closer than 200 mm apart, and open. Fracture-block system: tight.	Relatively low compressibility. E_i=500-1500MPa q_y=150-420kPa normally greater than 200kPa E_y≈50MPa Intact dry density is likely to have a limited effect on compressibility.
B(ii)	Structured chalk: discontinuities closer than 200mm apart, and open. Fracture-block system: loose.	Relatively low compressibility. E_i=300-500MPa q_y=150-420kPa, normally greater than 200kPa. E_y≈50MPa Intact dry density has little effect on compressibility.
C	Structureless chalk: a melange of fines and intact chalk lumps, with no regular orientation of bedding or jointing.	Relatively high compressibility. E_i=100-300MPa q_y unknown, but probably less than 200 kPa. Intact dry density has no effect on compressibility. Compressibility behaviour is likely to be effected by the relative proportions of fines and intact lumps of chalk

The most popular rock mass classifications used in rock engineering are the RMR and the Q-Systems. The factors used in these classifications are given in Table 2.31.

Table 2.31 Parameters used in the RMR system and the Q-System of rock mass classification

RMR System	Q-System
Uniaxial strength of intact rock	Rock Quality Designation (RQD)
Rock Quality Designation (RQD)	Number of joints sets
Spacing of discontinuities	Roughness of the most unfavourable discontinuity
Condition of discontinuities (wall roughness, aperture, wall strength, etc.)	Degree of alteration or filling along the weakest joint
Groundwater conditions	Water inflow
Orientation of discontinuities	Stress condition

Importance ratings are allocated to the different value ranges of the parameters, the higher rating indicating better rock mass conditions. The importance ratings are derived from experience and case histories. After the importance ratings for the parameters have been established they are combined to give an overall rating for the rock mass. This rating is most commonly used in assessing the support requirements for underground excavations (RMR and Q) or for estimating the rock mass compressibility for foundation design (RMR). The use of these rock mass classification systems together with others not mentioned here are discussed by Bieniawski (1989).

RECORDS OF BOREHOLES

Obtaining soil and rock samples f good quality requires much care and attention by the driller. As a result, this exercise can be expensive. This is particularly so in the case of rock. It is necessary therefore to justify this expense by maximizing the data gathered by this operation and presenting it in such a manner that it can be readily understood and interpreted. The final borehole logs should be based on the visual examination and description of the samples, the laboratory test results, and the driller's daily report forms. If, as is often the case, a well-trained drilling technician cannot be assigned to each drilling rig, any investigation cannot be better than the drillers responsible for the execution of the work. Even though the quality of the drilling and sampling may be good, if the driller is unable to observe and record adequately the results of his work, the final site investigation will be poor. Good driller's records are therefore often the key to good site investigation.

The driller's record should include the following information:

1. *contractual details*: contractor, client, location, and supervisor's name;
2. *borehole location*: borehole number, ground level at borehole, and inclination and bearing;
3. *drilling equipment*: type of drilling rig used, diameter of borehole, and details of casing. In the case of rotary drilling the type of bit and corebarrel should be given together with details of the type of flush used;
4. *progress*: date of start and finish of borehole, level at the end of each day's boring, and driller's name. In the case of rotary drilling, if the type of bit is changed for any reason the level at which it is changed should be recorded. If a bit needs to be replaced due to wear the level at which the new bit is started should be recorded;
5. *geotechnical data*: soil or rock description, with depth below ground surface, thickness of each soil or rock type, and level (relative to datum) of each change of soil or rock conditions;
6. *groundwater data*: levels at which water is encountered, levels at which water stabilizes in the borehole, rate of inflow, levels at which loss of return occurs, water and casing levels when taking water samples, and depth from which water samples taken;

7. *samples*: level of top and bottom of drive, diameter, type (e.g. open-drive, piston, double tube swivel type corebarrel with plastic liner (e.g. Core Line), etc.), reference number, and length of recovery;
8. in situ *testing*: level at which *in situ* test performed, type of test, and result of test.

In practice this record is usually achieved in two stages. The driller produces a hand-written borehole log at the end of each day's drilling, based on his records made as boring was in progress. This record is given to the engineer who carries out a preliminary engineering record. This record of the borehole is likely to be far from perfect, because at this stage the amount of laboratory testing has not been determined and therefore samples cannot be extracted and split for description, but it is produced to allow testing to be scheduled and additional boring to be varied.

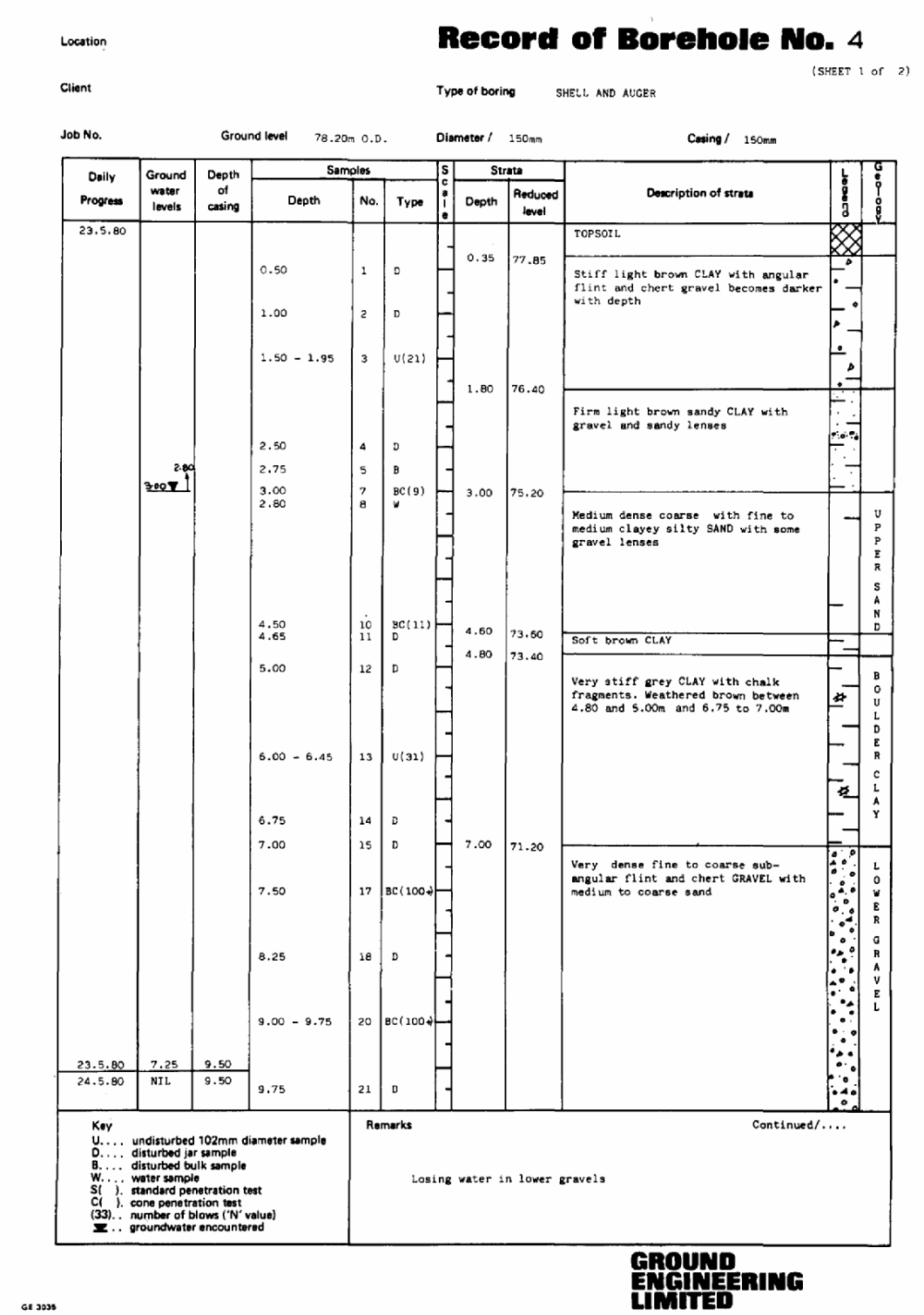
Two factors must combine to produce a good engineering borehole or drillhole record: accurate recording of sampling and soil or rock changes at the time of boring, and good soil and rock description. In order to produce good records of stratum changes the driller or drilling technician must not only be capable of producing a good description of soil and rock, but he must also be aware of the importance of features such as fissuring and fabric in soils and discontinuities in rock. This requires a level of training which is rare at present. In the authors' experience it is unusual for the design engineer to communicate the important features of his proposed structure to the site investigation engineer, and frequently the engineer in charge of the site investigation makes no attempt to discuss his detailed requirements with the drilling foreman.

The final borehole log must be laid out in a simple but informative manner. It must be remembered that if the layout is made over-complicated, important information may be missed by the engineer when attempting to read the log. Examples of borehole and drillhole logs are given in Figs 2.10 and 2.11. In soft ground boring (Fig. 2.10), in addition to the sample descriptions emphasis is given to the location and type of sampling and *in situ* testing. In rotary coring (Fig. 2.11), however, emphasis is placed upon the state of recovery of the core and the fracture state. Guidance on the logging of rock core may be found in the Geological Society of London's Engineering Group Working Party report on the 'Logging of Rock Cores for Engineering Purposes' (Geological Society of London 1970). A revision Working Party set up by the Engineering Group (Geological Society of London 1977) to examine the need for revising the proposals of the 1970 Working Party found that these have in general won acceptance in the UK, although many contractors had made minor modifications to suit individual circumstances. Most of the proposals of the 1970 Working Party are now embodied in the Code of Practice (BS 5930:1981).

It is common practice when logging rock core to take colour photographs of each core run laid out in the corebox. It is important that a colour scale and scale rule be included in the photograph to aid interpretation and comparison with the descriptions given in the log.

STATE OF RECOVERY OF CORE

The state of rock cores recovered is largely a function of the drilling method, and the amount of care employed by the driller during a core run and extraction of core from the corebarrel. Hence these factors must be considered when assessing core recovery and fracture state. The nature and amount of core recovered from good careful drilling can provide a valuable indication of the *in situ* condition and probable engineering behaviour of the rock mass. In any core recovered there will be fractures of natural and artificial origin. It is important that natural fracturing is distinguished from artificial fracturing on the log. Artificially induced fractures should not be ignored since they may assist in the assessment of rock excavation.



Location

Record of Borehole No. 4

(SHEET 1 of 2)

Client

Type of boring SHELL AND AUGER

Job No. Ground level 78.20m O.D. Diameter / 150mm Casing / 150mm

Daily Progress	Ground water levels	Depth of casing	Samples Depth	No.	Type	Scale	Strata Depth	Reduced level	Description of strata	Legend	Geology
23.5.80									TOPSOIL		
							0.35	77.85			
			0.50	1	D				Stiff light brown CLAY with angular flint and chert gravel becomes darker with depth		
			1.00	2	D						
			1.50 - 1.95	3	U(21)						
							1.80	76.40			
									Firm light brown sandy CLAY with gravel and sandy lenses		
			2.50	4	D						
	2.80		2.75	5	B						
	3.00▼		3.00	7	BC(9)		3.00	75.20			
			2.80	8	W				Medium dense coarse with fine to medium clayey silty SAND with some gravel lenses		UPPER SAND
			4.50	10	BC(11)						
			4.65	11	D		4.60	73.60	Soft brown CLAY		
							4.80	73.40			
			5.00	12	D				Very stiff grey CLAY with chalk fragments. Weathered brown between 4.80 and 5.00m and 6.75 to 7.00m		BOULDER CLAY
			6.00 - 6.45	13	U(31)						
			6.75	14	D						
			7.00	15	D		7.00	71.20			
			7.50	17	BC(100)				Very dense fine to coarse sub-angular flint and chert GRAVEL with medium to coarse sand		LOWER GRAVEL
			8.25	18	D						
			9.00 - 9.75	20	BC(100)						
23.5.80	7.25	9.50									
24.5.80	NIL	9.50	9.75	21	D						

Key
U.... undisturbed 102mm diameter sample
D.... disturbed jar sample
B.... disturbed bulk sample
W.... water sample
S( ). standard penetration test
C( ). cone penetration test
(33).. number of blows ('N' value)
▼.. groundwater encountered

Remarks

Losing water in lower gravels

Continued/....

GROUND ENGINEERING LIMITED

GE 3035

Fig. 2.10 Engineering borehole record in soft ground (courtesy of Ground Engineering Ltd).

The core recovered can be divided into five categories:

1. solid core greater than 0.1 m in length;
2. solid core less than 0.lm in length;
3. fragmental material not recovered as core;
4. additional material which may have been lost from the previous core: and
5. reduced length and/or diameter of core due to erosion of soft or friable material.

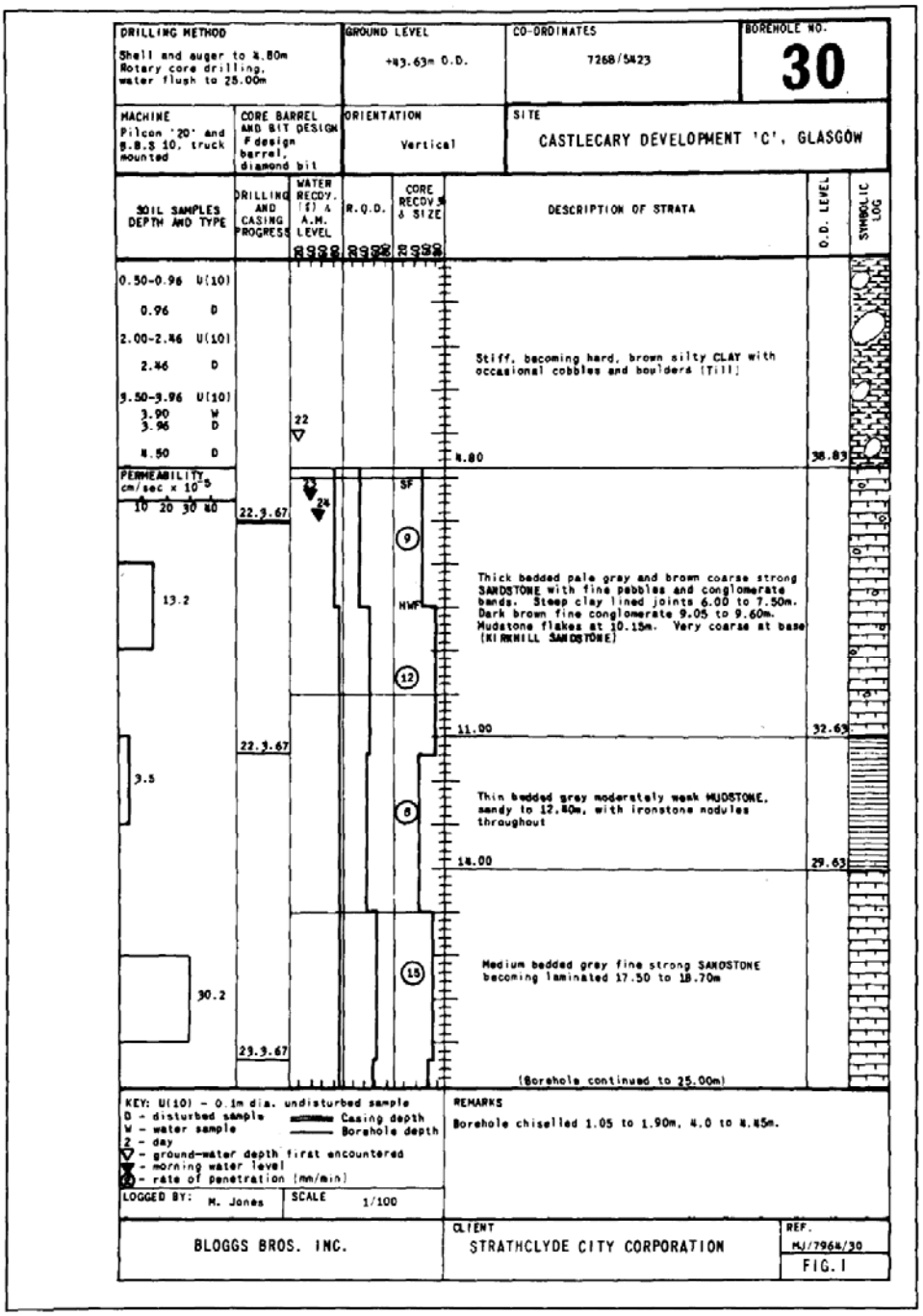


Fig. 2.11 Engineers' drillhole record (Geological Society of London 1970).

The quality of rock recovered may be classified in terms of total or solid core recovery or in terms of a quality index such as rock quality designation (RQD), fracture index or stability index, provided only natural fractures are considered. The definitions of these terms are given in Table 2.32. The determination of the more commonly used parameters are shown schematically in Fig. 2.12. Solid core recovery (SCR), RQD, fracture index and stability index may be used as criteria for a quantitative description of the fracture state of the cores. The simplest of these is solid core recovery and is always shown along with total core recovery in a graphical form in the borehole log. The stability index is the most complicated method of assessing rock quality and hence is rarely used in practice. Core recovery (total and/or solid), RQD and fracture index are normally shown in the borehole log in a graphical form with some indication of changes in corebarrel size (Fig. 2.11). The fracture state of the core recovered may be assessed using these parameters together with the fracture log discussed earlier.

Table 2.32 Methods of classifying the quality of rock cores

Classification	Definition	Category of core considered	Remarks
Total core recovery	Percentage of the rock recovered during a single coring 'run'	(1), (2), (3), (5), i.e. all the core placed in the core box	Gives indication of material that has been washed into suspension or the presence of natural voids
Solid core recovery	Percentage of full diameter core recovered during a single coring 'run'	(1) and (2)	Gives indication of fracture state
Rock quality designation (RQD) (Deere 1964)	Percentage of constant diameter solid core greater than 0.lm in length recovered during a single coring 'run'	(1)	Can give indication of fracture state but does not take changes in core diameter into account. The diameter of the core should preferably not be less than 55mm (NWX or NWM size)
Fracture index	Number of fractures per metre. This is generally calculated for each core run	(1) and (2)	Can give indication of fracture spacing
Stability index (Ege 1968)	Index no. = 0.1 x core loss (length drilled-total recovery) x 10^{-2}+no. of fractures per 0.3m (1 ft) + 0.1 x broken core (core <7.5 cm in length) + weathering (graded 1-4 from fresh to completely weathered) + hardness (graded 1-4 from very hard to incompetent)	(1), (2), (3), (5)	Can give indication of fracture state but does not take changes in core diameter into account

Since it was first introduced by Deere and his co-workers in 1967, RQD has become increasingly used. Indeed it has become an integral part of the rock mass classification systems used in underground excavation engineering and in rock foundations (Bieniawski 1989). For 50mm diameter cores, where a height to diameter ratio of 2:1 was used for strength and stiffness tests, RQD provides a useful indication of the number of times such a test result could occur in each core run, assuming the lithology was consistent. As such, the percentage of core over 100mm in length had a real value, especially in assessing the bearing capacity of a rock mass. However both the British Standard (BS 5930:1981) and the ISRM (ISRM 1981) recommend the RQD should be based on an axial measurement along the centre of the core rather than the solid core length. When there are discontinuities present that are inclined such that they make a relatively small angle with the core axis (for example, steeply dipping joints intersecting a vertically orientated core), the RQD can have a higher value than the SCR and have no relationship to the performance of that rock mass to any tested sample (Fig. 2.13).

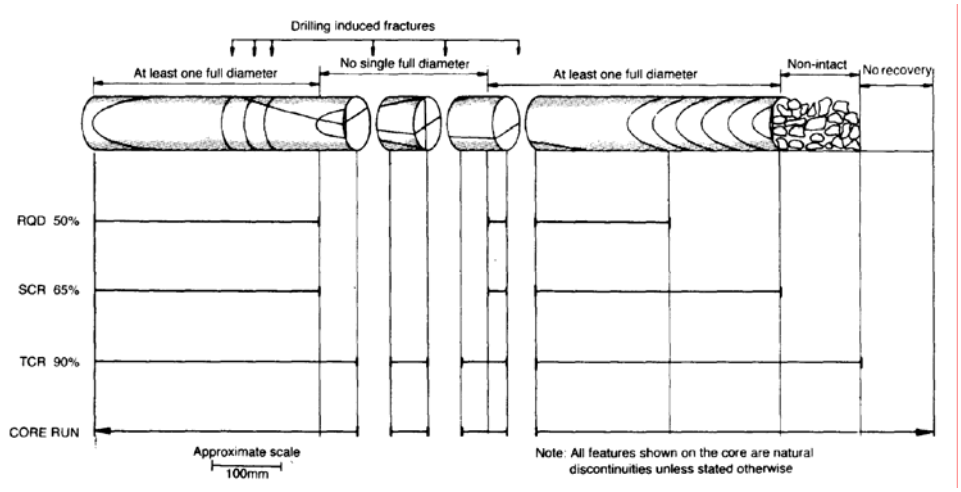


Fig. 2.12 Schematic illustration of fracture logging terms (after Norbury *et al.* (1986)).

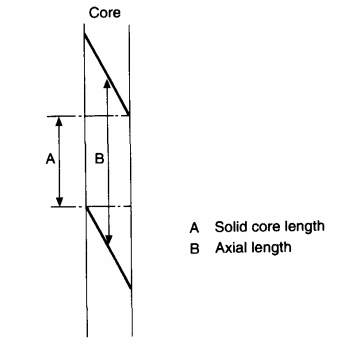


Fig. 2.13 Measured lengths related to an inclined discontinuities.

Hawkins (1986) suggests that a new rock quality designation value be introduced based on minimum core lengths of 300mm instead of 100 mm. This new value would be referred to as RQD_{300}. The reasoning behind this proposal is that 300 mm approximates to the maximum thickness that can be ripped in certain rock types. A more sound argument for changing the base length for RQD measurements relates to the comments made above concerning the value of RQD in relation to the number of test specimens that can be obtained from a single core run. Since it is now common practice to use P and S size corebarrels, particularly in weak rocks, solid core lengths of greater than 230 mm or 281mm respectively are required to meet the 2.5:1 length to diameter ratio recommended by ISRM for strength and stiffness tests (ISRM 1981). The disadvantage of using a different base length however is that RQD_{100} is required for the conventional rock mass classification systems such as the RMR or the Q-System (Bieniawski 1989).

The measurement of RQD does not take into account any changes in lithology within the core run. Changes in lithology are often associated with a change in the fracture pattern owing to the different mechanical properties of each rock type. There is a natural tendency for engineers to assume that a high RQD value comes from a stronger rock. In a sequence of interbedded limestones and mudstones, for example, the limestone units may dominate in contributing to a relatively high RQD value, thus masking the fracture state of the less competent mudstone units. Such an example is shown in Fig. 2.14. In the example shown, the mudstone units contribute nothing to the RQD value of 55% for each run since they are characterized by a horizontal fracture spacing of 90 mm. Hawkins (1986) suggests that this type of problem may be avoided if RQD values are based on the thickness of lithological units rather than core run length. The lithological quality designation (LQD) could be shown on core logs alongside the conventional RQD values. Fig. 2.14 shows the relationship between LQD and RQD for two adjacent core runs. Clearly LQD is of particular value when dealing with interbedded rocks of contrasting mechanical properties. In cases where the thickness of each rock unit is equal to or greater than the core run length the value of the LQD is diminished since the conventional RQD measurements are likely to reflect the changes in fracture state associated with lithology.

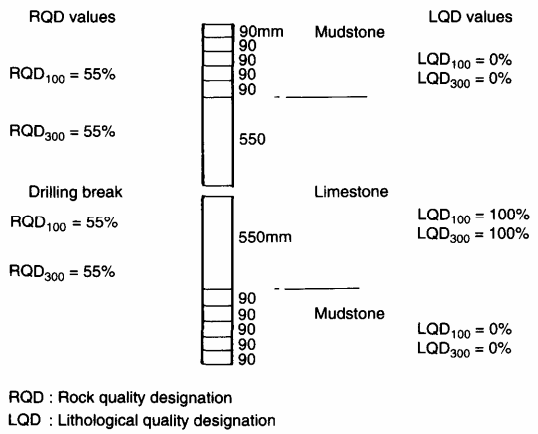


Fig. 2.14 Relationship of RQD and LQD on two adjacent core runs (after Hawkins 1986).

RECORDS OF TRIAL PITS AND SHAFTS

Trial pits are the cheapest method of obtaining samples and a continuous record of ground conditions in soils and soft rocks for shallow depths. Shafts may be used to investigate greater depths in cohesive soils and rocks, but generally provide less discontinuity data since it is more difficult to clean the debris caused by digging from the circular face of a shaft than it is from the flat faces of a trial pit. Furthermore, shafts are often more expensive to dig than trial pits because of the specialized plant involved. Trial pits are ideally suited for investigating superficial deposits which often exhibit a high degree of variability both laterally and vertically. They are also particularly valuable in examining fill material for voids, loosely compacted layers or deleterious material and for investigating slope failures. Trial pits allow hand cut samples of soil to be taken, thus minimizing the effects of mechanical disturbance. Where pits are dug in landslips it is often possible to take undisturbed samples of the major slip planes.

When recording information from trial pits, particular attention should be paid to the orientation of the faces of the pit, the orientation of undisturbed samples taken from it, and the orientation of discontinuities (fissures, joints, cleavage, bedding planes, slip planes, and faults) encountered within it. The logging of trial pits or shafts involves making a full engineering description of all the materials and discontinuities found in the faces and floor of the pit in the manner outlined earlier. Trial pits are generally logged by describing the materials encountered along a vertical line in one or more faces of the pit. If the soil or rock is highly variable, all the faces should be described in detail and a scale drawing made of each face. In some cases, it may be necessary to describe the materials in the floor of the pit. Discontinuities where present should be described in all the faces of the pit. In situations where more than one face is examined in detail, a plan of the pit should be drawn and the respective faces labelled and orientations noted on the plan in order to identify the various pit face logs. It is often useful to take photographs of each face of the pit for future reference. The Geological Society of London's Working Party Report on the Preparation of Maps and Plans in Terms of Engineering Geology (1972) gives some guidance on the presentation of trial pit logs.

In order to log the faces of a trial pit or to take samples it will be necessary for personnel to work at the bottom of the pit. This can be extremely hazardous since the sides of the pit may collapse with little or no warning. In some cases, pits are dug with one face battered back in order to allow rapid escape. Support should be provided for the sides of pits which are deeper than 1.2 m; methods of timbering are discussed by Tomlinson (1980).

Chapter 3

The desk study and walk-over survey

INTRODUCTION

The desk study and walk-over survey are the two essential components of ground investigation. Other parts (for example, boring, drilling and testing) may sometimes be omitted, but these parts of the site investigation process must always be carried out.

The desk study should be carried out at the start of site investigation. Its purpose is to provide as much information on the probable ground conditions, and the likely problems that they will produce for the proposed type of construction, as is available without commissioning new ground investigation work (see Chapter 1). This information is also necessary for the design of ground investigation work.

The walk-over survey is carried out after the desk study has been substantially completed, and once preliminary plans have been made for any ground investigation site work, in order to glean extra information on the geology and on likely construction problems, and to assess access for investigation plant and equipment.

Both the desk study and walk-over survey provide large quantities of invaluable information at negligible cost. They are by far the most cost-effective parts of the site investigation process. They should be used not only to look at the site (which will often be in the ownership of the client), but also at its surrounds (which perhaps cannot be the subject of direct methods of ground investigation). Once these surveys are complete, the results should be formally presented in a report which brings together the details of:

- site topography;
- geology;
- geotechnical problems and parameters;
- groundwater conditions;
- existing construction and services;
- previous land use;
- expected construction risks; and
- proposed ground investigation methods.

A lack of knowledge of existing construction around or below a proposed development site can be disastrous, as the following example shows. In September 1991, piles were driven into the bed of the Chicago River to provide protection from riverborne traffic to the Kinzie Bridge (Fig. 3.1). It had previously been appreciated that the bridge pier lay close above a tunnel, because a contractual requirement was that existing piles were to be extracted, and the new ones were to be installed down the same pathway. This requirement was later relaxed, presumably because the proximity of the tunnel was by now forgotten. Unfortunately the piling fractured the wall of an underlying tunnel, part of the city's 61-mile long tunnel system. This system had been built at the turn of the century to carry heating and construction materials into the city centre. In April 1992 the crack opened, allowing 250 million gallons of water to flood the basements of downtown Chicago (the USA's third largest city), shutting down the power and bringing the city's business district to a halt for several days. The President of the United States signed a disaster declaration, making the city eligible for Federal disaster aid.



Fig. 3.1 The Tampa Trbune, Thursday April 16, 1992
(sources: City of Chicago Mayor's office, Chicago Sun Times, Metropolitan Water Reclamation District of Greater Chicago, Associated press graphic)

The role of the desk study is therefore much wider than simply the determination of likely soil and rock conditions, although this is undoubtedly one of its most important functions. Not only should it aim to determine the position of adjacent services and structures, but it should also search for potential hazards to construction workers, for example from contaminated land.

Table 3.1 Types of information useful for desk studies

Aspect of investigation	**Type of information**
Site topography	Topographic maps Stereo air photographs
Geology	Geological maps Geological publications Regional guides Sheet memoirs Learned journals Air photographs Soil survey maps and records
Geotechnical problems and parameters	Geotechnical journals Engineering geology journals Civil engineering journals Newspapers Previous ground investigation reports
Groundwater conditions	Topographical maps Air photographs Well records Previous ground investigation reports
Meteorological conditions	Meteorological records
Existing construction and services	Construction (as-built) drawings Topographical maps Plans held by utilities Mining records Construction press
Previous land use	Out-of-print topographical maps Out-of-print geological maps Air photographs Airborne remote sensing Archaeological society records Mining records

SOURCES OF INFORMATION FOR DESK STUDIES

Available records come in many different forms. Some are readily available, whilst others are difficult to obtain. Examples are given in Table 3.1.

In the UK, geological and topographical maps, and air photographs are readily available from many sources. At the time of writing the main sources of information are given in Table 3.2. The main agent for Ordnance Survey Publications (topographical and geological maps) is currently: The London Map Centre, Cook, Hammond and Kell Ltd., 22-24 Caxton Street, Westminster, London SW1H OQU (tel: 0171-222-2466 fax: 0171-222-2619).

Table 3.2 Sources of maps and photographs

Type of information	Source
Current Ordnance Survey maps	Ordnance Survey distributors, Local authority engineer's department, Local library
Old Ordnance Survey maps	County archives or local muniment rooms, British Library (Reference Division) in: London, Aberystwyth, Edinburgh
Air photographs	Local authorities, Specialist air photography companies, Royal Commission on Ancient Monuments
Published geological maps	Ordnance Survey distributors, The Geological Museum, London
Handbooks on regional geology	Her Majesty's Stationery Office
Published sheet memoirs	Her Majesty's Stationery Office
Manuscript maps and out-of-print sheet memoirs	British Geological Survey Library, at the Geological Museum, London
General geological records	Refer to Geological Directory of the British Isles - A Guide to Reference Sources (Geological Society of London 1978)
Borehole records	Local authorities British Geological Survey
UK technical journals	Geotechnique, Quarterly Journal of Engineering Geology, Ground Engineering, Proceedings of the Institution of Civil Engineers, Geotechnical Engineering

Desk studies are carried out using existing air photographs, many of which are routinely taken (for example by the counties), for detailed topographical surveys. To obtain details of air photographs, a map showing the site, together with any preferred dates and scales of photography, should be sent to potential suppliers. The major sources of information on existing air photography in the UK are:

Air photography from 1971 onwards:

Air Photo Sales, The Ordnance Survey, Romsey Road, Maybush, Southampton SO16 4GU (tel: 01703-792584).

Old air photography:

Details of sources (including commercial air-photo organizations) can be obtained from: Air Photo Advisory Service, The Ordnance Survey (at the above address).

Air photographs for England are held by:

National Monuments Records Centre, Kemble Drive, Swindon SN2 2GZ (tel: 01793-414600/414700).

Air photographs for Scotland are held by:

Scottish Office, Air Photographs Unit, Room 121, New St. Andrew's House, St. James' Centre, Edinburgh EH1 3SZ (tel: 0131-244-4258 fax: 0131-244-4785).

Air photographs for Wales are held by:

Air Photograph Unit, Welsh Office, Cathay's Park, Cardiff CF1 3NQ (tel: 01222-823819 fax: 01222-825466).

Topographical maps

Topographical maps have been published in the UK for about 130 years. The most common scale available over recent years was the 1 in. to 1 mile (1 :63 360) map, but this has now been replaced by the 1:50 000 Second Series. Plate 3.1 shows examples of the same area as depicted at the two different scales, and shows that the 1 :50 000 plan is basically a photographic enlargement of the earlier map. These maps contain too little detail for many site investigations, where attention is devoted to a relatively small area of development, but they are very useful on extended sites such as highways, where existing road and footpath access is often complex.

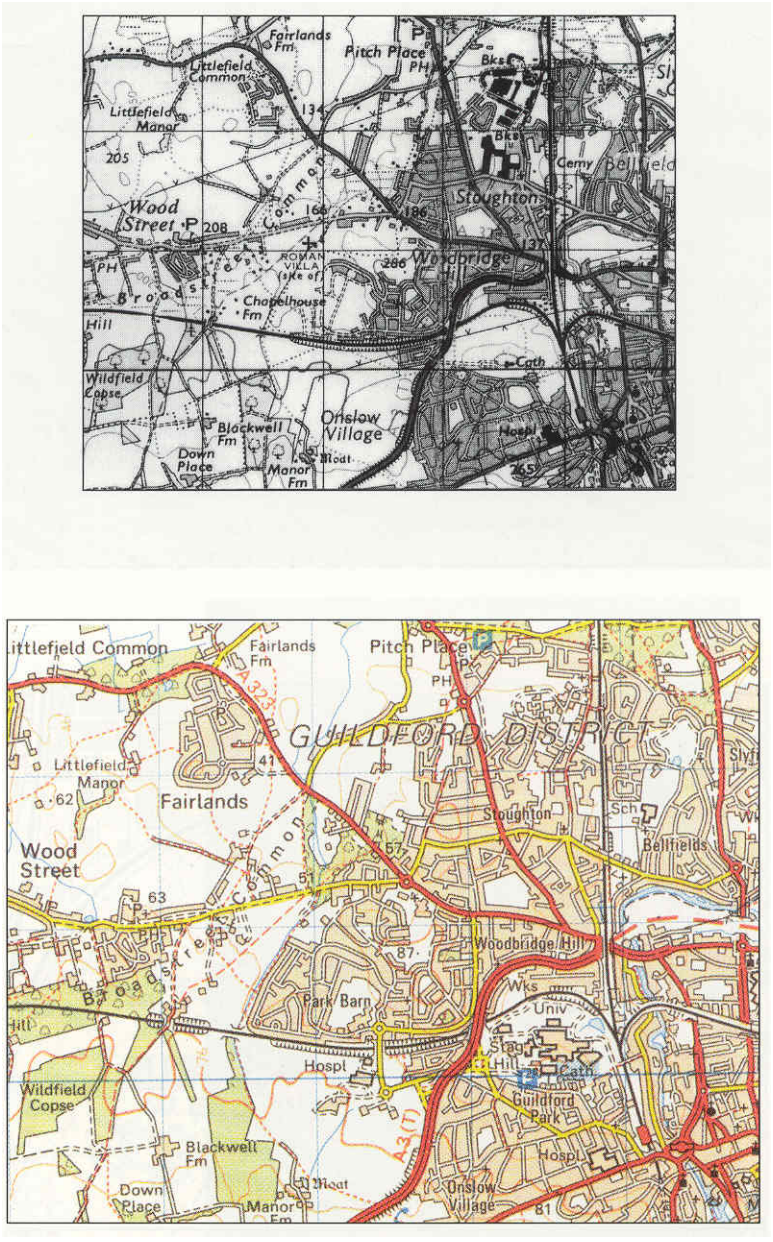


Plate 3.1 Two issues of small scale topographical maps: above 1:63 360 (1” to 1 mile); below 1:50 000. (© Crown copyright reserved)

The 1:25 000 map (approximately 2 inches to 1 mile) combines the advantages of the use of colour in the 1:50000 Second Series with a larger scale, and in common with the 1:50000 series is commonly available in good bookshops throughout the UK. The use of colour at this

scale allows the sites of springs, streams and rivers to be easily detected and in addition regular parallel patterns of artificial drains are often marked, giving advanced warning of a high water table (Plate 3.2).

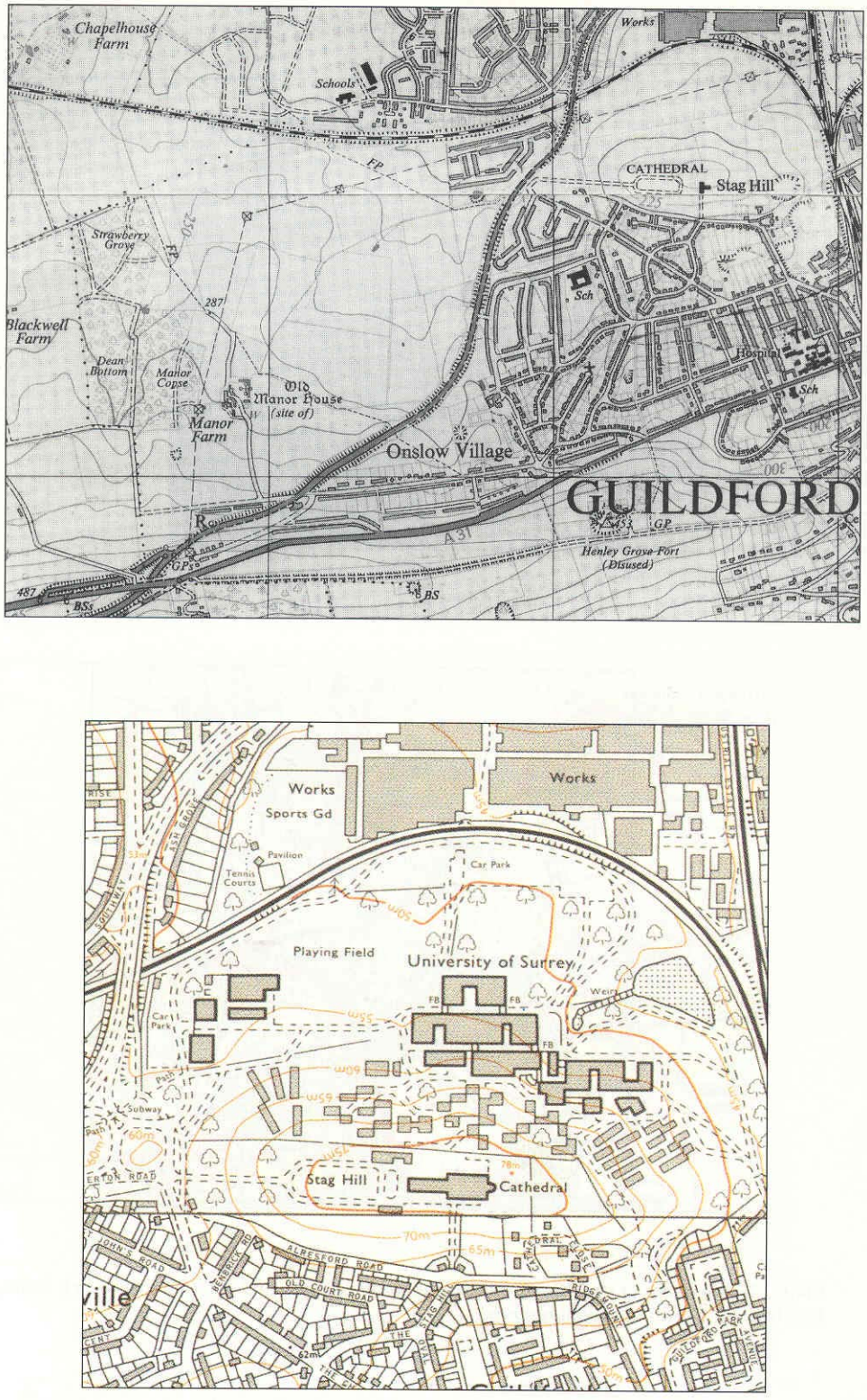


Plate 3.2 1:25 000 and 1: 10 000 maps of the Guildford area (© Crown copyright reserved)

The 1 :25 000 map is the largest scale with close contouring, in this case at a 25 ft (7.6 m) interval. Steep slopes which may suffer from, or be liable to, instability can be marked out for further investigation.

Large-scale maps are available from the main Ordnance Survey distributor in London at scales of 1:10 000, 1:2500 and 1:1250. The 1:10 000 map replaces the old 6in. to 1 mile (1:10 560) map, while the 1:2500 map is at a scale of approximately 25in. to 1 mile. 1:10 000 scale maps are contoured at 5 or 10m intervals, or at 7 and 8m intervals where contours are derived from a 25ft interval. 1:10 560 maps are usually contoured, typically at 100 ft (30.48 m) intervals, which is too coarse for most purposes. 1:2500 and 1:1250 scale maps are uncontoured (Harley 1975). Whilst the 1:10560 and 1:2500 maps are available for most of the country, the 1:1250 is only available for urban areas. For site investigation purposes, the 1:2500 scale map (or, site investigation when the ground levels at boreholes must be determined.

Harley and Phillips (1964) have provided a useful guide to the early editions of the Ordnance Survey. Maps of 1 in. to 1 mile were issued from 1805, and by 1840 covered most of the south of the UK. In 1840, the 6in. to 1 mile (1:10 560) survey was started, and the first revision of this series and the 25 in. to 1 mile maps (1:2500) was carried out between 1891 and 1914. Where County Archives, libraries and engineers departments do not possess copies of the relevant maps, full sets can be found in the British Library Reference Division at the British Museum, London, in the National Library of Wales, Aberystwyth and the National Library of Scotland, Edinburgh.

Case Study - Use of old topographical maps to detect made ground

Figure 3.2 shows extracts from the 25 in. to 1 mile (1:2500) maps of the Biddulph Moor area in Staffordshire. Maps have been found from five different dates: 1876, 1899, 1925, 1960 and 1968; it is possible that further editions exist. The site remains fairly undeveloped until the 1960 map. By 1968, development is taking place in the centre of the area shown. Considerable structural damage occurred to these houses. Investigations revealed that they had been built on made ground and were subsiding. The maps should have given warning of this. The 1899 and 1925 maps show a broken line in the centre of the area; this marks the position of a stream which once ran south through the area, under the road, to emerge from a culvert and continue its course southwards. No such features exist on the 1960 and 1968 maps because the site had, by then, been infilled and levelled, but the stream continues to emerge on the south side of the road.
The 1876 map does not show the stream. This indicates how important it is to collect all archive material (and especially different editions of maps and air photographs) if there is to be a high probability of detecting problems.

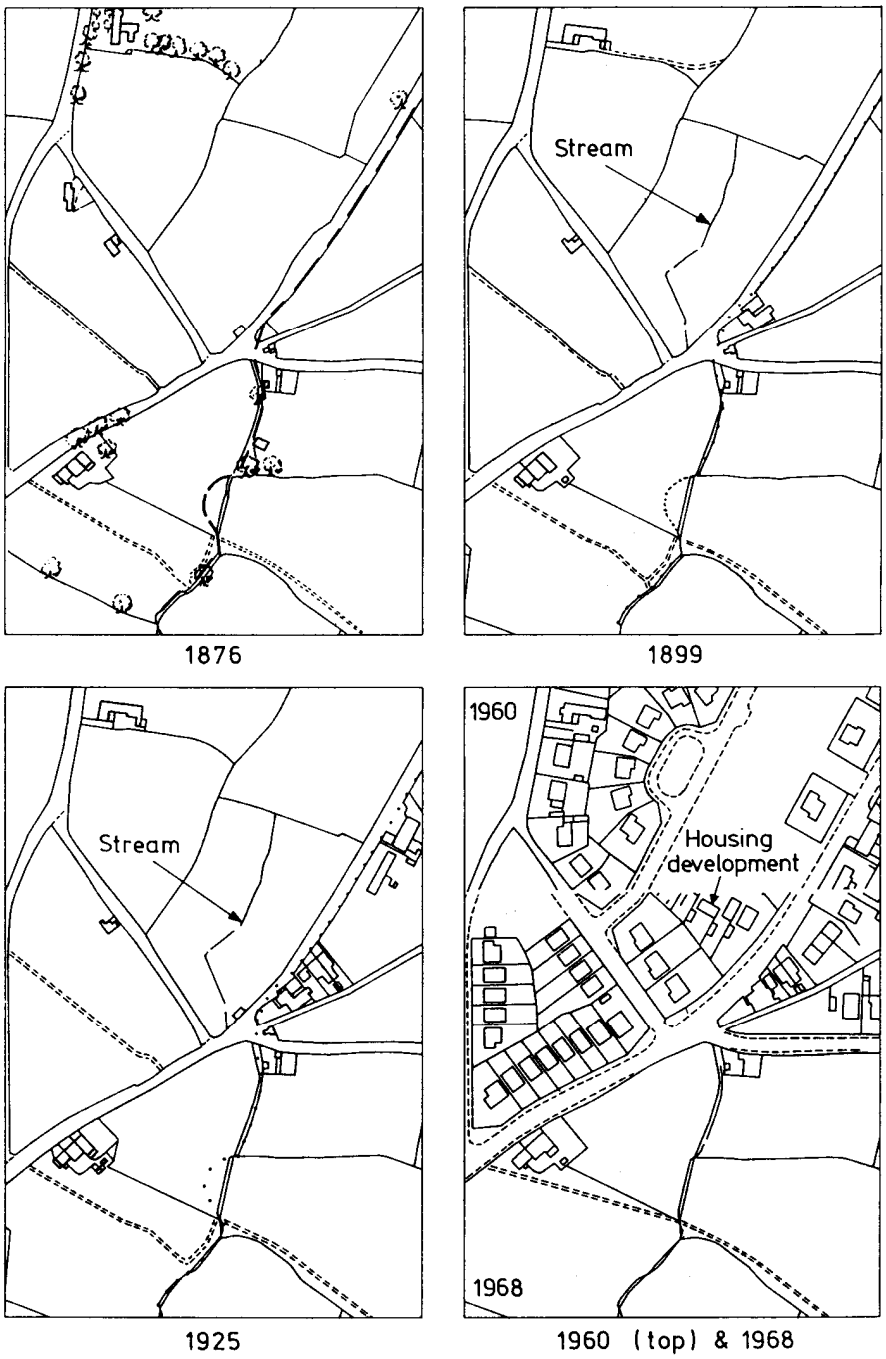


Figure 3.2 Extracts from 25 in. to 1 mile (1:2500) maps of the Biddulph Moor area, Staffordshire. (Ordnance Survey materials was used in this map. © Crown Copyright reserved.) Courtesy of J.H.R. Haswell and Partners, London.)

Geological records

The first reaction of an experienced site investigation engineer to a new problem or site will almost certainly be to look at a geological map of the area. With experience, a great deal of valuable information can be obtained from a knowledge of the location and stratigraphy of the site.

Many types of geotechnical problems are similar over large parts of the same type of deposit. For example, the droughts occurring in the UK during the summers of 1947 and 1976 led to frequent observations of structural distress in houses founded on fatty clays such as the London clay and Gault clay. It has been observed by Ward (1953) that under open grassland significant soil movements will occur down to below depths of 1 m, while where trees exist desiccation by roots may penetrate to 4-5 m below ground level. Clearly, when small structures are to be placed on such soils, their foundations will almost certainly need to go to greater depths than are dictated solely by the strength of the soil.

This type of problem is not the only example of its kind; London clay frequently contains excessive quantities of soluble sulphates, requiring the use of sulphate resisting cement, and as a further example chalk and limestone outcrops frequently contain infilled dissolution features which may become unstable and collapse if built upon.

Another group of problems that may be detected from the geological map relates to the combination of geological and topographical features. Cambering, valley bulging, gulls and dip/fault movements are often associated with the sides of valleys where hard rock overlies clay. Gulls take the form of crevices, often running parallel to the valley bottom, which are typically infilled with loose or soft material. Site investigation by drilling will only rarely reveal the existence of these features and there is therefore the danger that a structure, supposedly founded on top of the rock, would undergo excessive differential settlement. Similar problems can occur when structures are placed on or near to partially infilled dissolution features, which may be reactivated by the change of surface drainage patterns as a result of construction. Whilst swallow holes can be found on most outcrops of water soluble rocks, they are particularly frequent where thin layers of impervious material overly them, such as in the Horndean area of Hampshire, and at Mimms in Hertfordshire. In both of these areas, the relatively impervious Eocene beds are very thin and close to the edge of their outcrop, and overlie chalk.

Finally, with experience it is possible to judge the amount of investigation required, partly on the basis of the stratigraphy of the site. All deposits vary, both in thickness and in geotechnical properties, and the degree of investigation should be related to the expected uniformity of ground conditions. For example, London clay has well documented properties and tends to be fairly uniform and of great thickness. In contrast, where a stratum is of limited vertical extent (for example, Cornbrash rock which has a maximum thickness of about 10 m) small variations of thickness are much more significant. CP 2004 (Foundations) gives presumed bearing pressures of up to 4000kN/m2 for this type of rock but if such foundation pressures are to be applied, then a detailed investigation of the thickness of the rock will be required to ensure that the Oxford clay beneath will not be overstressed. Similarly, all alluvium tends to have internally more variable lithology, and to be less compact than other sedimentary deposits.

Geological maps are published by the British Geological Survey and are available at the Geological Museum in London, or from the official Ordnance Survey distributor. The availability of maps is given in *Government Publications Sectional List No. 45* (HMSO) which can be obtained free of charge. Published and unpublished maps that are available fall into the following categories:-

1. 1 in. to 1 mile and 1 :50 000. As with the Ordnance Survey topographical maps, the 1in. to 1 mile (1:63 360) map is being replaced by the 1:50 000. In the case of geological maps, the majority of cover now existing remains at a scale of 1 in. to 1 mile. These maps are available in 'solid', 'drift', or 'solid and drift' versions. 'Drift' is a 'dustbin' term used by geologists for all Pleistocene and recent deposits, such as alluvium, glacial material (till) and peat, while 'solid' refers to the sedimentary, metamorphic or igneous rocks which lie beneath any such material. 'Solid' maps do

not show so clearly the extent of any drift that may exist at the ground surface, which is a serious disadvantage in site investigation. 'Solid and drift' maps give the fullest information since they not only give the boundaries of drift materials, but indicate the positions of boundaries between solid strata, where these occur below drift deposits. Whilst it is not usual to find more than one type of map for each location, where there is a choice 'solid and drift' is to be preferred.

2. 1:25 000 (2 in. to 1 mile) and 1:21 120 (3 in. to 1 mile). A few maps of this scale are available, some of which give engineering geology and geotechnical details. At present the cover is limited to Milton Keynes New Town (1:25000). Belfast (1:21 120) and Peterborough (1:25 000).
3. 6 in. to .1 mile (1:10560). Only limited maps of this series are available for purchase, for areas of Northern Ireland, London and for coalfield areas. The great value of this series, however, lies in the manuscript copies held in the Geological Museum Library and at the British Geological Survey's office, and known as the 'County Series'. These maps can be referred to and photo-copies may be purchased, and provide an excellent starting point for geological investigations.

In addition to geological maps, in the UK the British Geological Survey publishes various written sources of information, and will also give access to unpublished information. The most important published works are the 'regional guides' and the 'sheet memoirs'.

Seventeen regional guides are published (Fig. 3.3), covering England, Wales and Scotland. For the non-geologist, these Handbooks on the Regional Geology of Great Britain provide a simple guide to a large section of the country and are therefore a good starting point for the fact-finding survey. More specific and detailed information, including lists of exposures, can be obtained for a particular 1 in. to one mile geological map in the form of the sheet memoir. Sheet memoirs contain detailed information on the local nature of each of the strata, descriptions of the exposures in the area, borehole and well records and groundwater supply details. In addition, slope stability and mineral resources are sometimes discussed. Only part of the available 1 in. maps are covered by sheet memoirs available for purchase, but those which are at present out of print can be referred to at the British Geological Survey Library in Exhibition Road, London, or at local BGS offices.

'Economic and coalfield memoirs' are similar to sheet memoirs, but cover a wider area of specific interest. Two examples are: The Mesozoic Ironstones of England- The Liassic Ironstone and Geology of the S. Wales Coalfield, Part 1, Country around Newport, both of which are currently in print.

The BGS offices also hold various types of unpublished information which may be particularly useful during a desk study. These include out of print sheet memoirs, records made during mapping for the 6in. County Series (known as 'field slips') and the Field Unit Borehole Collection. The Field Unit Borehole Collection may be especially useful as it contains previous site investigation records. Well catalogues can also give valuable information on the depths of different soil types. Well catalogues are now being augmented by the new series of well inventories.



Fig. 3.3 Index map to the areas described in the *Handbooks on the Regional Geology of Great Britain.*

Case Study – Use of well records

For example, well records for two wells made at the British Sugar Corporation's Factory at Sproughton near Ipswich (Table 3.3) show one of the problems of boring in the Gipping Valley.

Despite the fact that these two wells are only approximately 200 m apart, the surface of the Upper Chalk descends from 15.2 m below ground level to over 58.2 m below ground level. This infilled glacial valley often contains materials which are loose and highly compressible. Because the floors of buried channels are irregular, and their sides are very steep, it is rather difficult to detect their presence without closely spaced borings.

Table 3.3 British Sugar Corporation, Sproughton - well records

Well no.	National Grid ref.	Surface Level ft. (m)	Strata	Depth to base ft. (m)
207/139(a)	TM136448	+20 (6.1)	Drift (buried channel) Sand and Gravel (buried channel) End of well	22 (6.7) 191 (58.2)
207/139(b)	TM135450	+20 (6.1)	Drift Sand and Gravel Upper Chalk End of well	34 ½ (10.5) 50 (15.2) 250 (76.2)

Case study - Use of geological maps to plan foundations

Plate 3.3 shows a section of the 1:25000 geological map (solid and drift) for Milton Keynes Sheet SP83 and Parts of SP73, 74, 84, 93, and 94). The area within the box was the site proposed for a low-rise school complex. The geological map shows a fault ('Willen Fault') running through the site, which the cross-section given on the map indicates downthrows to the south. Ground conditions in the area consist of Head, Oxford clay, Kellaways Beds, and Cornbrash. The nature of the Head cannot be deduced from the map, except that it is a drift deposit and may, therefore, be loose or soft. The Oxford clay is described on the generalized vertical section at the edge of the map as 'Bluish grey mudstones about 220ft (67m)' thick, but appears as a stiff or very stiff clay near the surface. The Kellaways Beds are described as 'Fine grey sands overlying bluish grey clay 16-17ft (4.9-5.2m)' thick. The Cornbrash is a limestone rock 3.5-7ft (1.0-2.1m) thick.

Extensive trial pitting and borehole excavation carried out on this site showed that the Head was often soft and loose and therefore unsuitable as a founding stratum. It extended further to the west than shown on the geological map. The trial pits confirmed that, under the proposed building area, the Cornbrash was at a suitably shallow and uniform depth to provide a foundation for spread footings, with the exception of a small area in the south-west of the site. In this area the Cornbrash was found to be 10 m below ground level, and it was deduced that this was as a result of the fault. In this area the foundations were piled.

It will be apparent that although geological maps will not always be precise they can give excellent guidance on the likely ground conditions, their disposition and the approximate thicknesses of each soil or rock type.

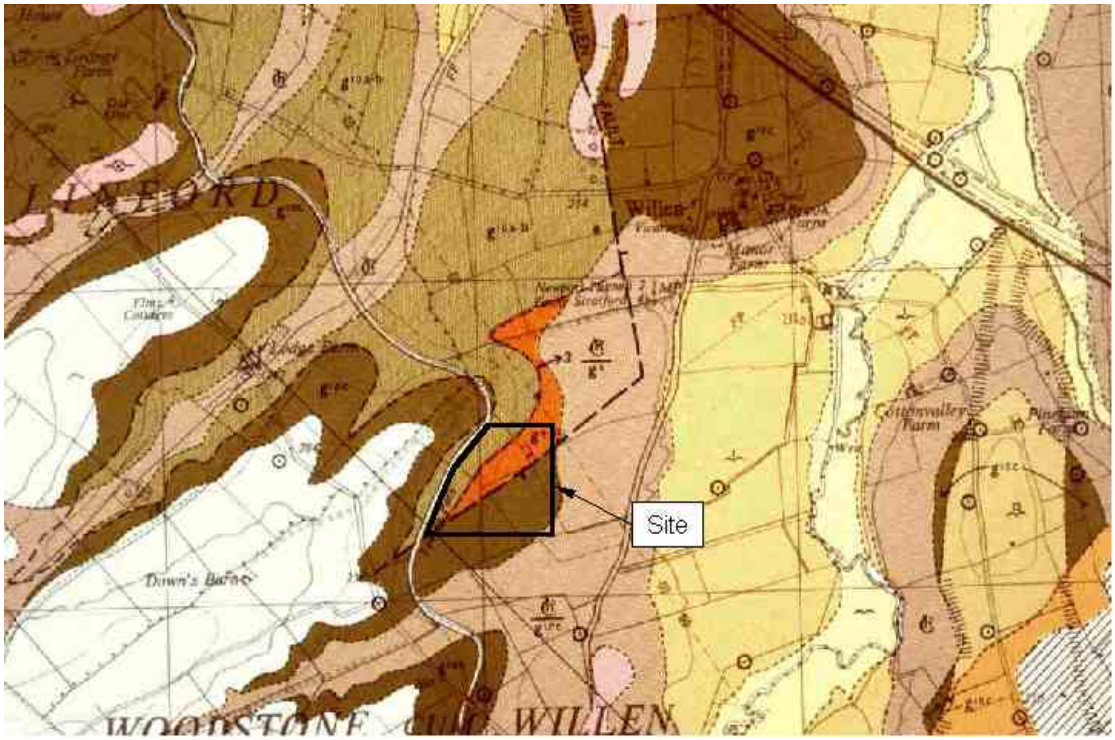


Plate 3.3 Part of the 1:25 000 geological map of Milton Keynes (Sheet SP83 and parts of SP73, 74, 84, 93 and 94). (Courtesy of the British Geological Survey)

Case study - Use of topographical maps and geological records to study housing subsidence

Figure 3.4 shows sections of topographic maps for an area in Dorset: Fig. 3.4a (1963) is the site before construction and Fig. 3.4b (1977) shows the development of detached houses in a cul-de-sac. The earlier map shows that the area of development was previously an excavation. The 1:50000 drift geological map of the area (Sheet 328) records that the site is underlain by Reading Beds, which in many parts of the UK consist of a highly shrinkable clay. This gives the clue that these excavations were formed to win clay for brickmaking. Enquiries at the Dorset County Records Office in Dorchester showed that brickmaking had been carried on at this site from at least 1811, when an Inclosure map showed what is now Watergates Lane as Brickyard Road.

Plate 3.4 shows a reproduction of the manuscript 6in. to 1 mile 'County Series' geological map of the area, held by the British Geological Survey library. The map, from 1895, contains notes on the nature of the materials seen in the area at the time of preparation of the map. These 'exposures' are no longer available. Further details of the materials in this area are given in the sheet memoir (Geology of the Country around Weymouth, Swanage, and Lulworth by W. J. Arkell (1947) and in Chapter XII there is specific reference to the particular brickpit in which the site now lies:-

> The Reading Beds are well exposed at Broadmayne, where for many years they have been made into the well-known speckled bricks. The last surviving brickyard of a line of six about 1/2 mile north-east of the village shows the following section. (see Table 3.4). According to the manager, who supplied the particulars of the lowest three beds, the Chalk lies 'a good way below'.

Table 3.4 Record of Webb, Major and Co's brickyard, Broadmayne (Arkell 1947)

Description	Thickness		Depth
	(ft.	in.)	(m)
Loamy soil full of subangu1ar flints and locally becoming a flint gravel	3-4	0	0.0 – 1.1
Red clay, mottled white	6	0	1.1 – 2.9
Brown loam with manganese nodules	10	0	2.9 – 5.9
Manganese concretionary sandrock; white quartz grains cemented in small black nodules; holds up water	2	0	5.9 – 6.5
White sand, mottled red (seen to7ft.)	8	0	6.5 – 8.9
Black flints, sparsely distributed	0	6	8.9 – 9.0
White tough clay, from which bricks have been made	1	0	9.0 – 9.3
Rough sand, proved to about	5	0	9.3 – 10.8

It can be seen that although a layer of shrinkable clay once existed in these brickpits, it was relatively thin (perhaps 2 m) and will have been worked out over much of the area. Loam is a sandy clay and does not have much potential for shrinkage. Below it, the sand layers should provide good founding layers. It is interesting to make comparison between this record and that produced in 1985 from a borehole just outside the area of the brickpit, details of which are given in Table 3.5.

Table 3.5 Borehole record, Broadmayne, 1985

Soil description	Depth (m)
Topsoil and made ground	0.0 - 1.3
Stiff grey and red mottled silty clay becoming sandy at 2.0m	1.3 - 3.3
Very weak yellowish grey and brown sandstone	3.3 - 8.7
Very stiff light grey silty clay	8.7 - 9.7
Inter-layered sand and clay	9.7 - 10.2
Weathered chalk	10.2 - 15.0

The generally sandy nature of the Reading Beds at this locality is confirmed, and it can be seen that there is quite good general agreement between the borehole record and the desk study information. A number of structures in the area near the disused brickpit have suffered from desiccation and foundation movement problems, because a significant thickness of shrinkable clay exists just below ground level. Within the pit, there may well be areas where all the shrinkable clay has been removed.

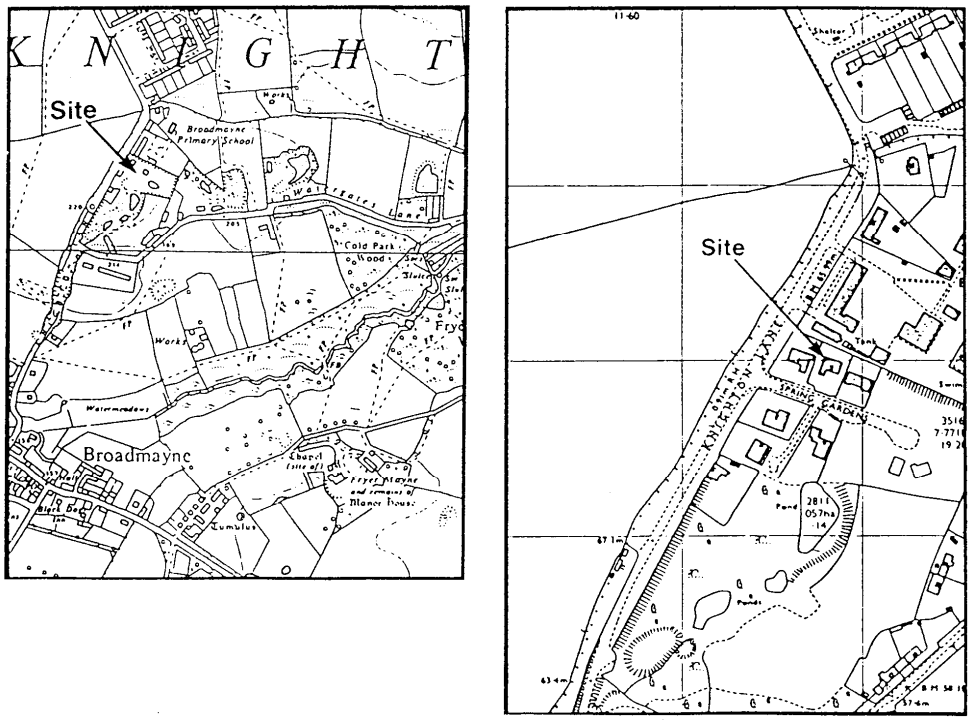


Fig. 3.4 Topographic maps of an area in Dorset showing developments in an old pit: (a) 1:10 560 scale map before construction (1963); (b) 1:25 000 scale map of site after construction (1977) (© Crown Copyright)

Mining records

Mining records are held by a wide variety of organizations, and may sometimes be very difficult to obtain. They are of obvious importance in assessing the effects of future mining activities on proposed structures, and in tracing the extent of past mineral workings which may collapse or settle below existing or proposed structures.

Mining in the UK is not restricted to deep coal mining; a wide variety of minerals has been extracted in the past, and is presently being obtained by both deep mining and quarrying. These minerals include stratified ironstone, shale, fireclay, limestone, chalk flint, sandstone, fluorspar, iron ore, gypsum, potash, galena, slate, gold, rocksalt, ball clay, Fuller's earth, tin and copper. In addition, considerable areas in the south-east of England have been quarried for sand and gravel.

A concise guide to the problems of construction over abandoned mine workings is given by Healy and Head (1984). They provide details of the many different types of mining that have been used over the centuries, and describe how a mining investigation should be carried out. Their methodology, shown in Fig. 3.5, relies on a thorough desk study and archival search both to assess the probability that mining has taken place, and to try to determine the extent of any mining activity. Appendix A of Healy and Head (1984) gives details of desk study sources of information useful during mining investigations.

The presence of mine workings can obviously be inferred from observations of spoil heaps, shafts and disturbed ground on standard records such as topographical maps and air photographs. More specific information can be obtained from geological maps and records, many of which show details of coal, iron deposits, limestone, sands and gravels, and other deposits of economic importance. Maps of the Second Land Utilisation Survey of Britain

provide partial coverage of England and Wales, and show the locations of extractive industries and active tips. These are available from the Ordnance Survey agents.

- Abandonment plans for coal and oil shale mines in the UK are now held centrally by: Mining Records Office, Operations Department, British Coal Corporation, Bretby Business Park, Bretby, Burton-on- Trent, Staffs DE15 OQD (tel: 01283-550500).
- Records of coal mine shafts and boreholes are held centrally by the Mining Reports section of the Operations Department, at the same address. When records of operational mines are required, these must be obtained directly from the working pit.
- For quarries and tips, and mines other than for coal and oil shale, Her Majesty's Inspectorate of Mines has now passed all records back to local authorities. These are held by a variety of departments, but primarily by county archivists.
- For salt mining, information on areas which are subject to brine solution mining, and which have been affected by subsidence can be obtained from: Cheshire Brine Compensation Board, 41 Chester Way, Northwich, Cheshire (tel: 01606-2172).
- For iron ore, some help may be obtained from the Iron Ore Mining Division of British Steel Corporation at: Iron Ore Mining Division, British Steel Corporation, Scunthorpe, Humberside (tel: 01724-843411).

General information on mining may be obtained from The Institution of Mining and Metallurgy, in London.

Mining has been carried out since time immemorial. Therefore one of the major, and unexpected, hazards that can affect construction is the collapse of ancient mine-workings. Figure 3.6 shows the result of such a collapse, which occurred in the city of Norwich (UK) in March 1988, probably as the result of the collapse of an ancient chalk mine, triggered by a waterpipe burst. The double-decker bus had just pulled away from a bus stop when the rear wheels sank into the road up to their axle. Within minutes the bus had sunk into a hole about 10m across, and 4m deep. Although the driver and passengers escaped unhurt, it was necessary to evacuate some 30 local residents because of gas leakages. The road was not re-opened to two-way traffic for more than a year, and the repair, involving the placement of a large volume of concrete in a complex of holes with a maximum depth reported as 17 m, was estimated to cost about £70,000.

Because of the wide occurrence of abandoned mines, it will be wise to carry out a thorough desk study when working in a new area, giving particular priority to the identification (via geological maps, old topographical maps and local historical records) of strata that may have been mined in the past, and of the problems that these cause (by reference to old newspaper articles, and technical journals). Speleological societies may provide useful records of underground cavities. In the UK, the Chelsea Speleological Society (previously the London Speleological Society) has a computerized database of records of over 2000 caves and mines in the south-east of England, compiled largely from their own records, but also from the records of industrial societies, archaeological societies, and natural history societies (for example, bat groups) Chelsea Speleological Society 1990).

Large national organizations whose work is affected by mining subsidence may hold records of abandoned mines near to their properties. In the UK, these include:

- British Rail (Derby);
- British Waterways (Leeds); and
- local authority mining valuers, who may hold data.

Also, in the UK, other sources of information on unstable ground include:

- the Ove Arup and Partners/Department of the Environment catalogue of areas of mining instability; and
- the Applied Geology Limited Natural Cavities Catalogue.

Case study - Use of geological maps to locate the presence of coal seams

Figure 3.7 shows part of the published 1:10 560 geological map of an area of south-east Bristol. The area, which had previously been the site of a jam factory (see later) was to be redeveloped as a large retailing facility, with provision for a petrol station, and parking for 700 cars. As part of the desk study for the project, the geological maps were examined. This map shows that the geological structure of the area is quite complex. The map"" gives considerable insight into the likely problems of redeveloping the site, showing alluvium in the valleys, and the presence of a fault and an unconformity. One obvious and important feature to be seen on the map is the existence of a number of thin coal seams, shown as thick black dashed lines. One of these (the 'Pot Seam') is likely to be at shallow depth beneath the proposed location of a large building, at the north end of the site. Thin coal seams (of the order of 300mm thick) are known to have been mined in some parts of the UK, and so the development of the site involved special and detailed drilling of this area, and subsequent grouting of the seams, which were found to lie at a depth of about 10 m below ground level.

Records to establish previous site use

Any plans for construction should take into account existing construction, and the impact that this may have on the proposed project. Nearby services, foundations and retaining structures may be adversely affected or, conversely, may cause difficulties for the new construction.

Case study - Use of construction records and technical journals to investigate the position and depth of old foundations

It is proposed to construct a service tunnel between St John's Wood (north London) and Clapham Common (south London). As part of the process of deciding on the precise route of the tunnel it was necessary to investigate the depths of foundations along the route, to ensure that the tunnel line would not pass through them. One of the largest structures along the route is in Victoria, and is currently occupied by the National Audit Office.

A desk study was carried out, to establish as far as possible the site history, and foundations used for the National Audit Office. The records used included:

- topographical maps;
- geological survey 'memoirs';
- geological maps;
- previous site investigation reports;
- articles in journals, including:
  - Railway Magazine (1960),
  - The Architect and Building News (1939),
  - Civil Engineering (1938),
  - Proceedings of the Institution of Civil Engineers (1946),
- British Rail drawings for Victoria Station; and
- approximately 70 engineering drawings, relating to the design of the building, many obtained from British Rail, and from the current occupiers of the building.

The site lies parallel to the tracks entering one of London's major railway termini, alongside what was once a small valley, containing a minor tributary to the River Thames. Available geological records indicate that the site is underlain by made ground, alluvium and London clay.

Records suggest that since the 17th century the site was adjacent to man-made waterways. In the 1830s these waterways were made navigable, and the site then became developed for wharfs and warehousing to serve what became known as the Grosvenor Canal, immediately to the north-east of the site. Initially, in the 19th century, the railway was constructed alongside the canal, but finally, around 1900, the canal basin was infilled to allow the development of the station terminus.

The building, as it stands today, was constructed in three phases. The records show that in 1937-38 the foundations for the first phase, including the large central tower, were planned. Articles in technical journals provided not only simplified borehole records, but also records of pile tests to establish bearing capacity. These records show the existence of an alluvial brown sand layer beneath the site, above the London clay. Current practice uses large-diameter bored piles to support heavy structures founded on the London clay, but in the 1930s the plant and technology were not available to construct them. It was therefore stated, on the basis of pile test results (Williams 1938) that 'the results of test piles definitely ruled out the use of piles…. to carry the very heavy loads of the main tower'. The tower was therefore founded on a shallow reinforced concrete raft, on the sand. Beneath the remaining areas of the building, developed over a period of almost 30 years, a wide variety of pile types and lengths was used. Either from records of pile length, or from a knowledge of ground conditions at the site, estimates of the maximum foundation depth were obtained, and compared with the proposed elevation of the tunnel.

As demand for land for development increases, there has been a rapid increase in the re-use of derelict industrial sites in the UK. The re-use of derelict land presents the following special problems to the geotechnical engineer.

1. Structural foundations must take into account the greater ground movements, and the possibility of attack of corrosive ground on foundations, which are often associated with made ground. Ground improvement may need to be carried out.
2. The special hazards to the health of construction workers, and to the occupiers of the site once construction has been completed, must be fully investigated. A preliminary estimate of the potential nature of these hazards can often be made on the basis of the precise nature of previous land use. Table 3.6 gives examples of the hazards associated with different types of land use.

The main stages to be carried out are an investigation of site history, using existing records, and a site visit (see later). The desk study must include a search of all available large-scale topographical maps (i.e. 1:10560 or larger), including maps that are now out of print.

Table 3.6 Contaminants associated with various industrial sites (from DD175: 1988)

Industry	Examples of sites	Likely contaminants
Chemicals	Acid/alkali works Dyeworks Fertilizers and pesticides	Acids; alkalis; metals; solvents, (e.g. toluene, benzene); phenols,

	Pharmaceuticals Paint works Wood treatment plants	specialized organic compounds
Petrochemicals	Oil refineries Tank farms Fuel storage depots Tar distilleries	Hydrocarbons; phenols; acids; alkalis and asbestos
Metals	Iron and steel works Foundries, smelters Electroplating, anodizing and galvanizing works Engineering works Shipbuilding/ shipbreaking Scrap reduction plants	Metals, especially Fe, Cu, Ni, Cr, Zn, Cd and Pb; asbestos
Energy	Gasworks Power stations	Combustible substances (e.g. coal and coke dust); phenols; cyanides; sulphur compounds; asbestos
Transport	Garages, vehicle builders and maintenance workshops Railway depots	Combustible substances; hydrocarbons; asbestos
Mineral extraction Land restoration (including waste disposal sites)	Mines and spoil heaps Pits and quarries Filled sites	Metals (e.g. Cu, Zn, Pb); gases (e.g. methane); leachates
Water supply and sewage treatment	Waterworks Sewage treatment plants	Metals (in sludges); microorganisms
Miscellaneous	Docks, wharfs and quays Tanneries Rubber works Military land	Metals; organic compounds; methane; toxic, flammable or explosive substances; micro-organisms

Note: Ubiquitous contaminants include hydrocarbons, polychlorinated biphenyls (PCBs), asbestos, sulphates and many metals used in paint pigments or coatings. These may be present on almost any site.

In the UK, the most useful maps are the 'County Series', produced from the mid-19th century up until the Second World War, and the Ordnance Survey National Grid mapping, which replaced the County Series. In addition to these plans, county libraries, muniment rooms, and archives will contain valuable historical information, for example:

- old mining records;
- trade directories (Kelly's - from about 1850);
- the Victoria County History;
- specialist archival materials on local industries;
- old newspaper files (also available from newspaper publishers);
- Public Health Act plans (c. 1850) which were prepared for the installation of the early Victorian sewer systems; and
- early maps, for example related to property ownership.

In addition, it will be useful to contact local waste disposal authorities (in the UK, the County Waste Disposal Authority), who may have information on old waste disposal sites.

Other records

Virtually any type of record may have practical significance to construction. Records which are less frequently used during desk studies include the following.

1. *Service records.* Service records are kept by utility companies, such as gas, electricity, telephone and water supply companies, as a record of their installations. These records have considerable significance when site investigations are planned in urban areas, where the presence and high density of services may put the safety of drilling operatives at risk.
2. *Soil Survey Records.* These are made primarily to give information on agricultural soil conditions. The results of pedalogical soil surveys are usually published as maps and accompanying soil survey records (in the UK, by the Soil Survey of England and Wales, who started this work in 1966). Soil surveys consider only shallow deposits, to a depth of about 1-1.5 m. The properties of these materials are related not only to vegetation and weathering, but also to the materials beneath them. Therefore soil survey maps reflect the underlying geology. In the UK, published soil survey maps exist at scales between 1:1 000 000 and 1:10 560, with the most useful cover being at 1:63 360 and 1:25 000 scales. Examples of the usefulness of this type of record are given by Aichison (1973), Allemeier (1973), McGown and Iley (1973) and Dumbleton and West (1976a).
3. *Meteorological records.* Meteorological records are of use in assessing future weather conditions, for example to provide an estimate of the time likely to be lost, during construction, as a result of bad weather conditions. They can also play an important role in the investigation of failures, or the investigation of lack of productivity of earthmoving plant. The Meteorological Office (Bracknell, UK) will provide certified statements of past weather (perhaps useful in preparing insurance claims), as well as the full range of specialist forecasting services for the construction industry.
4. *River authority records.* This form of record is of particular value in providing records of flooding, perhaps when construction is to take place close to a river.
5. *Earthquake records.* Although the UK occupies a relatively quiescent area of the world, this is not the case elsewhere. Large areas of Europe and the USA are subjected to frequent and intense earthquakes, but of varying magnitude. Earthquake records from around the world are available from the International Seismological Centre at Piper's Lane, Thatcham, Newbury, Berks RG13 4NF (tel. 01635-861022 fax. 01635-872351).Given a location (in terms of latitude and longitude) and a search radius, the date, time and magnitude of earthquakes, from approximately 1920 onwards, can be provided on payment of a fee. Local observatories will also generally be able to provide a similar service for their area.

AIR PHOTOGRAPHY AND REMOTE SENSING

Topographic and geological maps, although of great value in the preliminary desk study, give only a limited amount of data which is capable of geotechnical interpretation. The limitations of maps are a function of scale, subjectivity and frequency of revision. Features whose recognition is important for geotechnical purposes are often too small to be shown on conventional maps drawn at scales of 1:25000 or 1:50000. Such features may be shown on maps drawn at scales of 1:10000 or 1:2500, but in some cases the contour interval is too large for a minor topographic feature to be shown, or the subject of the map does not warrant the inclusion of certain features. It is not practical to indicate symbolically all surface features on a single map because the cost of surveying would be prohibitive and furthermore the map would be too complex to be read efficiently. Because of the time and cost involved in survey and map preparation, complete revision of maps is not carried out very frequently. The most frequent revision to existing maps is for major features such as roads and urban development. Minor revisions such as small changes in topography due to lands lip activity or changes in field boundaries are not always included. Geological maps undergo revision most infrequently. Of course the rocks are not expected to change over fifty or one hundred years,

but the interpretation, particularly in questionable areas and for superficial geology (drift) may change as new data become available.

The limitations of topographic and geological maps are overcome to a certain extent by the use of aerial photographs, which provide a detailed and definitive picture of the topography, lines of communication (roads, railways and canals), surface drainage and urban development. Furthermore, additional information is provided on land use, vegetation, erosion, and instability, which may be interpreted in geotechnical and geological terms. No surface detail is omitted in an aerial photograph, but some features may be obscured or hidden by vegetation (usually trees). Some detail may also be hidden by buildings, or diminished by the scale of the photographs.

The uses of aerial photographs are two-fold:

1. photogrammetry; and
2. air-photo interpretation.

Photogrammetry refers to the technique of making accurate measurements from aerial photographs and is discussed in detail by Kilford (1973). This technique requires the use of a certain type of aerial photograph (the vertical aerial photograph) which is discussed later. Photogrammetry is used widely in topographic surveying for map preparation, because the use of aerial photography is much less expensive than ground surveys. Clearly such a technique can be of immense value in site investigation for revising existing maps and plans, surveying remote unmapped areas, and siting boreholes. Most small-site investigation contractors however will not have the necessary facilities to carry out photogrammetric work.

Air-photo interpretation refers to the use of aerial photographs in the qualitative or semi-quantitative study of the character of the ground, or of vegetation or structures on it. Its fields of application are numerous. Military reconnaissance was probably the first application of air-photo interpretation, and World War II gave rise to some major advances in aerial photography for interpretation purposes. Since World War II air-photo interpretation has been applied extensively in the fields of geology, terrain evaluation, land use, agriculture, forestry, archaeology, hydrology, pedology, vegetation and environmental studies.

Air photographs are as readily available in most parts of the world as topographic and geological maps, and are similar in price. Indeed, in some areas (for example, Italy) published topographical maps consist of contoured vertical black and white air photography, with the names of prominent features superimposed upon them. The information that may be obtained from air photographs includes the following.

1. *Topography of the site and surrounding area.* The inclinations of slopes as well as small changes in topography may be seen from a three-dimensional image produced by viewing overlapping pairs of aerial photographs (the viewing of air-photos is discussed later).
2. *Geology of the site.* The superficial geology and solid geology can often be interpreted from features such as landforms, drainage patterns, land use, and vegetation.
3. *Site drainage.* The location of springs, seepages, poorly drained ground, ponds and potential flood zones can usually be identified from aerial photographs.
4. *Instability.* Landslip activity, whether recent or not, can often be identified. Examination of photographs taken at different times can be used to define the most active zones in landslip areas.
5. *Site history.* Previous uses of the site can be seen from a series of aerial photographs taken over a period of time.
6. *Site accessibility.* Gates and breaks in hedges or fences together with an overview of the general terrain to be covered can be seen on air-photos. This can be of great

assistance in planning the movement of drilling rigs and other equipment over the site.

7. *Identification and location of features of special engineering interest.* In some cases, features such as gulls, sink holes and mine shafts can be identified from aerial photographs.

It should be pointed out that much detail can be hidden by trees, particularly if the tree cover is dense. In such cases, aerial photographs may be of little use. The amount of tree cover can often be assessed prior to obtaining the air-photos of a site from topographic maps. The geotechnical and engineering geological interpretation of sites in the UK is discussed by Burton (1969), Norman (1969, 1970), Dumbleton and West (1970) and Norman et al. (1975). The importance of air photographs as a source of desk study information is hard to over-emphasize, not only because of their versatility, but also because of the unique role that they can play in allowing the identification of certain types feature which can be difficult, if not impossible, to detect in other ways, for example:

- slope instability; and
- solution features in carbonate rocks

and in addition because of their increasingly important role in providing a complete record of site conditions.

Types of photographic image

Two types of image are available for use in site investigation desk studies:

1. photographic images, taken by camera, typically from an aircraft (generally termed 'air photography'); and
2. digital images, typically produced by a multi-spectral scanner, most usually operating from a satellite (termed 'satellite imagery').

Air photographs are of most use during desk studies in developed countries, because the images are cheap, readily available, and do not need computer processing. As the resolution of multi-spectral scanners improves (currently SPOT imagery has a pixel size of about 10 m) it can be expected that this form of remote sensing will become more popular.

Generally the amount and type of data which may be obtained from aerial photographs will depend upon the following:

1. orientation of the camera axis with respect to the vertical;
2. the type of film and filters used;
3. the amount of overlap between adjacent photographs;
4. the scale of the photographs;
5. time of photography; and
6. season of photography.

The above parameters may be varied to suit the application for which the photographs are to be used. The interpretation of the photographic data will not only depend on the application but also on the type of aerial photograph which is a function of the first three parameters listed above.

Vertical / oblique photography

The types of aerial photographs available are normally defined by the orientation of the camera axis, the type of film and filters and the amount of overlap between adjacent photographs. Based on orientation of the camera axis, aerial photographs may be classified as either vertical or oblique. Vertical aerial photographs are those made with the camera axis orientated as near to vertical as possible. Truly vertical air-photos are rarely obtainable

because of the aircraft tilting and thus causing the camera to tilt. A slight unintentional tilt of about 10 to 50 from the vertical is normally tolerated for vertical photographs. Thus, for photogrammetric work, vertical aerial photographs are necessary. Aerial photographs taken with an intentional inclination of the camera axis to the vertical are termed oblique aerial photographs. Such photographs are classified as high oblique if an image of the horizon is included and low oblique if it is not.

Both oblique and vertical aerial photographs are used in site investigations. Vertical aerial photographs, however, are used more extensively than oblique aerial photographs for interpretation purposes. This is due to three important factors. First, vertical photography covering most of the UK is readily available, whereas oblique coverage is very limited. Secondly, accurate measurements generally cannot be made from oblique aerial photographs except in areas of low relief where co-planar control points can be used (Matheson 1939). Thirdly, the qualitative data obtained from oblique photographs are generally less comprehensive than those obtained from vertical air photographs for the following reasons.

1. The change in scale across the photographs can be rapid in the case of high oblique photographs and complex in areas of high relief.
2. The distortion of shapes on oblique photographs can give the wrong impression of the importance of a ground feature.
3. A considerable amount of ground can be hidden from view by hills (dead ground).
4. The production of a print laydown using oblique photographs is difficult and in many cases impossible.

Despite the disadvantages of oblique aerial photographs, they can be very useful in supplementing data obtained from vertical aerial photography. Some of the disadvantages can be overcome by using low oblique photography. Topographic features, however (particularly subdued topography), are generally shown more clearly on high oblique aerial photographs.

Case study - The use of oblique air photography to investigate the morphology of a landslide at Stag Hill, near Guildford, Surrey

The north face of Stag Hill near Guildford, Surrey, was the site chosen for the University of Surrey. The reason why a site so close to the town centre of Guildford had not been previously developed was because of a large landslip which occupies the north face of Stag Hill. This landslip occurred on a 9° slope in brown London clay (Fig. 3.8) and its extent is clearly visible on vertical aerial photographs taken before construction of the university began in 1967. An example of such a photograph (taken during 1961) is shown in Fig. 3.9.

The landslip is identified on Fig. 3.9 mainly by shadow and relief. The rear scarp and toe of the landslip form a small step in the slope which can be seen when the vertical air photographs are viewed stereoscopically (Fig. 3.10). These topographic features are enhanced by shadow allowing the limits of the slip to be estimated readily from a single photograph (Fig. 3.9). It will be seen from Fig. 3.9 that a line of trees and bushes marks the position of part of the rear scarp and toe. The topographic expression of these features appears to be more pronounced here than elsewhere, making cultivation across them very difficult. Thus natural vegetation has been allowed to become established on these parts of the rear scarp and toe. If early photographs of the site before development (1949 or before) are compared with that shown in Fig. 3.9 the increase in the amount of vegetation in these regions of the rear scarp and toe is most noticeable, and may indicate relatively recent movement.

Several coalescing landslips of different ages give rise to the overall feature seen in Fig. 3.9. Although the contact scale of the photograph in Fig. 3.9 is 1:4000, many of the minor features associated with the individual landslips are difficult to identify. The photographs were taken nearly two hours from noon, but the inclination of the sun's rays was clearly not low enough for shadow to be used to enhance the subdued topography of these features. The minimal

shadow normally required for vertical air photography can severely limit the detection of minor landslip features. As mentioned earlier, however, oblique photographs (when available) can often overcome this problem. It is fortuitous that the Stag Hill site is immediately adjacent to the site of Guildford Cathedral; numerous oblique aerial photographs have been taken of the Cathedral and most of these include the north face of Stag Hill. This allows the landslip features to be examined using oblique air photographs over a period of about 35 years (from the start of construction of the Cathedral to the start of construction of the university). The oblique air photograph shown in Fig. 3.11 was taken with a low sun angle. Minor topographic features associated with the landslip are clearly visible because of the long shadows. A feature indicative of a toe of an old landslip may be seen in the field to the north-east of the cathedral (marked A on Fig. 3.11). This feature cannot be identified on the vertical photographs. Both oblique and vertical aerial photographs have been used to produce a map of the Stag Hill site showing the probable sequence of landslips. This map is shown in Fig. 3.8.

Figure 3.12 shows the Stag Hill site in 1971 when the main phase of construction of the university was completed. The toe of the landslip is still visible at A. The contact scale of the photograph is 1:12000. At this scale, only the major features such as the toe of the landslip and some tonal contrast within the slipped area are identifiable. A comparison of Figs 3.9 and 3.12 illustrates the loss of detail with decreasing photo scale.

Aerial photographs can be of value in locating areas of landfill. Figure 3.9 clearly shows the location and extent of a brick pit in London clay to the east of the cathedral. Comparing this photograph with that shown in Figure 3.12, it will be seen that the brick pit has been backfilled and the university buildings sited to avoid the fill. It is always advisable to examine early aerial photography as well as recent aerial photographs in order to detect old as well as recent landfill sites. An example of the detection of a wartime military trench illustrates this point well, and is given later.

Film type

The type of film used in the camera gives rise to the following general types of aerial photograph:

1. black and white;
2. colour; and
3. infra-red (in colour or black and white).

The most common type of film used in aerial photography is black and white panchromatic film. The data obtained from black and white photography are normally sufficient for interpretation in most site investigation applications. The use of colour photographs, however, can provide additional information. This is because the human eye is capable of separating at least one hundred times more colour combinations than grey scale values (Beaumont 1979). Colour photography has proved most effective for geological interpretation (Fischer 1958; Chaves and Schuster 1964), but it is more expensive and less readily available than black and white photography and hence its use tends to be limited in site investigations.

Infra-red films record reflected radiation in the visible part of the spectrum, but are also sensitive to reflected infra-red radiation (to wavelengths of about 0.9ftm), which is invisible to the naked eye. The dyes used in colour infra-red film produce false colours. Blue images result from objects reflecting primarily green energy; green images result from objects reflecting primarily red energy, and red images result from objects reflecting primarily in the photographic infra-red portion of the spectrum (0.7-0.9 μm). Reflected energy of different wavelengths appears as different tones of grey in black and white infra-red photographs. The advantage of colour infra-red photography over black and white infra-red photography is basically the same as that mentioned earlier for normal colour photography compared with

normal black and white photography. Colour infra-red photography (or false colour photography) is used extensively in forestry, agriculture and vegetation studies. This is because differences in reflectivity between different flora and between healthy and unhealthy flora are most pronounced in the infra-red part of the spectrum.

Water totally absorbs infra-red radiation, making colour infra-red photography most useful for studying drainage. Springs and seepages can be easily located using this film. Objects which reflect primarily blue energy appear as black images on both colour and black and white infra-red photographs since blue energy is normally filtered out. Free standing unpolluted water therefore appears as a black image on an infra-red photograph. Polluted water is highlighted by colour infra-red photography.

Despite the additional data provided by infra-red photography, it is little used in site investigation. This is because the cost of infra-red photography is greater than normal colour photography, and the interpretation of such photographs is a highly specialized field requiring a knowledge of film processing technology. It should be pointed out that an understanding of how films are processed is also helpful for the interpretation of normal colour photographs.

The identification of ground features is greatly aided by the use of multi-band photography. This technique allows small wavelength bands or spectral regions of the visible and invisible parts of the spectrum to be sampled. This is achieved using different filters and film types. Multi-band photography is carried out using a special multi-lens camera (generally four lenses are used) which allows different bands to be sampled at the same point in space and time. Within a single band, certain ground features will be enhanced while others are suppressed. Thus an examination and comparison of the images produced within each band enables ground features which are normally subdued in conventional photographs to be identified easily. Multi-band photographic systems are limited to the 0.3-0.9 μm spectral range because of the spectral sensitivity of photographic film. Multi-spectral scanners described by Lillesand and Kiefer (1979) are capable of sampling a far greater range of wavelengths (0.3-14 μm) and should not be confused with multi-band photography. Multi-band photography has been used with success in the fields of photogeology (Ray and Fischer 1960; Fischer 1962), soil mapping (Tanguay and Miles 1970) and archaeology (Hampton 1974). Some of the applications of multi-band photography in site investigation would include detection of sink holes, abandoned mine shafts, areas of fill, and unstable ground. However, multi-band photographs of the UK are not readily available and hence have to be specially flown. This severely restricts their use in site investigation on the grounds of cost.

Amount of overlap

The interpretation of aerial photographs and photogrammetric measurements involves the utilization of stereoscopic viewing to provide a three-dimensional image of topographic relief. This effect is possible because our eyes are separated by a small distance (eye base) allowing objects to be viewed simultaneously from two different positions. The eye base is such that the brain is able to merge the two images resulting in a three-dimensional image. This phenomenon is termed depth perception. If two aerial photographs overlap, the objects within the area of overlap will be seen from two viewpoints as a finite distance exists between the centres of the photographs (the distance between camera positions is termed the air base). Thus if the left photograph of the pair is viewed with the left eye and the right photograph with the right eye, a three-dimensional image (stereomodel) of the area within the overlap can be produced. A pair of photographs which can be viewed in this way is termed a stereopair. The stereo viewing of aerial photographs is aided by the use of a stereoscope. The simplest stereoscope (pocket stereoscope Fig. 3.13a) merely consists of a pair of lenses mounted on a frame. The photographs have to be placed close together when using this device which makes stereoviewing of stereopairs with a large overlap difficult without bending the photograph.

The mirror stereoscope optically extends the eye base allowing a stereomodel to be formed with the photographs separated by several centimetres as shown in Fig. 3.13b.

Clearly, the amount of overlap between adjacent aerial photographs will affect the area which can be viewed stereoscopically. An overlap of 60% ensures that the whole area covered by each photograph in a traverse line (run) can be viewed stereoscopically with the exception of the first and last photographs in the traverse. This amount of overlap is normally used for air-photo interpretation and photogrammetric work. The minimum overlap generally accepted for stereoscopic viewing is 20%. This amount of overlap allows only a small area to be viewed stereoscopically. In a sequence of photographs taken along a traverse line, 60% of each photograph cannot be viewed stereoscopically. The use of such a small overlap is thus limited.

The effect of viewing stereopairs with an eye base far less than the air base of the photographs and a viewing height considerably less than the distance between the camera and the ground (flying height) is the exaggeration of vertical scales. This is referred to as the vertical exaggeration. The amount of vertical exaggeration is dependent upon the ratio of air base to flying height. The larger the air base/flying height ratio, the greater the vertical exaggeration. Clearly if the flying height remains constant, then the vertical exaggeration will increase with decreasing amount of overlap between adjacent photographs. For 60% overlap, the terrain is normally seen exaggerated in height by about three or four times. The vertical exaggeration will also cause slopes to appear steeper than they are in reality. Figure 3.14 shows the relationship between actual slope angle and apparent slope angle.

Scale

The amount of detail that can be seen on an aerial photograph will depend to a large extent on the scale of the photograph. Unlike a topographic map, the scale of vertical aerial photographs varies in relation to the terrain elevation. Thus if the differences in elevation are small, the variation in scale is small, but large differences will result in significant variations of scale. For this reason an average scale (contact scale) is given which is a function of the focal length of the camera and the average flying height above ground level:

$$S_{av} = \frac{f}{H - h_{av}} \tag{3.1}$$

where S_{av} = contact scale, f = focal length of camera, H = flying height above sea level and h_{av} = average terrain elevation.

The scale of aerial photographs can vary from about 1:4 000 000 (for photographic images produced by satellites) to about 1:2000. Plate 3.5 shows the area of ground covered by air photographs (conventional 230 x 230 mm contact prints) taken at different scales. Clearly as the scale increases the area covered by the photograph decreases making it difficult to place large features such as major landforms into an environmental setting.

The optimum range of photo-scales for more local geological surveys is between 1:10 000 and 1:20 000 (Norman 1968b). Webster (1968) points out that at photo-scales less than 1:40000 meaningful interpretation becomes very difficult since much of the topographic detail used as landmarks in walk-over surveys is lost and the attributes of the land used to detect soil differences become indistinct. This scale marks the lower limit for most air-photo interpretation purposes, particularly in the UK. Much larger photo scales are required for geotechnical interpretation. There are two reasons for this. First, the geotechnical maps produced during site investigations are normally drawn at scales larger than 1:10 000, and the air photographs used in the preparation of these maps should be at the same scale or larger.

Secondly, many ground features that are used to aid geotechnical interpretation become indistinct at small photo-scales. The optimum range of photo-scales from which meaningful geotechnical interpretation can be carried out is between 1:2500 and 1:10 000. Most of the available SI aerial photography of the UK is at a photo-scale of between 1:10 000 and 1:30 000. The most popular photo-scale appears to be about 1:10 000 and hence much of the geotechnical interpretation must be carried out at the minimum scale in the range mentioned earlier on the grounds of general availability.

Time and season of photography

The time of day at which aerial photographs are taken can have a great. influence on the appearance of ground features. The controlling factor is the sun's elevation. At low elevations (i.e. during early morning or late evening) long shadows are cast by objects on the ground. The shadow will enhance subdued topography which may be associated with features of geotechnical interest, such as areas of slope instability (Fig. 3.11). Hackman (1967) has shown in model tests that subdued morphology becomes more distinct when the sun is very low and the inclination of its rays are 100 or less with respect to the horizontal. The subdued pattern of relief associated with many archaeological features makes low sun angle photography extremely useful in aerial archaeological surveys. Examples of archaeological features enhanced by shadow are shown in Figs 3.28 and 3.29.

While shadow can be an aid to interpretation, it can also be a hindrance, particularly in areas of high relief, since important detail may be hidden. It is for this reason that most vertical air photography is taken during late morning or early afternoon when the sun's elevation is at or near maximum, and hence the amount of shadow is minimal. Low sun angle vertical aerial photography is therefore not readily available for most of the UK. Oblique air photography taken during early morning or late afternoon tends to be more common, but existing oblique photography is very limited in terms of the areas covered.

The season in which aerial photographs are taken is often a critical factor in detecting features of geotechnical interest, such as soil patterns, areas of instability and springs and seepages. Soil patterns may be used to locate old stream channels (Fig. 3.18), areas of peat, and to obtain an overall idea of soil types and their variability over the site. Webster (1968) points out that most surface differences in soil depth and type are most apparent on aerial photographs when there is a large range of soil suction. The larger the soil water deficit, the larger the total range and the greater the tonal contrasts produced on the photograph. On these grounds, Webster has found photography taken in later summer (August and September) to be best. Evans (1972) suggests that most soil patterns are clearly shown on photographs taken during March and April (Fig. 3.15). For the photogeological interpretation of faults, rock contacts and soil contacts, Norman (1968b) recommends photographs taken during the autumn. However; it should be pointed out that of the photographs examined by Norman, less than 5% were taken during autumn. In some cases, features of geotechnical interest may be hidden by crops in cultivated areas. In such cases photographs taken during the months when the ground is bare of crops will be necessary. Springs and seepages may not be visible on photographs taken during summer when the groundwater table is low. In general, photographs taken during spring or autumn will normally provide sufficient data for the interpretation of most geotechnical features. Of course the choice of season is limited to a large extent by the availability of existing photography.

In the UK, periods of good weather favourable for air photography (i.e. minimum cloud and good visibility) amount to less than 500h in a year. The summer offers the most settled conditions, but spring and autumn have the highest proportion of days with good visibility (St Joseph 1977) and hence photographs taken during these seasons should be readily available.

The interpretation of aerial photographs

The ability to interpret aerial Photographs for site investigations depends primarily on having a knowledge of geology, geomorphology and geotechnics, and acquiring experience in recognizing features of interest from the air. Keen powers of observation together with imagination and patience are clearly necessary prerequisites for this exercise. The detailed analysis of ground features which appear on aerial photographs not only requires a great deal of experience, but also a knowledge of the science of photography. This can be a critical factor when interpreting colour, infra-red and multi-band photography. Interpretation of air photographs without studying all the available information concerning the site is of little value even if the interpreter has satisfied the above requirements.

Good use can generally be made of black and white panchromatic and colour aerial photographs without special experience or an extensive knowledge of geology, geomorphology and photography. However, the specialist interpreter will be capable of making a much more detailed analysis of the photographs. Large organizations involved in site investigation are likely to employ a geologist, engineering geologist, or geotechnical engineer who is able to make such detailed interpretations. Smaller organizations lacking such expertise can make use of companies which offer an air photograph interpretation service. Examples of such UK based companies are given by Burton (1969).

The limited availability (of existing photographs) and the high cost of colour and infra-red aerial photography compared with that of (black-and-white) panchromatic film result in the latter being used extensively for interpretation purposes in site investigations. The following discussion is essentially devoted to the interpretation of black and white vertical aerial photographs although much of it will apply equally to the interpretation of oblique photographs.

Air-photo interpretation involves a systematic examination of each stereopair covering the area under consideration. Anyone looking at a photograph for whatever purpose is performing interpretation. However, an aerial photograph of the area which includes one's house is more meaningful than an aerial photograph of another part of the country. This is because one's knowledge of the area in which one lives is generally greater than one's knowledge of other areas. This fundamental difference in capacity for interpretation is clearly a function of the amount of knowledge stored in the mind of the interpreter and is termed the reference level of the interpreter (Tait 1970). In practice, the successful interpretation of aerial photographs depends upon the basic reference level of the interpreter, the degree to which this basic reference level can be extended for each site, and the ability of the interpreter to make full use of interpretative aids, such as image and physical characteristics.

For site investigations the basic reference level required by the interpreter is a knowledge of physical and cultural features and their relationships with geology, geomorphology and land use. A study of these features should enable points of geotechnical interest to be identified. Ideally the interpreter should be familiar with the site and surrounding area. The necessary extension of the basic reference level is achieved to some extent by studying all the data collected during the preliminary desk study. At the very least this should include topographic and geological maps together with soil survey maps when available. Further familiarization is provided by a walk-over survey of the site.

Aids used in interpretation

Every photograph must be treated on its own merits, and hence there are no rules which define how a photograph should be interpreted. This makes teaching photo-interpretation for any purpose difficult without practical examples. The interpreter will only improve with practical experience. However there are several aids which are recognized as necessary for

successful interpretation. These aids may be termed collectively as image characteristics and physical characteristics as in Table 3.7.

Table 3.7 Aids to interpreting a photograph

Image characteristics	Physical characteristics
Shape Size Pattern Shadow Tone Texture Site	Landforms Vegetation Land use Drainage and erosion Lineations (natural and man-made)

IMAGE CHARACTERISTICS

1. *Shape*. Shape refers to the shape of features seen from the air. Man-made features are normally characterized by straight lines or regular curves, and hence are often recognizable by shape alone. Many natural features have distinctive shapes. For example, mudflows are generally lobate and sink holes are commonly circular. However, in general, natural features may be difficult to identify on the basis of shape alone, and require other image or physical characteristics to be taken into consideration.

2. *Size*. The size of objects can often aid identification. It should be pointed out that the size of objects on the photograph must be considered in relation to the scale of the photograph, or in relation to objects of known size to avoid misinterpretation. Vertical exaggeration will make objects look much higher than in reality.

3. *Pattern*. The spatial arrangement of features on the ground often gives rise to patterns. The most noticeable pattern seen on most aerial photographs of the UK is a 'patchwork' of fields. Land use patterns are essentially a physical characteristic and are discussed later. Variation in near-surface soil types may give rise to distinctive patterns such as polygons and stripes (Fig. 3.16). Drainage patterns can generally be related to soil or rock type together with geological structure. The use of drainage patterns in interpretation is discussed later. A regular pattern of lines (lineations) is often present on exposed rock surfaces as a result of bedrock jointing. If there is a single dominant joint direction and the joints are closely spaced, the resulting texture is similar to that of wood grain. Bedding in rock can also produce distinctive patterns.

4. *Shadow*. The shape of objects and relief shown on aerial photographs are enhanced by shadow. As mentioned earlier, the amount of shadow can be varied by the choice of the time of day at which the photographs are taken. Shadow affords a profile view of objects and can aid interpretation considerably. However, shadow can hinder interpretation by obscuring important detail.

5. *Tone*. Tone refers to the colour or reflective brightness of features shown on the photographs. Thus differences in reflectivity of surfaces give rise to tonal variations which may be associated with differences in composition, colour, or moisture content of the materials forming the surface. Most of the features which appear on aerial photographs are identified on the basis of tone or tonal variations. The patterns discussed earlier appear as repeated tonal variations on the photographs. Fine tonal variations give rise to textures which are discussed later. In general, dark tones are indicative of wet conditions or dark-coloured materials such as basalt, or peat, while

light tones indicate dry conditions or light-coloured materials, such as chalk. The relationship between tone and moisture content is clearly useful in the recognition of seepages, springs, and water-logged or marshy ground from aerial photographs.

6. *Texture*. The frequency of tonal change on a photograph can give rise to distinctive textures which can aid interpretation. Generally, texture (as seen on the photograph) is produced by an aggregation of features which individually are too small to be identified. Thus texture is not a function of photographic tone alone, but also the shape and size of the individual features, together with the pattern of shadows produced by them. Trees are easily recognized on aerial photographs owing to the characteristic texture produced by tree leaves. The tonal variations produced by crop or cultivation patterns could be considered in terms of texture. Texture is most useful in the identification of slope instability. The hummocky ground (often enhanced by shadows), and the impedance of drainage give rise to variations in tone which often result in characteristic textures.' A 'turbulent' texture is commonly associated with landslips that have involved the flow of material down slope (e.g., solifluction lobes and mudflows). An example of this texture is given in Fig. 3.36.

7. *Site.* The location of objects or features in relation to other objects or features can be an important aid to interpretation and reduce the possibility of misinterpretation. For example, dark-toned arcuate features observed in river valleys may be interpreted as infilled ox-bow lakes but the interpretation of similar features which are not associated with river valleys would be different.

PHYSICAL CHARACTERISTICS

1. *Landforms.* When aerial photographs are viewed stereoscopically, the first feature that is noticed is relief. Using the stereo image, various distinctive landforms may be identified and hence the area covered by the photographs can be broken down into major landforms. Such landforms are often related to different soil and rock types. For example, areas underlain by soft rocks such as clay will tend to form relatively flat ground whereas hard rocks such as limestone are usually characterized by steep slopes. The type of landform is not only a function of resistance to erosion but also geological structure. For example, dipping strata with alternating resistance to erosion give rise to the distinctive landform of dip and scarp slopes which trend parallel to the strike. Such landforms may be modified by the presence of boulder clay. In order to understand fully the significance of landforms in relation to the underlying geology, a knowledge of geomorphology is required.

2. *Vegetation.* Vegetation in cultivated areas is controlled by various environmental factors, the most important of which are soil type and the availability of water. Moisture conditions and the general groundwater regime can be surveyed by observing hydrophilic vegetation (Svensson 1972). Other assemblages of flora may be indicative of different soil types, but it is very difficult to identify plant species from aerial photographs without experience and a knowledge of botany. Different types of vegetation can be identified by interpreters without specialized knowledge on the grounds of tonal differences. Thus, the area covered by the photographs can be broken down into sub-areas based on vegetation. These data may be used in conjunction with other physical characteristics in interpreting soil and rock boundaries together with other features of geotechnical interest. Clearly the season of photography is important in making the best use of vegetation as an aid to interpretation. Svensson (1972) shows that for areas in the temperate zone an extreme situation during the period preceding the photography is required to give the most informative picture of cultivated areas. Such an extreme situation would be a period of little rainfall.

3. *Land use.* Land use may be divided into four broad categories: (i) agriculture; (ii) urban development; (iii) moorland; and (iv) materials extraction (quarrying and mining). In general the most useful category for interpreting ground conditions is agriculture. Farming in the UK is intensive, particularly in the lowland areas of the country. During the long history of farming in Britain, farmers have adjusted their farm management to take account of the differences in soil type (Webster 1968). This is often reflected in the size and pattern of fields. Webster (1965c) discusses the significance of land use in relation to different soil types in the Upper Thames Valley. The well established boundaries between cultivation and moorland usually mark changes in both soil and rock type. Where superficial deposits are relatively thin, the type of cultivation may be indicative of underlying rock types.

4. *Drainage and erosion.* Drainage patterns are easily identified from aerial photographs. The type of drainage pattern and the density of the drainage network (texture) of the pattern, are often indicative of the types of soil/rock beneath the site. In some cases the area covered by the photographs will not be sufficient to show the complete drainage pattern. However a more complete picture may be obtained from topographic maps. 1.

   Dendritic patterns indicate generally homogeneous materials. The texture of such patterns is related to the permeability of the underlying materials. Coarse-textured patterns (Fig. 3.17a) develop on materials with good internal drainage with little surface run-off. Fine-textured patterns (Fig. 3.17b) develop on materials with poor internal drainage and high surface run-off. The texture is also a function of resistance to erosion of the underlying materials. Coarse textures tend to be associated with resistant materials such as granite, and fine textures with easily erodable materials, such as clay and shales. A good example of a dendritic pattern is seen in the infilled channels shown in Fig. 3.18.

Case study - Polygons and stripes shown by differential ripening

The aerial photograph shown in Fig. 3.16 is of an area east of Narborough, Norfolk (TF 765113) which is in the chalkland north of the Breckland District. The geology of this area is essentially chalk overlain by superficial deposits. These superficial deposits comprise dark brown sandy drift over a very pale brown sand chalk drift (Evans 1972). The patterned ground is shown clearly on the aerial photograph by the tonal contrast produced by differential ripening of crops. Such vegetational patterns (crop marks) are not usually so clear unless photographed under extreme weather situations. The photograph in Fig. 3.16 was taken during the summer of 1976 which was a period of severe drought in Britain. Thus the differential ripening of crops strongly reflects variation in the underlying soil types. The distinctive pattern produced is related to undulations of the chalk (Fig. 3.19). The dark tones indicate where the chalk is nearest to the surface and the light tones indicate the infilling of ice wedges, formed during the Pleistocene. On sloping ground solifluction has drawn out the polygonal pattern to form stripes. Two sets of stripes are seen on Fig. 3.16 marking a slight linear depression running WSW - ENE across the eastern part of the area shown.

In general, polygons and stripes may show up on air photographs as a contrast of light coloured (calcareous) and dark coloured (non-calcareous) soils in fallow fields, or as crop marks resulting from differential ripening, differential growth, disease or patterns created by different assemblages of natural vegetation. Clearly the season of photography and the conditions (physical and meteorological) preceding the

photography effect the way in which these features appear on the photographs. Most of the patterns observed by Evans (1972) were on photographs taken when the fields were without crops (during March and April). The use of air photographs in studying patterned ground is discussed by Perrin (1963).

Well-developed polygons and stripes tend only to occur in the presence of a thin layer of sandy drift overlying chalk. The patterning appears to be particularly sensitive to the thickness of the superficial deposits, and will not occur when it exceeds about 2.5 m (Watt et al. 1966). The patterns cease abruptly on the eastern and southern margins of the Breckland area where the sandy drift is underlain by boulder clay instead of chalk. Small polygonal patterns have been observed on loamy sands overlying Triassic derived boulder clay and on sandy clay overlying Corallian limestone (Evans 1972).

Patterned ground is commonly produced in permafrost areas by frost action. Such features were produced in the periglacial zone of the UK during the Pleistocene, and have been preserved as fossil patterned ground. These fossil patterns are most common in the Breckland District of East Anglia and were first described in this country by Watt (1955). In plan the patterns are reticulate or polygonal on level ground and on slopes of less than 1° (Evans 1972), but with increasing slope they become first vermicular and then lengthen into stripes which may be simple or anastomosing (Watt et al. 1966). Stripes occur on slopes of 1°-5° but rarely on 6° slopes (Evans 1972). The polygons or near-circular features are generally between 5 m and 10 m in diameter and the interval between the stripes is commonly about 7 m.

The drainage and erosional features used in identifying the three commonest glacial soil types are shown in Fig. 3.20. These features may be used to identify soils of similar composition but of non-glacial origin.

5. Lineations. Many features, both natural and man-made, appear on aerial photographs as lineations. Such features are commonly the linear expression of characteristics such as tone, texture, landforms, drainage and vegetation. In some cases, these characteristics may be disrupted in a linear fashion, thus giving rise to linear features. Natural lineations can be used to interpret soil and rock boundaries, together with structural features such as faults and bedrock jointing. Norman (1968a) has made an extensive study of the significance of natural lineations in photogeology. Some lineations represent buried features of man-made origin, such as services and infilled trenches. An example of such a linear feature is shown in Fig. 3.33.

Use of air photography

In general the interpretation of aerial photographs involves four stages:

1. preliminary examination;
2. detailed examination;
3. interpretation; and
4. compilation.

The preliminary examination stage involves obtaining a general overview of the site and surrounding area, allowing the interpreter to become familiar with the area. The basic reference level of the interpreter is extended during this stage by the examination of maps and reports concerning the site or nearby sites together with the aerial photographs. In order to obtain an overview of the site, it is often necessary to work from the whole to the part. This is best achieved either by laying the photographs side by side or by cutting the photographs to make a print laydown. An example of a print laydown made from six individual photographs

is shown in Fig. 3.36. The preparation of a print laydown requires an extra set of photographs since there must be an undamaged set retained for stereoscopic examination. The photographs should be examined stereoscopically during this stage to observe the topography of the site.

Little distinction is made between the detailed examination stage and the interpretation stage, as in practice they are normally carried out simultaneously. The detailed examination stage involves a systematic stereoscopic examination of each stereopair. Full use is made of image and physical characteristics. It is often useful to separate each physical characteristic by making a series of overlays. Overlays may also be used to separate some image characteristics such as tone and texture. These overlays may be combined or examined individually when attempting to interpret certain features. In many cases, small features of geotechnical interest are easily missed because they are overshadowed by larger features of lesser relevance. The most efficient method of searching for such features is that of logical search (Tait 1970) which involves the use of probabilities and some systematic scanning. The areas of the image in which the features are most likely to occur are singled out and examined first. The remaining area can be systematically scanned. An overlay showing roads, tracks, paths and breaks in field boundaries can be made during this stage, for use in planning access of drilling rigs and site vehicles. Estimates can also be made from the photographs of widths of entrances and tracks since these are critical to the access of drilling rigs.

The interpretation of data collected during the previous stages should be carried out in terms of:

- geology;
- geotechnical hazards;
- site drainage;
- archaeology; and
- access for plant.
-

Geology

The interpretation of underlying geology from air photographs (termed photogeology) is a specialist skill which uses information contained on the photographs relating to geomorphology, drainage patterns, erosion features, slope instability, land use, vegetation, and lineations, together with image characteristics, to determine the soil and rock types, and the structure (i.e. dip, folding and faulting, etc.) of the beds beneath a site. Even in areas of minimal vegetation, where there is maximum exposure of the bedrock, this requires very considerable training, and is beyond the skills of most geotechnical engineers and engineering geologists. In temperate regions, such as the UK, where superficial deposits may be relatively thick, and vegetation is often controlled by agriculture, photogeology is made even more difficult.

The use of photogeology in engineering geology is discussed by Belcher (1946), Ray (1960), Miller (1961), Mollard (1962), Allum (1966), Norman (1968a,b,1969), and Lillesand and Kiefer (1979). The use of airphotography in soil science is discussed by Webster (1965a,b,c), Webster and Wong (1969), Evans (1972) and Perrin (1977). In most developed countries it will not be necessary to use air photographs to deduce the general geology of a site area, because good quality geological maps will normally be readily available. Photogeology may, however, be of use in determining the detailed ground conditions in the area of proposed construction, as the following example shows.

Case study - Use of large-scale aerial photography for the interpretation in detail of ground conditions at a proposed earth-dam site in North Wales

THE BRENIG DAM SITE, NEAR DENBIGH, CLWYD, NORTH WALES

The Brenig reservoir is situated south-west of Denbigh, Clwyd, North Wales, as shown in Fig. 3.21. This reservoir together with the adjacent Alwen reservoir (Fig. 3.21) provide water for Birkenhead. Investigations for a dam in the Brenig Valley were initiated at the end of the nineteenth century. The site chosen for the dam is shown in Fig. 3.22. The main Brenig Valley here is wide, relatively flat-bottomed and generally shallow with a small valley incised on the eastern side. The preliminary subsurface investigations carried out in 1948 indicated the main valley to be a pre-glacial till-filled valley underlain by shales and gritstones of Silurian age. Subsequent design investigations were carried out during 1969, 1972, and 1973 (Carter 1983). The main objectives of these investigations were:

1. to determine the nature of the bedrock and the depth to bedrock across the main valley;
2. to determine the geology of the drift deposits at the dam site and locate any weak clay or highly permeable sands and gravels;
3. to take undisturbed samples of till and any weaker layers in order to measure their strength and compressibility in the laboratory;
4. to measure the permeability of any sand or gravel layers within the foundations;
5. to locate suitable construction materials and determine whether there were sufficient quantities to build the dam;
6. to determine whether the reservoir basin was sufficiently permeable to allow reservoir impounding to the full height of the proposed top water level.

Clearly, air photography can provide valuable information which could aid in fulfilling objectives (2), (5), and (6). The basic interpretation of the black-and-white aerial photographs taken of the dam site is described below. The contact scales of the original photographs were 1:12000 and 1:3000. (The illustrations shown here have been reduced.) It is unusual find photography taken at contact scales as large as 1:3000 in areas of high relief such as this due to the obvious difficulties of maintaining a constant altitude. Such large-scale photography was necessary in this case to identify soil boundaries due to the complex heterogeneous nature of glacial deposits.

The main topographic features associated with the project area are best identified by examining the small-scale photography (Fig. 3.23) stereoscopically. These features include three drumlins (A, B, and C in Fig. 3.22) and a minor steep sided V-shaped valley (D in Fig. 3.22). Drumlins are glacial features composed of till moulded into smooth elongate hills resembling inverted spoons. Normally drumlins have a length to breadth ratio of about 2.5:1 with the long axis more or less parallel to the direction of ice movement at the time of formation (Boulton 1972; McGown et at. 1974; McGown and Derbyshire 1977). Another characteristic feature of drumlins is that the end which faced the oncoming ice is generally steeper than the down-ice end. Drumlins A and B in Fig. 3.23 are clearly defined on the grounds of topographic relief, shape and association. Association is probably the most important element in recognizing these hills as drumlins since such features are associated only with areas which have been subject to glaciation. The Brenig area is known to have been subject to these conditions during the Pleistocene period. A significant area of seepage is clearly visible on the north-west. side of drumlin A and around the southern end of drumlin B (springs and seepages are marked S in Fig. 3.22) indicating the presence of permeable material, possibly glacial sands and gravels. Drumlin C is smaller and less well developed than the others.

Thirty-five drumlins have been identified from the 1: 12 000 scale aerial photographs within the immediate vicinity of the dam site (Carter 1983). The long axes of these drumlins are shown in Fig. 3.24a. The rosette (Fig. 3.24b) indicates a general NE to SW trend. The form of these, drumlin A and around the southern end of drumlin B (springs and seepages of soil fabric and the pattern of discontinuities within some glacial deposits (e.g. lodgement till», are

closely related to the direction of ice movement. Soil fabric and discontinuities have a strong influence on the geotechnical behaviour of engineering soils (Rowe 1972), hence the determination of the direction of ice movement is of importance. The scale of photography is important in the identification of these drumlins. At 1:12000 these features can be easily viewed in context with their surroundings. At large scales, the task is made more difficult. For example, at 1:3000 drumlin A almost fills the area covered by a 230 x 230mm contact print. At the other extreme, a contact scale of 1:80000 may result in these features becoming too small to be readily identified, particularly by the inexperienced interpreter.

The eastern side of drumlin A has been oversteepened by the erosive action of the Afon Brenig. Much of this erosion probably occurred during the downwasting of the last glaciers which occupied the Brenig and adjoining valleys. The steep-sided channel thus formed was clearly a critical factor in the choice of the final dam site. Stereoscopic examination of D reveals the east side of the channel to be steeper and less rounded than the western side. Furthermore at E (Figs 3.22 and 3.25) there is a feature which probably represents an old landslip scar, since it occurs in a place where the river is most likely to undercut the slopes of the drumlin. A similar but smaller feature is seen to the north of E in Fig. 3.22. No similar landslip features exist on the eastern side of the channel. In fact, where landslipping is most likely (at F in Fig. 3.22) the slopes appear to be very steep and stable. This suggests that whilst the western side of the channel is clearly formed of till (part of the drumlin) the eastern side is composed of a much stronger material, possibly bedrock. An outcrop of bedrock can be seen at G (Fig. 3.22). This outcrop was quarried for construction materials. The existence of bedrock in the east side of the minor valley was confirmed by site inspection (Carter 1983).

The ground between the drumlins A and C (Fig. 3.22) is traversed by a series of drainage ditches. These are clearly identified by pattern and the fact that the spoil has been placed along one side of each ditch in most cases. These ditches and the dark tones suggest poorly drained ground. Peat probably covers most of the area between the drumlins giving rise to the dark tones. In places small ridges of till rise above the peat and are indicated by lighter tones. The stream on the west side of drumlin C is fed by a series of springs and seepages identified by a marked tonal contrast. These springs and seepages indicate an extensive horizon of permeable material in the west side of the main Brenig Valley. The dark tones associated with the stream probably result from the presence of peat.

In order to carry out a more detailed examination of ground conditions, photography taken at a larger scale is necessary. Figure 3.25 shows the minor valley at a suitable scale for mapping complex soil boundaries and locating small features which may prove significant. Wet areas probably associated with peat are clearly seen on the photographs. Careful examination reveals deep rutting of vehicle tracks at R indicating areas of soft ground. The small patch of peat at P, and the evidence of minor landslipping at L, cannot be seen clearly on the smaller-scale photography. The large-scale photography facilitates a closer examination of the morphology of the sides of the minor valley and hence allows a more accurate delineation of areas of bedrock, till and alluvium.

The interpretation described above is typical of what may be achieved from an examination of the photography with little background knowledge of the project area. Carter (1983) carried out a detailed air-photo interpretation of the dam site using the 1:12000 and 1:3000 black-and-white vertical photography together with coloured oblique photography. Carter's reference level had been considerably extended through studying all the available information about the project area and site inspection. Examples of Carter's interpretation are shown in Figs 3.26 and 3.27. The object of this detailed study was to find the best layout for a subsurface investigation which would establish more thoroughly than earlier investigations the stratigraphy and nature of the materials comprising the foundations of the dam and in particular to locate areas in which permeable materials might exist. The layout of earlier subsurface investigations which had concentrated on the centre lines of the envisaged

engineering structures failed to reveal the complex heterogeneous nature of the glacial deposits. In the later investigations, more emphasis was placed on the geology rather than the position of engineering structures, hence the need for a detailed air-photo interpretation. Individual drillholes were laid out on a framework inferred from the geological characteristics of the materials that were expected to be associated with the glacial landform features identified from the air-photo interpretation. The drillholes and test trenches therefore served not only to examine areas considered to be of direct engineering significance, but also to ensure that all the various landform features identified from the aerial photography were investigated in sufficient detail to attempt a reconstruction of the three-dimensional glacial stratigraphy of the dam foundations. Carter found this approach most useful in unravelling the complex nature of the glacial deposits which greatly aided the design of the dam foundations.

Geotechnical hazards

Despite the skill and experience required to make satisfactory interpretations of the geology of a site, geotechnical engineers and engineering geologists should still find the examination of air photographs a rewarding experience. The authors received little formal training in air-photo interpretation, yet have found them to be one of the most useful sources of information available during desk studies. In the following section we give details of our approach. The fundamental point to be made is, however, that almost anyone, however inexperienced, can gain useful information from air photographs.

Air photographs can provide very good quality information on:

- the presence of archaeological sites;
- site history;
- pre-existing vegetation;
- made ground;
- slope instability;dissolution features; and
- cambering and valley bulging.

Some of these features (for example, pre-existing vegetation, and the presence of made ground, slope instability and solution features) may be extremely difficult, if not impossible, to detect in any other way.

Archaeological features

The most efficient method of recognizing sites of archaeological interest is by the use of aerial photographs. The value of air photographs in archaeological research was first recognized during the period between World War I and World War II by Major Allen, who formed a pioneer collection of air photographs of the Oxford region, and Dr Crawford, who was made the first Archaeology Officer of the Ordnance Survey (St Joseph 1977). A brief account of the early history of application of air photography to archaeology is given by Crawford and Keiller (1928) and St Joseph (1957).

Archaeological features show up on aerial photographs as crop marks, subdued topography and tonal contrasts. For these reasons, the air photograph requirements of the archaeologist are quite specialized. The time and season of photography together with crop types are critical in many cases if archaeological features are to be identified. Ancient trenches, pits, post holes and foundations produce localized variations in the surface layers of soil. Such variations are often reflected in the differential growth or ripening of crops which are only visible during certain seasons.

As mentioned earlier, crop marks are best seen if extreme conditions, such as a period of drought, precede the photography of the crops. The period between May and July generally yields the best results. The crops most sensitive to local variations in soil type are cereals such as wheat and barley (St Joseph 1977). Hay and meadow grass are frequently unresponsive to such variations.

In order to enhance crop marks and subdued topography associated with foundations and ancient agricultural patterns, conditions of clear sunlight and long shadows are required. Such conditions occur only during early morning and late afternoon. Since minimum shadow is normally a fundamental requirement for conventional vertical air photography, photography for archaeological purposes is often specially commissioned. In many cases the best results are produced using oblique aerial photography. Figures 3.28 and 3.29 illustrate the use of shadow in detecting archaeological features.

Figure 3.28 shows an example of an ancient enclosure on Marlborough Down. The rectangular or trapezoidal areas are separated by small ditches. It is these ditches that give rise to the shadows which help define the pattern of this enclosure. Such features are also characteristic of deserted medieval villages. A later development is seen in the centre of the photograph in the form of an irregular circle made by a ditch superimposed on the older system of ditches.

Figure 3.29 shows an excellent example of ridge and furrow. The low sun angle allows each ridge and furrow to be picked out clearly. When seen at ground level, these features are often mistaken for some form of land drainage system, whereas an aerial view shows clearly that this is not the case. Ridge and furrow is the characteristic pattern associated with the medieval 'open field' system of farming (Beresford and St Joseph 1979). The photograph illustrates how the modem field system is superimposed on the medieval 'open fields'. The presence of ridge and furrow indicates the close proximity of a medieval settlement. Many towns and villages in Britain have grown from such settlements. There are however numerous deserted medieval villages in this country (Beresford and Hurst 1971).

Figure 3.30 shows an example of an early Iron Age fort (Maiden Bower, Hertfordshire). The characteristic shape of such a fortress is circular. A ditch bounded by a ridge made with the material taken from the ditch forms the perimeter of this circle. In the photograph, the ridge is marked by trees and bushes. It will be seen from the photograph that this fort has been in danger of being destroyed by the adjacent chalk quarry. Modern chalk and gravel workings are responsible for the total destruction of a great number of ancient sites such as this. The road in the foreground of the photograph is part of the A5 from London to Holyhead. It follows the line of the Roman Watling Street, but the cutting and embankment at the top of this photograph were made by Thomas Telford.

Pre-existing vegetation

The rapid increase in insurance claims for 'subsidence damage' to low-rise buildings, principally housing, in the UK has emphasized the need for a careful appraisal of vegetation that may have existed prior to development, where construction is to take place on shrinkable clay. The removal of trees or large shrubs leads to swelling of the clay, and observations have shown that this swelling can go on for several decades. This means that even though a construction site may be clear of vegetation at the time of purchase, the removal of trees or hedges some time before may still present a threat. Air photographs can provide a valuable source of information on the vegetation existing on a site over a long period before construction.

Case study - Houses damaged by heave, following the removal of trees

Figure 3.31 shows sections of three air photographs, taken over a period of years, of an area of Basildon in Essex. They show clearly the development of housing in the area. In Fig. 3.31a, the site is undeveloped and can be seen to be partially wooded. Figure 3.31b shows the construction of houses in progress. Figure 3.31c shows the houses complete. The geological map of the area shows that the site is underlain by London clay, which is known to be a shrinkable clay. Following construction, some of the houses in these terraces began to crack, and investigations showed that parts of the structure were heaving. The air photographs provide not only a permanent record of construction but, more importantly in this case, permanent records of the size and position of trees and other vegetation which may have been removed some years before development was envisaged. In this case, it can be seen that one part of the site was densely wooded whilst another appears to have been grassland. It would be reasonable to expect large differential movements and damage to occur where long structures cross from one area to the other, as a result of differential heave due to tree removal. This did, in fact, occur on this site.

Made ground

It is important to locate any areas of made ground during the site investigation. Made ground may be the result of backfilling trenches used for buried services, backfilling old quarries, brick pits or gravel pits, or the disposal of waste materials. Any disturbed ground can usually be recognized on aerial photographs by variations in tone with the surrounding undisturbed ground. Buried services, such as gas mains, appear as linear features which generally"have a light tone. Backfilled quarries and pits can be difficult to detect unless the existence of the quarry or pit is known beforehand. This clearly illustrates the importance of examining topographic maps and other data in conjunction with the photographs.

Case study - Infilled World War II anti-tank ditch

Figures 3.32 and 3.33 show two air photographs of an area to the south of Basingstoke. Figure 3.32 was taken in 1963. The 1 in. to 1 mile geological map of the area indicates that the site is probably underlain by a thin layer of 'clay with flints', a Head material, below which lies chalk to great depth. The site (marked' A') was developed in the late 1960s as a small housing estate. Subsequently, brickwork in one house cracked in such a pattern that it was obviously caused by downward foundation movement of part of the house. In order to undertake remedial works (underpinning was proposed), the precise cause of the damage had to be established: air photograph cover was sought. In particular, there was concern that the house might have been positioned over a solution feature in the chalk, which might have been infilled with loose or soft soil, or perhaps might contain a void.

The 1963 air photograph shows that solution features are present in the area: see the feature marked 'S' as a typical example of the expression of an infilled 'pipe' in the chalk. None of these features are to be seen, however, at the position of the house. A light-toned linear feature was also seen (indicated by arrows) and measurements from the photograph indicated that the house lay partly over this feature. This feature is clearly quite recent because there is evidence on the photograph that trees and hedgerows have been removed along its line - marked B. Initially it was feared that it might have been the trace of a large service pipe, but investigations of local Electricity Board service plans gave the clue that this was, in fact, an infilled World War II anti-tank trench. It was probably excavated in the summer of 1940 and backfilled in 1945 or 1946. Subsequent searches produced Fig. 3.33, which was taken by the Royal Air Force in 1947, and shows the trace of the trench to be quite fresh (Fig. 3.33).

Further investigations, and discussions with military historians and retired army personnel, produced an estimate of the probable shape of this defensive work, as built. Figure 3.34a shows this estimated cross-section. Subsequently it became possible to excavate an exploratory trench across the position shown on the air photographs. Figure 3.34b shows the profile of the anti-tank trench, which, as expected, contained uncompacted fill. There are virtually no records to indicate the position of these infilled trenches. Yet, had the air photographs been examined at the time that the layout of the estate was being planned, it seems likely that the problem could have been avoided.

Slope instability

In the UK, any site in clay with a slope of more than about 5° may be potentially unstable, and contain pre-existing shear surfaces as a result of slope movements that occurred under periglacial conditions, at the end of the last ice age. Excavation at the toe, or loading at the top of such slopes will lead to large-scale instability, often involving hundreds of thousands of cubic metres of ground. Cases where construction projects have been seriously disrupted by the reactivation of pre-existing slope instability are relatively commonplace, and therefore the examination of air photographs for signs of instability must be a major activity in many site investigations.

In many other geologically younger and tectonically more active areas of the world pre-existing instability occurs in a much wider range of materials, including granular soils and rocks.

Whilst the presence and extent of instability will not always be determinable from air photographs, experience has shown that they provide one of the best sources of information available to the geotechnical engineer. It is unlikely that boreholes can be used to find the very thin shear surfaces which typically lie at the base of a landslip mass, and geophysical methods are not normally of practical use. Where, as is often the case, the shear surface lies within a few metres of ground surface, trial pitting (see Chapter 5) provides the only other satisfactory method, in most instances.

The use of oblique air photographs to map the extent and detail of the Stag Hill landslip (Guildford, Surrey) has already been described above. In the case records below we discuss the use of vertical air photography.

Case study - Solifluction lobes near Sevenoaks, Kent

The construction of the Sevenoaks bypass faced problems with earthwork failures. Subsequent investigation (Weeks 1970) revealed that the route crossed several solifluction lobes which are in a state of limiting equilibrium. These solifluction lobes are underlain by an extensive solifluction sheet which contains several principal slip surfaces located at the base of the sheet (Weeks 1969, 1970). The lobes appear to be more unstable than the underlying sheet.

The geology of the area is shown in Fig. 3.35. The Hythe beds which comprise ragstone (sandy limestone), hassock (calcareous limestone) and sandy silty clay layers form a steep escarpment. The Hythe beds are underlain by the Atherfield clay and the Weald clay, both of which outcrop at the base of the escarpment. The slope below the escarpment is between 3° and 10° which is characteristic of the underlying weak rocks. The lobes consist of blocks of sandstone and limestone from the Hythe beds in a matrix of sandy silty clay, and have moved down a7° slope for about 400m. Intermittent movement of the front ('nose' or 'toe') of these lobes still occurs during periods of high water table.

The print laydown (Fig. 3.36) shows the area where the instability occurred during the construction of the bypass. Most of the lobes are clearly visible, having a typically lobate shape, and 'turbulent' texture. The 'turbulent' texture is produced by the hummocky ground and poor drainage which is characteristic of such landslip features. The sides and 'nose' of each lobe are characterized by steep slopes (Fig. 3.37) which are clearly seen when the photographs are viewed stereoscopically. Within the area covered by the print laydown there is a total of five lobes. The position and extent of each lobe are shown in Fig. 3.35. Lobes E, F, G and H are easily identified. The 'noses' of lobes F and G are marked by a line of small scars and bushes which allow their extent to be defined.

Lobe H is most noticeable since the field in which it is situated is covered by a series of light toned lineations which are disrupted by the lobe and help enhance its characteristic 'turbulent' texture. Lobe 0 is not easily identified because the field boundaries follow the edges of the lobe to a large extent. The upper part of the eastern edge of the lobe occurs downslope of the field boundary and is easily identified by a change in slope and characteristic texture. Recent movement of lobe E is evidenced by the displacement of the field boundary on the western edge of the lobe. The road which runs North - South across the eastern edge of the print laydown crosses the top of lobe H, but there appears to be no evidence of movement.

From ground level the lobes appear as areas of hummocky ground (Fig. 3.37). The change in slope which marks the edge of the lobes (particularly the 'nose' of each lobe) is a most noticeable feature at ground level. However, it would be impossible to appreciate fully the extent and shape of these features simply from a walk-over survey.

Aerial photography has the advantage of providing the necessary overview which allows the rapid assessment of ground features such as these.

The photographs which were used to make the print laydown in Fig. 3.36 were taken during August 1961 at mid-day and at a contact scale of about 1:2500. The contact scale ,of the print laydown is about 1:7000 as it has been photographically reduced. Norman et at. (1975) is an extensive study of the use of air photographs in the detection of landslip features in the Sevenoaks area found that detection is improved when the sun is low (Table 3.8). The scale of the photographs is an important factor in the detection of these and other landslip features. The solifluction lobes were clearly visible on the original aerial photographs and still clear on the photographically reduced print laydown, despite the unavoidable loss of quality.

Figure 3.38 shows, however, that the number of features which can be detected is proportional to the scale of photography. Norman et al. (1975) examined a range of film types and found that excluding economic considerations, the best photographs for studying these landslip features were those produced from infra-red colour film, using infra-red colour prints and black-and-white prints from the red and infra-red part of the spectrum from this film.

More information on the form and extent of the solifluction lobes could be obtained from a series of high oblique aerial photographs taken looking towards the escarpment. These photographs would only provide qualitative data, but would nevertheless be valuable.

Table 3.8 Influence of time of photography on the detectability of landslips (after Norman et al. (1975))

Time of day	Hours from noon	Percentage detection of landslip features	Number of features covered by sortie
10:00	2	41	65
14:00	2	55	109

15:00	3	68	25
15:00	3	80	10
8:00	4	83	119

The photo scale was 1:10 000 in each case.

Figure 3.38 shows, however, that the number of features which can be detected is proportional to the scale of photography. Norman et al. (1975) examined a range of film types and found that excluding economic considerations, the best photographs for studying these landslip features were those produced from infra-red colour film, using infra-red colour prints and black-and-white prints from the red and infra-red part of the spectrum from this film.

More information on the form and extent of the solifluction lobes could be obtained from a series of high oblique aerial photographs taken looking towards the escarpment. These photographs would only provide qualitative data, but would nevertheless be valuable.

Dissolution features

Cylindrical pipes or inverted cone-shaped sink holes are common features in soluble rocks such as limestone and chalk. These features are produced by localized solution weathering of the rock surface by acidic (CO_2 rich) water. Sink holes associated with the chalk are particularly common in the south of England. They tend, however, to be particularly common in areas where chalk is present beneath a thin cover of younger Tertiary sediments or superficial deposits. The formation of these features is possibly related in part to periglacial conditions (Higginbottom and Fookes 1970). Higginbottom (1971) has pointed out that a concentrated inflow of near freezing water is a much more effective carbonate solvent than water at normal temperature. The pipes and sink holes in chalk are usually filled with material derived from the over-lying deposits. This infilling is often loose and may contain weakly bridged cavities (Higginbottom 1965), and hence it is generally weaker and more compressible than the chalk host rock. Clearly if such features are not detected they can present a potential hazard to any structure founded over them. Thus the detection of sink holes and pipes is an important part of site investigations on chalk or limestone. Detection by direct method of sub-surface exploration alone is difficult and in most cases not cost effective. In some cases, ground-based geophysical methods are used, but by far the most cost effective indirect method is the use of aerial photography at the preliminary desk study stage. Potential targets may be identified on air photographs and proved by drilling. Air photography may also be used to plan ground-based geophysical surveys if drilling each feature does not appear to be cost effective.

In some cases sink holes and pipes are associated with ground surface depressions which are easily identified on air photographs. Figure 3.39 shows an example of these surface features. The area shown in the photograph is south of Stokenchurch. Buckinghamshire. The geology of this area is Upper Chalk overlain by clay-with-flints. The light tones at A and B are indicative of chalk close to the ground surface. When viewed stereoscopically, it is found that these features are associated with two large depressions in the ground surface. Such depressions typically mark the location of sink holes. The chalk on the eastern side of these features is covered by clay-with-flints, hence the absence of light tones. A smaller depression is seen at C within the clay-with-flints.

The circular feature, A, in Fig. 3.16 appears as a large depression when viewed stereoscopically. The centre of the depression is hidden by trees. The fact that natural vegetation has been allowed to flourish in the middle of a field is highly suspicious. Features such as this occurring over chalk suggest the presence of sink holes.

The light-toned circular feature at S in Fig. 3.33 marks the position of a slight surface depression. Such features are produced by the subsidence of material into sink holes caused

by solution of the underlying chalk. These features can cause serious foundation problems if undetected during site investigation. The M3 motorway now passes over the feature shown on the photograph. A comparison of Fig. 3.33 with Fig. 3.32 shows how the season of photography can affect the detection of these features. The feature is most clearly visible on the photograph taken during spring (Fig. 3.33). Although still visible it is less clear on the photograph taken during summer (Fig. 3.32).

Figure 3.40 shows the structure of a typical sink hole in chalk. It will be seen from this diagram that these features can extend to some depth. Tonal patterns produced by variations in soil type or vegetation are often useful in locating sink holes and pipes which are not associated with ground surface depressions. This type of feature is often difficult to locate and detectability is dependent to a large extent on the season of photography. The use of multi-band photography or remote sensing techniques such as aerial thermography or multi-spectral scanning (described by Beaumont (1979) and Lillesand and Kiefer (1979» or ground-based geophysics may be employed if conventional panchromatic or colour air photography fails to provide any useful data.

Sink holes and pipes are common features in the karst regions of the UK. Figure 3.41 shows a typical example of sink holes in Carboniferous limestone. It will be seen on this photograph that there is an abundance of these features over most of the area covered but their presence ceases abruptly along the line A-A. This marks the boundary of the limestone and a stratum of shale. The presence of sink holes is a useful aid to the field geologist and photogeologist in mapping limestone units. A well-developed surface drainage system is seen over the shale in contrast with that over the limestone, where most of the surface water finds its way underground via the sink holes. Streams can be seen disappearing underground in the bottom of the photograph. Most of the sink holes in the photograph appear as circular depressions in the boulder clay which covers the limestone.

SATELLITE REMOTE SENSING

Photographic remote sensing (described above, and conventionally termed 'air photography') is by far the most common form of imagery used during site investigations. But for large and remote sites satellite imagery can be of considerable use.

Whereas photographic systems typically are used to produced images from a limited number of wavelengths of electro-magnetic radiation, multi-spectral scanners (MSS) allow the detection simultaneously of both reflected and emitted radiation in several spectral bands. The operation of a typical MSS system is illustrated in Fig. 3.42.

Points on the Earth's surface are scanned in a raster fashion, normally by means of a spinning prism or oscillating mirror, in the case of optical/mechanical systems (in some landsat satellites), or by means of a fixed linear array of sensors (in Spot satellites) (Fig. 3.43). The Earth's surface is therefore sequentially sampled for radiation in discrete areas, and these later appear on MSS images as pixels (picture elements). The size of the pixels on the Earth's surface are an obvious limit to the resolution of such systems. The Landsat Satellite System was initiated by NASA in conjunction with the US Department of the Interior in 1967. Present generation Landsat systems offer spatial resolutions of about 30 m. The SPOT system (Systeme Probatoire d'Observation de la Terre) was initiated by the launch of the first Spot satellite in 1985, becoming operational in May 1986. Spot was originated by France's National Space Studies Centre (CNES), acting as an agency for its Ministry for Research and Space. Spot-3 is under assembly at the time of writing (1993), and will provide a spatial resolution of 10 m in panchromatic, or 20 m in colour. Much better spatial resolutions (of the order of 1 or 2 m) can be obtained when MSS systems are mounted in aircraft.

Once the radiation for a particular pixel is captured, it is dispersed into its various spectral components by means of a prism or diffraction grating system. This splits the incoming radiation into a series of spectral channels, or 'bands'. An array of electronic detectors, placed at an appropriate position behind the grating, detects the strength of radiation within the wavelength region to which it is sensitive. The amplified signal is then digitized, and either recorded (in the case of aircraft-based systems) or transmitted to a ground satellite-receiver station.

For Spot satellites each image covers some 3600 km2. Individual digital images cost between FF7000 and FF19000 at the time of writing, and must then be processed on computer to produce the desired end-product. Satellite image processing is a complex task, beyond the scope of the book. A concise guide to satellite imagery and the available processing techniques is given in Kennie and Matthews (1985). The application of satellite remote sensing images in site investigations has, to date, been limited by considerations of cost (both of the digital data and subsequent processing) and spatial resolution. In remote areas, however, satellite imagery may be a most important source of information for preliminary studies (for example in selecting highway routes), for environmental and water resource engineering, and in the search for construction materials. The reader is referred to Kennie and Matthews (1985) for further information.

THE WALK-OVER SURVEY

The walk-over survey involves an inspection of the site and surrounding area on foot, the examination of local records concerning the site, and the questioning of local inhabitants about the site. The object of this exercise is to confirm, amplify and supplement the information collec.ted during earlier stages of the site investigation (Dumbleton and West 1976a). It is essential that all the information concerning the site is studied thoroughly before carrying out a walk-over survey. This will allow a greater understanding of the significance of features seen on and around the site and enable more effective research of local records.
In general, walk-over surveys may be divided into two operations:

- site inspection; and
- local enquiries.

The site inspection involves a thorough visual examination of the site and its environs making full use of maps (topographic and geological), site plans, and air photographs. Before carrying out the site inspection, permission to gain access to the site must be obtained from both the owner and occupier. It is important when inspecting a site to be suitably equipped. The list given below (based on that given by Dumbleton and West (1976a» gives some guidance on the necessary equipment for this operation:

notebook, pencil, measuring tape, compass, clinometer, camera, binoculars, Abney level, topographic and geological maps, site plans, preliminary geotechnical map, air photographs, pocket stereoscope, 'chinagraph' pencil, wooden pegs, portable hand auger, Mackintosh prospecting tool, geological hammer, trenching tool, polythene bags, ties, labels, waterproof marker pen, penknife, hand lens, dilute hydrochloric acid, plumb line.

Many features may be observed during a site inspection. Only with experience can the relative importance and significance of these features be interpreted in the field. The checklist given below details those features which should be inspected and noted.

(A) GEOMORPHOLOGY

1. General features. Note slope angles, types of slope (convex or concave) and sudden changes in slope angle. This information can give a guide to geology. For example, hard rocks resistant to erosion often form steep scarp slopes.

2. Glacial features. Note the presence of mounds and hummocks in more-or-less flat country. These features are often associated with glacial deposits such as till and glacial sand and gravel. Glacial landforms such as 'U'-shaped valleys and overflow channels should be noted where recognized.
3. Mass movement. The presence of hummocky broken or terraced ground on hill slopes should be noted since these features are normally associated with land-slipping. Landslip areas present a potential engineering hazard and therefore must be inspected thoroughly. The extent and type of landslip should be noted on the site plan. The air photographs will aid the classification and mapping of landslip areas in the field. The relative age of the landslip should be noted where possible. In the case of rotational landslips the amount of degradation of the rear scarp may give an indication of relative age. Structures situated on or adjacent to a landslip should be inspected for structural damage. The alignment of fences and hedges crossing these features should be compared with maps and air photographs for evidence of recent movement. The positions of tension cracks and ponds associated with the landslip should be noted on the site plan. The presence of small steps in hill slopes and inclined tree trunks are indications of soil creep and hence should be noted. Evidence of soil creep is particularly noticeable where a hedge or other barrier traps the creep material on the upslope side.

(B) SOLID AND DRIFT GEOLOGY

1. Exposures. Exposures of rock or soil may be found in cliffs, stream and river beds, quarries, pits and cuttings. The material seen in such exposures should be described in the manner outlined in Chapter 2. Samples of soil may be taken for moisture contents and index tests. Note should be made of discontinuities (fissures, joints and bedding planes), seen in rock or soil exposures. This should include the type of discontinuities, the dip and dip direction of major discontinuities, the average spacing and persistence of major discontinuities and the nature of the major discontinuities (for example, open, closed, or infilled). If necessary a detailed rock mass description can be carried out during a later stage of the site investigation.
2. Solid geology. Exposures, geomorphological features, land use and vegetational changes may be used in conjunction with the geological map of the site and air photographs to establish the solid geology where it is not hidden by superficial deposits. An elementary treatment of the techniques used in mapping solid geology is given by Himus and Sweeting (1968).
3. Superficial geology. Superficial deposits are often the cause of geotechnical problems and hence require attention during the early stages of the site investigation. The main types of deposit associated with the site and surrounding area will be known already from the geological map. The extent of such deposits may have been tentatively mapped using air photographs. The site visit serves to check the geological map and any air-photo interpretation, and make a close examination of areas of poor ground. Exposures, geomorphological features, land use and vegetation may be used to aid the location of superficial deposits. For example the flat ground associated with rivers or streams normally marks the extent of alluvium; areas of reeds, rushes and willow trees often indicate wet ground conditions which may be associated with areas of peat. Where it is necessary to sample or prove the extent of certain superficial deposits such as peat, then the Mackintosh prospecting tool or a hand auger may be used.
4. Construction materials. The cost of transporting bulk construction materials, such as sand and aggregates can be prohibitive if the distance between the source and the site is great. It is therefore important to identify potential or existing local sources of such materials. Superficial deposits, such as plateau gravel, river terrace gravels, glacial sands and gravels are often excellent sources of sand and aggregate.

(C) GROUNDWATER CONDITIONS

The presence of springs and seepages should be noted on the site plans. Features observed on air photographs which have been interpreted as springs or areas of wet ground should be inspected. Vegetational features such as unusual green patches, reeds, rushes and willow trees can aid the identification of wet ground conditions. Special note should be made of any evidence of seepage erosion. Shallow wells (not generally included in the BGS collection of records) located on or near the site may give a reasonable indication of groundwater levels.

(D) SURFACE WATER AND EROSION

Note should be made of ponds and streams which are not shown on the maps of the site. The likelihood of flooding should be assessed for the site and surrounding area. Any history of flooding in the area may be obtained through local enquiries. Any evidence of active soil erosion by surface water, such as gullies, should be noted.

(E) SITE ACCESS

Site access for drilling equipment and other site vehicles should be assessed, making full use of air photographs in the field. It should be pointed out that different types of drilling rig require different types of access. The shell and auger drilling rigs is perhaps the most versatile in terms of access requirements. In situations where access is limited, such as inside a building, the shell and auger rig can be dismantled outside and reassembled over the borehole location. Furthermore, in situations where access is made difficult because of steep gradients of poor trafficability, this type of rig can be winched into position. Lorry-mounted drilling rigs cannot usually be driven up steep gradients in off-the-road situations and access is further limited by the large turning circle required by these vehicles. The access of small track-mounted rigs is not as restricted by trafficability and manoeuvrability as that of lorry-mounted rigs.

In view of the problems of access in terms of time and cost of getting the drilling equipment into position, care should be taken to site boreholes for easy access wherever possible. For example, siting boreholes over ponds should be avoided unless absolutely necessary. The siting of boreholes directly beneath overhead cables should be avoided as this can prove dangerous, particularly if they are high voltage cables. Where boreholes are sited beneath electricity cables, the local electricity board should be informed of the times when drilling is to take place since it is possible to have the electricity turned off in these cables.

All entrances (for example, gates and openings in walls or hedges) should be photographed. Such photographs are often useful if any outrageous claims are made by local farmers and landowners for damage done by vehicles during the subsequent stages of the site investigation.

(F) DAMAGE OF EXISTING STRUCTURES

Structures situated on or near the site should be inspected for damage. The pattern and extent of cracks in damaged structures should be noted. Measurements of cracks may be made and the damage classified using a system such as that outlined in Table 3.9. Other signs of distortion, such as non-verticality of walls should be noted. Photographs of the damage should be taken for future reference, or as a defence against any claims that construction has caused damage to existing structures adjacent to the site. Photographs, however, are unlikely to show signs of slight damage very clearly. It should be pointed out that cracks and other signs of distortion of a structure may represent an accumulation of many movements over a long period of time and hence cannot always be interpreted meaningfully without studying the history of damage, and in particular a record of cracking. The type of foundation and the foundation material are also important considerations when interpreting damage to existing

structures. Information of foundations for small structures may be obtained from local builders.

Table 3.9 Classification of visible damage to walls with particular reference to ease of repair of plaster and brickwork or masonry (after Burland et al. (1977))

Degree of damage	Description of typical damage* (ease of repair is in italics)	Approximate crack width (mm)
1 Very slight	*Fine cracks which can easily be treated during normal decoration.* Perhaps isolated slight fracturing in building. Cracks in external brickwork visible on close inspection	< 0.1 †
2 Slight	*Cracks which are easily filled. Redecoration probably required.* Several slight fractures showing inside of building. Cracks are visible externally and some repointing may be required externally to ensure weathertightness. Doors and windows may stick slightly.	< 5.0 †
3 Moderate	*The cracks require some opening up and can be patched by a mason.* Recurrent cracks can be masked by suitable linings. Repointing of external brickwork and possibly a small amount of brickwork to be replaced. Doors and windows sticking. Service pipes may fracture. Weathertightness often impaired.	5 – 15 † or a number of cracks »3.0
4 Severe	*Extensive repair work involving breaking-out and replacing sections of walls, especially over doors and windows.* Window and door frames distorted, floor sloping noticeably. Walls leaning or bulging noticeably, some loss of bearing in beams. Service pipes disrupted.	15 – 25 † but also depends on number of cracks
5 Very severe	This requires a major repair job involving partial or complete rebuilding. Beams lose bearings, walls lean badly and require shoring. Windows broken with distortion. Danger of instability.	> 25 † but depends on number of cracks

Hairline cracks of less than about 0.1 mm width are classed as negligible

* It must be emphasized that in assessing the degree of damage account must be taken of the location in the building or structure in which it occurs.

† Crack width is one factor in assessing degree of damage and should not be used on its own as a direct measure of damage.

The common causes of structural damage to buildings include: clay heave or shrinkage; excessive differential consolidation settlement; settlement due to made ground; slope instability; groundwater lowering; soil erosion; structural failure of foundations; subsidence due to mining or sink holes; vibration; and chemical attack.

Local enquiries involve the acquisition of local knowledge concerning the site. Such enquiries include the following:

1. Local builders and civil engineering contractors. Information on the types of foundations used in the area together with any construction problems associated with the ground may be obtained from local builders and civil engineering contractors.
2. Local authority engineers and surveyors. Information on flood levels, general ground conditions in the area, and previous uses of the site may be obtained from the local authority engineer's and surveyor's offices. Also further information on existing damage to structures can sometimes be obtained from these sources.
3. Local statutory undertakers. These include: the local electricity utility company, the distributor (e.g., National Grid pIc in the UK) and the central electricity generator; gas suppliers; telecommunications companies (e.g. British Telecom); water utilities; and sewerage and waste-water organizations. Information on the location of services may be obtained from these sources, in order to avoid damage to underground pipes, ducts, and cables during drilling. On occasion, the frequent need for maintenance of pipes may give a clue to the existence of ground movements.
4. Local archives. Old maps held in local archives may provide information on areas of fill or previous works on the site. Records of flooding, landslipping and mining activity may be found in local archives.
5. Local inhabitants. Local inhabitants who have lived in the area for some time are often a useful source of information concerning previous uses of the site, structural damage to buildings on or near site, mining subsidence, the location of old mine shafts, flooding and landslipping. In rural areas information on drainage, landslipping, subsidence and trafficability may be obtained from local farmers. A certain amount of caution should be employed when assessing any information given by local inhabitants as it is sometimes exaggerated, vague or ambiguous.
6. Local clubs and societies. Local clubs and societies can often provide valuable information concerning the site and surrounding area. Such clubs and societies include: archaeology societies; industrial archaeology societies; local history societies; caving clubs; geological societies and natural history societies.
7. Schools, colleges and universities. Local educational establishments, particularly colleges and universities are often a valuable source of local information. Many colleges and universities have Departments of Geology, Geography and Civil Engineering (with geotechnical expertise). It is likely that some of these departments have carried out detailed studies of various local areas at some time. The information from such studies can be most valuable, particularly if the study area includes the site. Unpublished geological records are available for a wide variety of areas from University Geology Department libraries.

Chapter 4

Subsurface exploration: engineering geophysics

INTRODUCTION

The most widespread site investigation techniques, such as those described in Chapters 5, 7, 8 and 9, involve the drilling of holes in the ground, sampling at discrete points, and in situ or laboratory testing. Given the relatively small sums of money involved in ground investigations of this type, only a very small proportion of the volume of soil and rock that will affect construction can be sampled and tested.

Geophysical techniques offer the chance to overcome some of the problems inherent in more conventional ground investigation techniques. Many methods exist with the potential of providing profiles and sections, so that (for example) the ground between boreholes can be checked to see whether ground conditions at the boreholes are representative of those elsewhere. Geophysical techniques also exist which can be of help in locating cavities, backfilled mineshafts, and dissolution features in carbonate rocks, and there are other techniques which can be extremely useful in determining the stiffness properties of the ground.

Yet, at the time of writing, geophysics is only rarely used in ground investigations. Various reasons have been put forward to explain this fact, including:

1. poor planning of geophysical surveys (see Chapter 1), by engineers ignorant of the techniques; and
2. overoptimism by geophysicists, leading to a poor reputation for the available techniques.

Most geophysics carried out during ground investigations is controlled by geologists or physicists. Generally, their educational background is either a geology or physics first degree, with a Masters postgraduate degree in geophysics. Such people often have considerable expertise in the geophysical techniques they offer, but they have very little idea of the contractual constraints within which civil and construction engineers must work. The writers' view is that an understanding of geophysics is not beyond the capabilities of most civil engineers, who generally have a good education in physics. This chapter therefore provides an engineer's view of engineering geophysics. It does not provide the conventional views of engineering geophysics, but deliberately concentrates on identifying those situations where geophysical techniques are likely to be of most help to engineers.

Engineering needs

Most geophysical techniques have as their origin the oil and mining industries. In such industries the primary need of a developer is to identify the locations of minerals for exploitation, against a background of relatively large financial rewards once such deposits are found. Geophysics plays a vital role intermediate between geological interpretation of the ground and its structure, and the drilling of exploratory holes to confirm the presence of ores, oil or gas. In most cases the minerals are deep, and drilling is very expensive — geophysics

allows optimization of the drilling investigation, amongst other things. Many mineral explorations will be centred upon deep deposits, where ground conditions are spatially relatively uniform, and geological structures are large. Geophysical techniques are relatively cheap, and are highly regarded in such a speculative environment, even though they may not always be successful.

In contrast, ground investigations are carried out against a professional, contractual and legal background which often demands relatively fine resolution and certainty of result. The idea that a test or method will be successful and yield useful data only in a proportion of the cases in which it is used is unacceptable. As was noted in Chapter 1, it is necessary for a geotechnical engineer to weigh carefully the need for each element of his ground investigation. He will often have to persuade his client to provide additional funds for non-standard techniques, and if these are unsuccessful the client may not be convinced of his competence. Therefore, when using geophysical techniques during ground investigation, as part of the engineering design process, great care is needed. The engineer should convince himself of the need for such a survey, and should take care of ensure that only appropriate and properly designed surveys are carried out.

Unfortunately, there has been a widespread failure of geophysical techniques to perform as expected during ground investigations.

> So far as geophysical methods of subsoil exploration are concerned, there can be no doubt about their desirability and merits, because they are extremely cheap: they are even cheaper than geologists.
>
> They have only one disadvantage, and that is we never know in advance whether they are going to work or not!
>
> During my professional career, I have been intimately connected with seven geophysical surveys. In every case the physicists in charge of the exploration anticipated and promised satisfactory results. Yet only the first one was a success; the six others were rather dismal failures.
>
> Terzaghi, 1957

More recently, the view has emerged that it is the planning of the surveys which has been at fault, and that if proper geological advice is sought then all will be well.

> Too often in the past geophysical methods have been used without due reference to the geological situation, and consequently the results have been disappointing.
>
> These failures always appear to be blamed on the method rather than on the misapplication of the method, and this is why many engineers mistrust geophysical methods. The solution to this difficulty is to take better geological advice during the planning of the investigation, and to maintain close geological supervision during its execution.
>
> Burton, 1975

It is our belief that both of these attitudes are over-simplistic. Whilst it is true that most professionals will be overoptimistic in their attempts to gain work, experience of the application of certain techniques during ground investigation is now sufficient to give guidance on techniques which will have a much better than one in seven chance of success. On the other hand, there is a growing trend for geophysical techniques to be integrated into ground investigations in a way which is difficult for a geophysicist to understand — the geophysicist cannot readily appreciate the engineer's priorities and requirements, and may not be sufficiently familiar with the science of soil mechanics. In summary, the majority of problems arise because of:

1. the expectations of engineers that all techniques will be 100% successful;

2. poor inter-disciplinary understanding between engineers, engineering geologists and geophysicists;
3. a lack of communication, particularly with respect to the objectives of ground investigation;
4. a lack of objective appraisal of the previous success of geophysical techniques in the particular geological setting, given the objectives of the survey;
5. poor planning of the execution of the survey; and
6. the use of inappropriate science (for example, measurements of compressional wave velocities).

It is important to be clear as to the reason for using geophysics in ground investigations. In practice there are at least five different functions which can be fulfilled.

The variability of natural near-surface ground has already been noted, as has the limited finance available to make boreholes. Geophysical techniques can contribute very greatly to the process of ground investigation by allowing an assessment, in qualitative terms, of the *lateral variability* of the near-surface materials beneath a site. Non-contacting techniques such as ground conductivity, magnetometry, and gravity surveying are very useful, as are some surface techniques (for example, electrical resistivity traversing).

Geophysical techniques can also be used for *vertical profiling.* Here the objective is to determine the junctions between the different beds of soil or rock, in order either to correlate among boreholes or to infill between them. Techniques used for this purpose include electrical resistivity depth profiling, seismic methods, the surface wave technique, and geophysical borehole logging. Some are surface techniques, but the majority are carried out down-hole.

Sectioning is carried out to provide cross-sections of the ground, generally to give details of beds and layers. It is potentially useful when there are marked contrasts in the properties of the ground (as between the stiffness and strength of clay and rock), and the investigation is targeted at finding the position of a geometrically complex interface, or when there is a need to find hard inclusions or cavities. In addition, as with vertical profiling, these techniques can allow extrapolation of borehole data to areas of the site which have not been the subject of borehole investigation. Examples of such techniques are seismic tomography, ground probing radar, and seismic reflection.

One of the major needs of any ground investigation is the *classification* of the subsoil into groups with similar geotechnical characteristics. Geophysical techniques are not generally of great use in this respect, except in limited circumstances. An example occurs where there is a need to distinguish between cohesive and noncohesive soils. Provided that the salinity of the groundwater is low, it is normally possible to distinguish between these two groups of materials using either electrical resistivity or ground conductivity.

Finally, almost all geotechnical ground investigations aim to determine stiffness, strength, and other parameters in order to allow design calculations to be carried out. Traditionally, geotechnical engineers felt that the determination of geotechnical parameters from geophysical tests was impossible. The acceptance, within the last decade or so, that the small strain stiffnesses relevant to the design of civil engineering and building works may, in many circumstances be quite similar to the very small strain stiffness (Go or Gmax) that can be determined from seismic methods has led to a worldwide reawakening of interest in this type of method.

In selecting a particular geophysical technique for use on a given site, it is essential that the following questions are asked.

1. What is the objective of the survey?
   It is generally true that users of geophysics expect the survey to provide a number of types of information. In fact, the converse is true. The survey should normally be designed to provide information on a single aspect of the site. A number of examples are given below:
   - depth to rockhead;
   - position of old mineshafts;
   - corrosivity of the ground;
   - very small strain stiffness of the ground;
   - extent of saline intrusion of groundwater;
   - position of cohesive and granular deposits along a pipeline route.

2. What is the physical property to be measured?
   Given the 'target' of the geophysical investigation, the physical property that is to be measured may be obvious. For example, if the target is to be the determination of the very small strain stiffness of the ground, then it follows that the property that must be measured is the seismic shear wave velocity. But in many cases the success of a geophysical survey will depend upon the choice of the best geophysical method, and the best geophysical method is likely to be the one which is most sensitive to the variations in the ground properties associated with the target. For example, in trying to locate a mineshaft it might be relevant to consider whether metallic debris (for example, winding gear) has been left at the location (making magnetic methods likely to be successful) or whether the shaft is empty and relatively close to the ground (making gravity methods attractive).

3. Which method is most suited to the geometry of the target?
   Geophysical targets range from cavities to boundaries between rock types, and from measurements of stiffness to the location of geological marker beds. The particular geometry of the target may make one particular technique very favourable (for example, the determination of the position of rockhead (i.e. the junction between rock and the overlying soil) or gassy sediments beneath the sea is often carried out using seismic reflection techniques.

4. Is there previous published experience of the use of this method for this purpose?
   Unfortunately it is unusual for engineers and geophysicists to publish their failures, so that the reporting of the successful use of a particular technique for a given target cannot be taken as a guarantee that it will work in a given situation. Conversely, however, the lack of evidence of success in the past should act as a warning. In assessing the likely success of a geophysical method, it will be helpful to consult as widely as possible with specialists and academic researchers.

Some geophysical methods have a very high rate of success, provided that the work is carried out by experienced personnel. Others will have very little chance of success, however well the work is executed.

5. Is the site 'noisy'?
   Geophysical methods require the acquisition of data in the field, and that data may be overwhelmed by the presence of interference. The interference will be specific to the chosen geophysical method; seismic surveys may be rendered impractical by the ground-borne noise from nearby roads, or from construction plant; resistivity and conductivity surveys may be interfered with by electrical cables, and gravity surveys will need to be corrected for the effects of nearby buildings, known basements, and embankments.

6. Are there any records of the ground conditions available?
   If borehole or other records are available then two approaches are possible. The information can be used to refine the interpretation of the geophysical output, or the geophysical method can be tested blind. The latter method is really only suitable in instances where the geophysical method is claimed to work with great certainty, for example when testing a contractor's ability to detect voids. In most cases geophysicists will require a reasonable knowledge of the ground conditions in order to optimize the geophysical test method, and the withholding of available data will only jeopardize the success of a survey.

7. Is the sub-soil geometry sufficiently simple to allow interpretation?
   Some methods of interpretation rely on there being a relatively simple sub-soil geometry, and one which is sufficiently similar to simple physical models used in forward modelling, to provide 'master curves'. Most models will assume that there is no out-of-plane variability, and that the ground is layered, with each layer being isotropic and homogeneous. Complex three-dimensional structure cannot be interpreted.

8. Is the target too small or too deep to be detected?

9. Is it necessary to use more than one geophysical method for a given site?

Classification of geophysical techniques

There are many geophysical techniques available during ground investigation. In this section we attempt to classify them in different ways, to allow the reader to develop a framework within which to select the most appropriate technique(s) for his job.

Geophysical techniques may be categorized by the following:

Control of input

Geophysical methods may be divided into two groups.

1. *Passive techniques.* The anomalies measured by the technique pre-exist. They cannot be varied by the investigator. Repeat surveys can be carried out to investigate the effects of variations of background 'noise', but apart from varying the time of the survey, and the equipment used, no refinement is possible. In using passive techniques, the choice of the precise technique and the equipment to be used are very important. Generally these techniques involve measurements of local variations in the Earth's natural force fields (for example, gravity and magnetic fields).

2. *Active techniques.* These techniques measure perturbations created by an input, such as seismic energy or nuclear radiation. Signal-to-noise ratio can be improved by adding together the results of several surveys (stacking), or by altering the input geometry.

In general, interpretation is more positive for active than for passive techniques, but the cost of active techniques tends to be greater than for passive techniques.

Types of measurement

Some geophysical techniques detect the spatial difference in the properties of the ground. Such differences (for example, the difference between the density of the ground and that of a water-filled cavity) lead to perturbations of the background level of a particular measurement

(in this case, gravitational pull) which are measured, and must then be interpreted. These perturbations are termed 'anomalies'. Other geophysical techniques measure particular events (for example, seismic shear wave arrivals, as a function of time), and during interpretation these measurements are converted into properties (in this case, seismic shear wave velocity).

A particular geophysical technique will make measurements of only a single type. Techniques that are commonly available measure:

- seismic wave amplitude, as a function of time;
- electrical resistivity or conductivity;
- electromagnetic radiation;
- radioactive radiation;
- magnetic flux density; and
- gravitational pull.

From a geotechnical point of view, passive techniques require relatively little explanation. The apparatus associated with them can often be regarded as 'black boxes'. It is sufficient, for example, to note that:

1. gravity methods respond to differences in the mass of their surroundings, which results either from contrasts in the density of the ground, or from variations in geometry (cavities and voids, embankments, hills, etc.);
2. magnetic methods detect differences in the Earth's magnetic field, which are produced locally by the degree of the magnetic susceptibility (the degree to which a body can be magnetized) of the surroundings. Such methods will primarily detect the differences in the iron content of the ground, whether natural or artificial;
3. Natural gamma logging detects the very small background radiation emitted by certain layers in the ground.

Active methods require more consideration, because surveys using these methods can often be optimized if the principles of the methods are understood.

The *seismic method* is rapidly becoming more popular in geotechnical investigations because of its ability to give valuable information on the stiffness variations in the ground.

The seismic method relies on the differences in velocity of elastic or seismic waves through different geological or man-made materials. An elastic wave is generated in the ground by impactive force (a falling weight or hammer blow) or explosive charge. The resulting ground motion is detected at the surface by vibration detectors (geophones). Measurements of time intervals between the generation of the wave and its reception at the geophones enable the velocity of the elastic wave through different media in the ground to be determined.

A seismic disturbance in elastically homogenous ground, whether natural or artificially induced, will cause the propagation of four types of elastic wave, which travel at different velocities. These waves are as follows.

1. *Longitudinal waves ('P' waves).* These are propagated as spherical fronts from the source of the seismic disturbance. The motion of the ground is in the direction of propagation. These waves travel faster than any other type of wave generated by the seismic disturbance.
2. *Transverse or shear waves ('S' waves).* Transverse waves, like longitudinal waves, are propagated as spherical fronts. The ground motion, however, is perpendicular to the direction of propagation in this case. S waves have two degrees of freedom unlike P waves which only have one. In practice, the S wave motion is resolved into

components parallel and perpendicular to the surface of the ground, which are known respectively as SH and SV waves. The maximum velocity of an S wave is about 70% of the P wave velocity through the same medium.

3. *Rayleigh waves.* These waves travel only along the ground surface. The particle motion associated with these waves is elliptical (in the vertical plane). Rayleigh waves generally attenuate rapidly with distance. The velocity of these waves depends on wavelength and the thickness of the surface layer. In general, Rayleigh waves travel slower than P or S waves.
4. *Love waves.* These are surface waves which occur only when the surface layer has a low P wave velocity with respect to the underlying layer. The wave motion is horizontal and transverse. The velocity of these waves may be equal to the S wave velocity of the surface layer or the underlying layer depending on the wavelength of the Love wave. Energy sources used in seismic work do not generate Love waves to a significant degree. Love waves are therefore generally considered unimportant in seismic investigation.

Soils generally comprise two phases (the soil skeleton and its interstitial water) and may have three phases (soil, water and air). P-wave energy travels through both the skeleton and the pore fluid, whilst S-wave energy travels only through the skeleton, because the pore fluid has no shear resistance.

Traditionally, the geophysical industry has made almost exclusive use of P waves. These are easy to detect, since they are the first arrivals on the seismic record. However, in relative soft saturated near-surface sediments, such as are typically encountered in the temperate regions of the world, the P-wave velocity is dominated by the bulk modulus of the pore fluid. If the ground is saturated, and the skeleton relatively compressible (i.e. $B = 1$ (Skempton 1954)), the P-wave velocity will not be much different from that of water (about 1500 *m/s).* Therefore it is not possible to distinguish between different types of ground on the basis of P-wave velocities until the bulk modulus of the skeleton of the soil or rock is substantially greater than that of water. This is only the case for relatively unweathered and unfractured rocks, for which the P-wave velocity may rise to as much as 7000 m/s.

Shear wave energy travels at a speed which is determined primarily by the shear modulus of the soil or rock skeleton, modified by its state of fracturing:

$$V_s = \sqrt{\left(\frac{G_\circ}{\rho}\right)} \qquad (4.1)$$

where V, = shear wave velocity, G0 = shear modulus at very small strain and P = bulk density.

Since the bulk density of soil is not very variable, typically ranging from 1.6 Mg/rn3 for a soft soil to 3.0Mg/rn3 for a dense rock, the variation of S-wave velocity gives a good guide to the very small strain stiffness variations of the ground. Further, the last decade has brought a realization that (except at very small strain levels) soil does not behave in a linear-elastic manner, and that the strains around engineering works are typically very small. Thus it is now realized that stiffnesses obtained from geophysical methods may be acceptably close to those required for design. For rocks the operational stiffness may be very similar to that obtained from field geophysics, while for soils it is likely to be of the order of two or three times lower. Given the uncertainties of many methods available for determining ground stiffnesses, seismic geophysical methods of determining S-wave velocities are becoming increasingly important in geotechnical site investigations.

Whereas the determination of seismic shear wave velocity has gained increased prominence in site investigations, the use of seismic methods to determine the geometry of the sub-soil appears to have undergone a general decline. This is probably associated with a low level of success of these techniques in shallow investigations. The propagation of seismic waves through near-surface deposits is extremely complex. The particulate, layered and fractured nature of the ground means that waves undergo not only reflection and refraction but also diffraction, thus making modelling of seismic energy transmission impractical. Anisotropy, and complex and gradational soil boundaries often make interpretation impossible.

Electrical resistivity and conductivity methods rely on measuring subsurface variations of electrical current flow which are manifest by an increase or decrease in electrical potential between two electrodes. This is represented in terms of electrical resistivity which may be related to changes in rock or soil types. The electrical resistivity methods is commonly used therefore to map lateral and vertical changes in geological (or man-made) materials. The method may also used to:

1. assess the quality of rock/soil masses in engineering terms;
2. determine the depth to the water table (normally in arid or semi-arid areas);
3. map the saline/fresh water interface in coastal regions;
4. locate economic deposits of sand and gravel; and
5. locate buried features such as cavities, pipelines, clay-filled sink holes and buried channels.

The electrical resistivity of a material is defined as the resistance offered by a unit cube of that material to the flow of electrical current between two opposite faces. Most common rock-forming minerals are insulators, with the exception of metalliferous minerals which are usually good conductors. In general, therefore, rocks and soils conduct electricity by electrolytic conduction within the water contained in their pores, fissures, or joints. It follows that the conductivity of rocks and soils is largely dependent upon the amount of water present, the conductivity of the water, and the manner in which the water is distributed within the material (i.e., the porosity, degree of saturation, degree of cementation, and fracture state). These factors are related by Archie's empirical equation (Archie 1942):

$$\rho = a\rho_w n^m s^l \qquad (4.2)$$

where ρ = resistivity of the rock or soil, ρ_w = resistivity of the pore water, n = porosity, s degree of saturation, 1=2, m = 1.3-2.5, and a = 0.5-2.5.

The manner in which the water is distributed in the rock determines the factor m (cementation factor) which for loose (uncemented sands) is about 1.3 (Van Zijl 1978). The validity of Archie's equation is, however, dependent on various factors such as the presence or absence of clay minerals (Griffiths 1946). Guyod (1964) gives a simplified version of Archie's equation:

$$\rho = \frac{\rho_w}{n^2} \qquad (4.3)$$

where ρ = resistivity of the rock/soil, ρ_w = resistivity of the pore water, and n = porosity.

Because the conduction of electrical current through the pore water is essentially electrolytic, the conductivity of the pore water must be related to the amount and type of electrolyte within it. Figure 4.1 shows the relation between the salinity of the pore water and the measured resistivity for materials with different porosities. As the salinity of the pore water increases, there is a significant decrease in measured resistivity.

The above relationships suggest that the more porous (or the more fissured/jointed) the soil or rock is, the lower is its resistivity. Thus, in general, crystalline rocks such as igneous rocks which exhibit a low porosity, have a high resistivity compared with the more porous sedimentary rocks such as sandstones. Clay-bearing rocks and soils will tend to have lower resistivities than non clay-bearing rocks and soils. These generalizations are reflected in the typical resistivity values for different soil and rock types given in Table 4.1.

Table 4.1 Typical electrical resistivity values for different soil and rock types

Material	Resistivity (Ω m)
Clay *	3-30
Saturated organic clay or silt †	5-20
Sandy clay *	5-40
Saturated inorganic clay or silt †	10-50
Clayey sand *	30-100
Hard, partially saturated clays † and silts, saturated sands and gravels †	50-150
Shales, dry clays, silts †	100-500
Sand, gravel *	100-4000
Sandstone *	100-8000
Sandstones, dry sands and gravels †	200-1000
Crystalline rocks *	200-10000
Sound crystalline rocks †	1000-10000
Rocksalt, anhydrite *	>1100

* Values from Dohr (1975).
† Values from Sowers and Sowers (1970).

Degree of contact with the ground
The speed and cost of geophysical methods is strongly related to the amount of work necessary to set up the testing. Techniques (and therefore costs) vary very widely.

Non-contacting techniques are often relatively quick and easy to use. Examples are ground conductivity, magnetic and ground penetrating radar (GPR) techniques. Here the user carries the instrument across the site. Data are either collected automatically, on a time or distance basis, or upon demand (for example at predetermined positions, perhaps on a grid, across the site). These techniques are suitable for determining the variability of shallow soil or rock deposits, and can be economical ways of investigating large areas of ground in searches for particular hazards, such as mineshafts and dissolution features. Disturbance to the ground is minimal, and the equipment is often light enough to be carried by a single person.

Surface techniques are slower than non-contacting techniques, because sensor elements forming part of the geophysical measuring system must be attached to the ground before measurements can be made. Examples of such techniques are electrical resistivity, seismic refraction and reflection, and the surface wave technique. Although the method requires contact with the ground, it remains minimal and damage to the site will normally be negligible. Surface techniques are more expensive than non- contacting techniques, but can often allow a greater depth of investigation.

Down-hole techniques are generally the most expensive and time-consuming amongst the geophysical techniques. Sensors must be inserted down relatively deep preformed holes, or pushed to the required depth (perhaps 20m) using penetration testing equipment (see Chapter 5 and Chapter 9). Examples of this class of geophysics are seismic up-hole, down-hole and cross-hole testing, seismic tomography and down-hole logging. Despite their relative

slowness and cost, down-hole seismic techniques represent some of the best methods for use in geotechnical investigations, because they can be used to obtain good profiles of the very small strain stiffness of the ground.

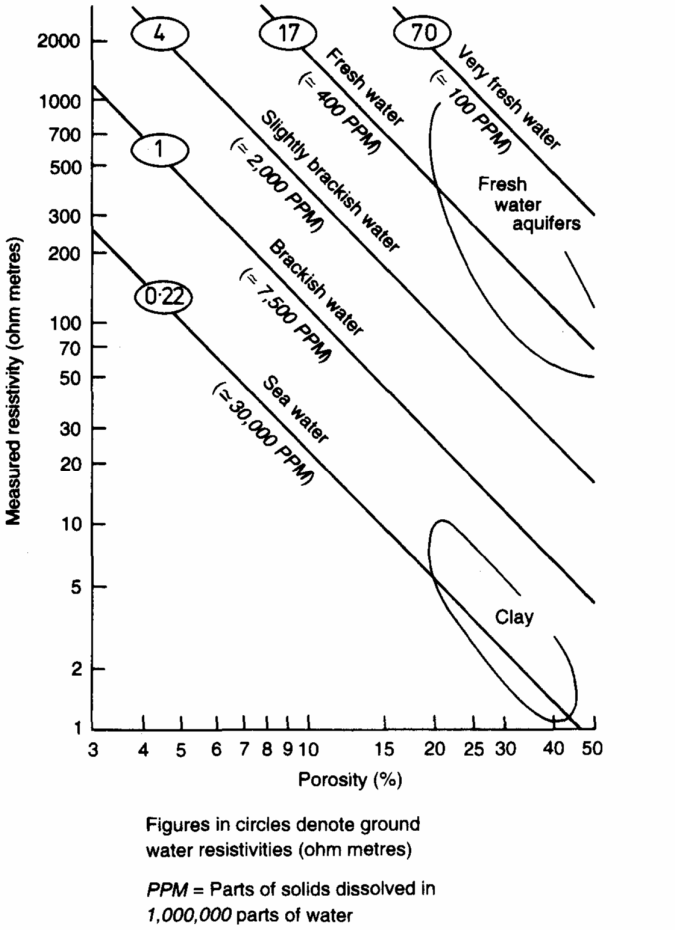


Fig. 4.1 Effects of porosity and salinity of groundwater on measured resistivity (from Guyod, 1964).

Success rate

The success rate of geophysics depends greatly upon the care taken in its planning and execution. The planning of geophysics has been discussed in Chapter 1. Geophysical surveys should, in general, be carried out by experienced personnel.

In addition to the variable factors noted above, however, it should be recognized that some techniques are intrinsically more reliable, in a geotechnical setting, than others. This perception of reliability stems from a combination of the way in which data are obtained and the purpose for which they are intended, and is tempered by a knowledge of the relative difficulty of obtaining data in other ways. Table 4.2 gives a brief summary of some of the most useful techniques for ground investigation.

Table 4.2 Geophysical methods with possible application in ground investigation

Application	Possibly viable methods	Comments
Lateral variability	Ground conductivity Magnetometry Gravity Electrical resistivity	These techniques are best used as an aid to selecting borehole locations. Preliminary borehole information will be particularly useful in selecting the best technique. The depth investigated will generally be small (of the order of a few metres) so that these techniques will find most use on shallow, extended investigations, for example for pipelines. Microgravity is claimed to have a high success rate in cavity detection.
Vertical profiling	Electrical resistivity depth probing Natural gamma logging Seismic down-hole logging	Electrical resistivity depth probing is a surface technique which utilizes curve fitting for interpretation. Therefore the sub-soil geometry must be simple. Geophysical logging of deep boreholes, for inter-borehole correlation, has been most successfully carried out using natural gamma logs. It provides additional information at relatively little extra cost.
Sectioning	Seismic tomography Ground penetrating radar Seismic reflection Electrical resistivity imaging	Seismic tomography is a complex technique, which should be used with caution. It works best on deep sections. Its success rate in cavity detection is low. Ground penetrating radar is a shallow technique. Its technical development has been rapid in recent years, and it shows great promise for the future. Seismic reflection is best used over water, although development of shallow seismic reflection techniques may permit more land use in the future.
Ground classification	Electrical resistivity Ground conductivity	Both techniques are limited to classifying the ground as cohesive or non-cohesive.
Stiffness determination	Cross-hole seismic Up/Down-hole seismic Seismic tomography Surface wave	Seismic methods are generally successful, provided that background noise levels are low. They provide extremely valuable, and relatively cheap information on the stiffness of the ground.

LATERAL VARIABILITY

The positioning of boreholes on a site can only be carried out on the basis of geological maps and records, which generally give limited detail, or on grids or sections. Where large areas are to be investigated to shallow depth, the positioning of boreholes and trial pits becomes very difficult, since the aim is to sample representative ground. Similarly, when limited targets, such as dissolution features, mineshafts, and cavities are to be searched for, borehole investigation cannot be considered as viable. The cost of drilling holes precludes sufficient investigation to guarantee that all such hazards will be found. In these situations certain geophysical techniques can be very valuable, because they can cover large areas of ground at very little cost. The following techniques may prove useful.

Ground conductivity

Ground conductivity is an electromagnetic method. Electromagnetic methods are widely used in mineral exploration, in identifying materials that are relatively good electrical conductors and are at shallow depth. As the name implies, the method generally involves the propagation of continuous or transient electromagnetic fields in and over the ground. It can also use electromagnetic fields generated by others (i.e. the high-power VLF (very-low frequency) transmissions in the 15—25kHz range, which are used for air and marine navigation).

Figure 4.2 shows the principle of operation of the Geonics EM31 and EM34 ground conductivity meters, which are widely used in relatively shallow investigation work. They use Frequency Domain Electromagnetics (FDEM). The equipment comprises two vertical coplanar coils, each connected to a power source and a measuring instrument. There is no contact with the ground. The power source passes alternating current through coil A at a fixed frequency (of the order of 0.4—10kHz). The current in the coil produces a magnetic field whose magnitude varies continuously according to the strength of the current. This induces a current flow in the ground, which can be visualized as a third coil. The amount of current flow depends upon the conductivity of the ground, and controls the strength of magnetic field that it produces. This field changes continuously with time, and generates a current in coil B. Coil B also receives the direct magnetic field from the transmitter coil, coil A, and the instrument recording the current in coil B must be designed to distinguish between those currents directly induced by coil A, and those induced by the ground. Ground conductivity instruments of this type provide a direct readout (in millisiemens per metre) which is identical to that given by conventional resistivity instruments over a uniform half space. The output can be recorded continuously, and can be linked to a portable data acquisition unit, to allow rapid downloading of data to computer.

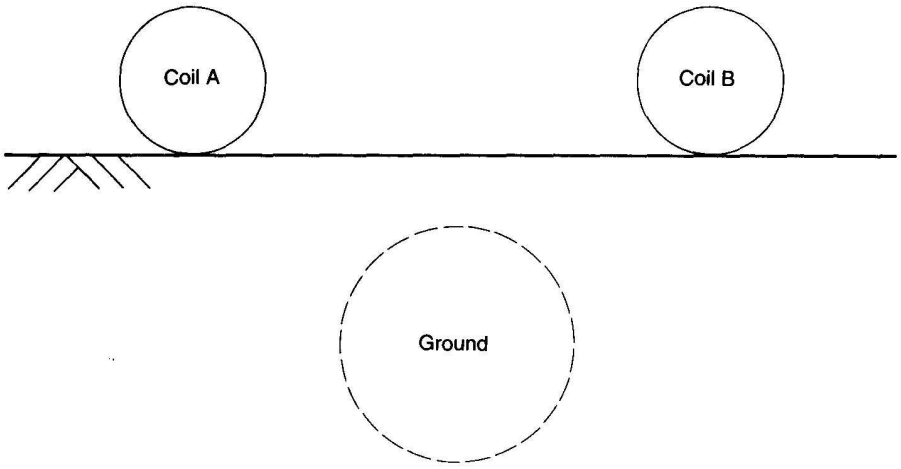


Fig. 4.2 Principle of ground conductivity surveying.

The EM31 (Fig. 4.3) is a lightweight (11kg) one-man instrument. It comprises a 4m long boom, with the coils mounted at both ends, with the operator controls and power pack at the centre. The effective depth of investigation is about 6m. The EM34 operates on the same principle, but uses two 63cm diameter coils which are carried by the two operators at a fixed spacing. The coils' spacings may be l0m, 20m or 40m, and the instrument senses to about 0.75 of the intercoil spacing in the vertical coplanar mode. Both instruments give a rapid speed of survey, and produce data which are simple to use. The site is traversed with a given coil geometry, and the output data are simply located on plan, and contoured.



Fig. 4.3 Geonics EM31 ground conductivity meter

It has been noted that modern ground conductivity meters provide data which correspond to that produced by electrical resistivity meters. Therefore the main geotechnical use of ground conductivity will be the detection of contrasts in resistivity, which (as noted above) depends primarily on the clay content of the ground and the soluble salt content of the groundwater. Therefore, conductivity surveying can be a rapid method for differentiating between areas of cohesive and non-cohesive soil, and for detecting areas of groundwater contamination. The manufacturers claim that the EM31 can also be used to locate small metallic objects, such as small ore bodies or buried metal drums in waste sites.

Magnetometry

Magnetic methods are based on the measurement of local variations in the Earth's magnetic field. Such variations are associated with differences in magnetic susceptibility (the degree to which a body is magnetized) of rocks and soils or the presence of permanently magnetized bodies. Since magnetic methods measure variations in a natural force field, the resulting data cannot be readily interpreted in a quantitative manner (i.e. depths and dimensions of subsurface features cannot be determined directly from field data). Magnetic techniques are particularly useful in locating localized subsurface features of engineering interest such as abandoned mineshafts, sink holes, and buried services. The success rate in locating such features is moderate to good when used in favourable conditions. The main advantage however, of the method is the fact that magnetic measurements can be made extremely fast and hence the use of the method is reasonably cheap.

The measurements made in magnetic surveying may be of the vertical component of the Earth's magnetic field or of the Earth's total magnetic field strength. Measurements of the vertical component of the Earth's magnetic field are made mechanically using magnetic balances. The total field strength is measured using fluxgate or proton instruments. For most engineering investigations the proton precession magnetometer is used. Extremely fast magnetic measurements can be made (usually less than 30s are spent at each station) using this instrument because it employs a remote sensing head which requires no levelling. The proton magnetometer is accurate to ± 1 nT[1], compared with ±5 nT for the fluxgate instrument. The strength of the Earth's magnetic field varies between 47000 nT to about 49000 nT from south to north across the British Isles.

Observations are normally made on a grid. The station interval along each traverse line forming the grid should not exceed the expected dimensions of the feature to be located. A station interval of between 1 and 2m is normally used for the location of abandoned mineshafts. For the location of clay filled sink holes in chalk McDowell (1975) suggests a station interval (and distance between traverses) of less than half the expected lateral extent of the feature. The field data must be corrected for diurnal and secular variations in the Earth's magnetic field. Diurnal variation is measured throughout the survey by periodically returning to a base station and measuring the field strength. The field data once corrected for these variations are normally presented in the form of a contoured magnetic map. Figure 4.4 shows a magnetic map for a site at Mangotsfield, Bristol. The contour values are relative to the regional magnetic field strength. Characteristic shapes may be recognized from magnetic maps and related to subsurface bodies in terms of general geometry and orientation (if they are not equidimensional) magnetic profiles are often drawn across anomalies to aid interpretation.

Interpretation of magnetic data is qualitative. Detailed analysis of the data may be carried out by comparing field data with theoretical anomalies produced by physical models. These models are altered to produce a best fit with the field data. The field data, however, can be highly ambiguous and a unique relationship between the anomalies produced by a single physical model and the field prototype rarely exists. In some cases the depth of a subsurface feature may be estimated from the width of the anomaly produced.

Measured field strengths are seriously affected by interference from electrical cables, electric railways, moving vehicles and highly heterogeneous ground. The latter is a common feature of urban areas in which the abundance of old foundations, buried services, and waste material gives rise to complex anomalies which can easily mask anomalies produced by singular features of engineering interest.

Ideal sites for the use of magnetic methods are on open little-developed land, free from extraneous interference. The method may be used successfully in developed areas, but care should be taken in choosing the magnetic method. Moreover, the engineering geophysicist should be presented with all the available data concerning the history of the site, which may have been obtained during the preliminary desk study.

The magnitude of magnetic anomalies associated with localized features will depend on the depth of the feature and the height of the sensor above the ground. A sensor height of 1 m above ground level has proved convenient and adequately free from magnetic variations of the top soil (Hooper and McDowell 1977). In general, the magnetic anomaly produced by a subsurface feature decreases rapidly as the depth of overburden increases. This can make detection of deep features difficult, particularly if they are of limited extent and do not show a very high susceptibility contrast with the surrounding ground. Anomalies produced by local features generally become difficult to identify when the lateral dimensions are less than the

[1] A nanotesla (nT) is a unit of magnetic field strength (1γ= 10 T= inT)

depth of the cover. The maximum depth of burial at which the detection of abandoned mineshafts generally becomes difficult is about 2—3m depending on the diameter of the shaft and the magnetic susceptibility contrast with the surrounding material. The magnitude and shape of the anomaly produced by elongate (or linear) features (i.e. elongate in the vertical plane) .such as dykes are affected by the orientation of the feature as well as the orientation of the magnetic field. Linear features which trend magnetic north— south are difficult to locate.

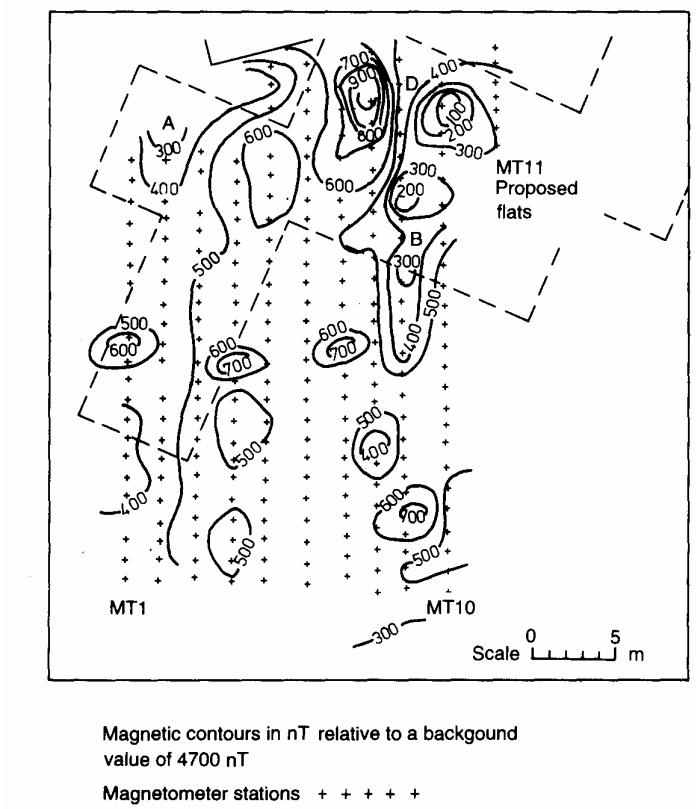


Fig. 4.4 Magnetic field strength map of Mangotsfield, Bristol (after Hooper and McDowell, 1977).

The advantage of magnetic methods is the speed at which measurements can be taken. With a proton magnetometer it is possible to cover an area of 1500 m2 in a day taking measurements on a 1 m grid. It is possible to cover 1000 m2 in 60 mm taking measurements on a 2 m grid with the less sensitive fluxgate magnetometer (Dearman et a!. 1977). Since essentially only one correction is applied to the field measurements, data reduction, presentation and interpretation can be carried out rapidly. Clearly the magnetic methods can be most cost effective in site investigations that require localized features at shallow depths to be located provided of course that the conditions are recognized as being suitable for the method to have a reasonable chance of success.

The main use of magnetic methods in site investigation appears to be for the location of abandoned mineshafts. The successful location of such features using magnetic methods has been reported by Raybould and Price (1966), Maxwell (1976), Dearman *et al.* (1977) and Hooper and McDowell (1977). It is unfortunate, however, that unsuccessful cases are not also reported, as these are equally (if not more) numerous than the successful cases. The publication of unsuccessful cases would give a better insight to some of the drawbacks of the method and perhaps allow some improvements to be made. The main problem in detecting abandoned shafts is that there is a great variety of anomalies associated with these features. Each shaft may be different from another in terms of:

1. whether it is capped or uncapped:
2. the type of capping material;
3. the type of shaft lining;
4. the type of shaft infilling material (if present);
5. groundwater table; and
6. the nature of the surrounding debris.

Thus all anomalies must be investigated by direct methods. Often the method is used in the wrong conditions, such as areas where the ground is particularly heterogeneous (which is not unusual in mining areas) or areas where there is likelihood of extraneous interference. The chances of success in such cases are minimal. The limitations of the method mentioned earlier clearly reduce the number of situations where success is possible.

When using the magnetic methods to locate geological features it should be borne in mind that the method was initially developed for prospecting and was used to locate large-scale geological features. If the geology beneath a site is particularly complex, interpretation of magnetic data will be difficult, if not impossible, in extreme cases. Locally complex geology will also present problems in locating man-made features.

McDowell (1975) discusses the use of magnetic methods in the detection of clay- filled sink holes in chalk. In favourable circumstances the proton precession magnetometer can be used to locate these features very rapidly and at little cost compared with employing direct methods of investigation, or other geophysical methods. A magnetic map for a test site in Upper Enham, Hampshire, is shown in Fig. 4.5.

The magnetic method may also be used to locate basic igneous dykes below a cover of superficial deposits (Higginbottom 1976) and buried services such as clay or metal pipes.

Electrical resistivity traversing

Resistivity traversing is normally carried out to map horizontal changes in resistivity across a site. Lateral changes in resistivity are detected by using a fixed electrode separation and moving the whole electrode array between each resistivity. The interpretation of resistivity traverses is generally qualitative unless it is carried out in conjunction with sounding techniques. The use of traversing and sounding together is quite common in resistivity surveying (for example, see McDowell (1971)). The electrode configurations normally used in electrical traversing are the Wenner configuration and the Schlumberger configuration, Table 4.3. The Wenner configuration has the simplest geometry and is therefore easier to use and quicker than employing a Schlumberger configuration. If the resistivity station interval is the same as the electrode spacing it is possible to move from station to station along a traverse line by moving only one electrode each time. This is not possible with a Schlumberger configuration as the potential electrode spacing is not one-third of the current electrode spacing.

Resistivity equipment passes a small low-frequency a.c. current (of up to 100 mA) to the ground via the current electrodes (C1 and C2). The resistance between the potential electrodes (P1 and P2) is determined by measuring the voltage between them. This voltage is normally amplified by the measuring device. The current source is then switched to an internal bridge circuit (via the amplifier) in which a resistance is altered using a potentiometer to give the same output voltage as measured between the potential electrodes. In general, as the electrode spacing is increased the measured resistance decreases. It is therefore necessary for the equipment to be capable of measuring small resistances. The resistances that may be measured with some a.c. devices are in the range 0.0003 Ω to 10 kΩ (ABEM Terrameter). The resistivity of the ground between the potential electrodes is determined from the

measured resistance. The way in which it is determined will depend upon the electrode configuration used. Over homogeneous ground the measured resistivity will be constant for any electrode configuration. The ground is, however, rarely homogeneous and, in practice, the measured resistivity will depend upon the electrode configuration. This measured resistivity for any electrode configuration is called the apparent resistivity.

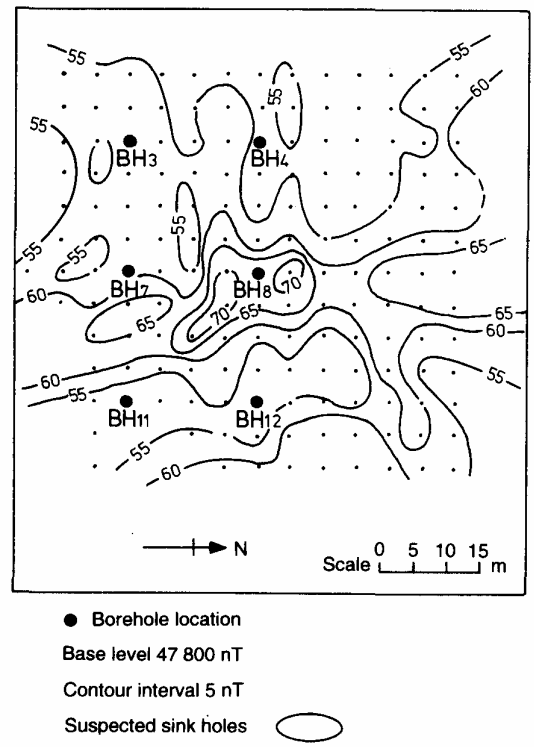


Fig. 4.5 Magnetic field strength map of Upper Enham, Hampshire (after McDowell 1975).

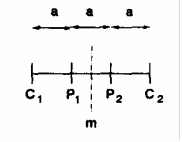
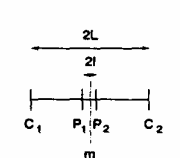
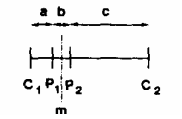
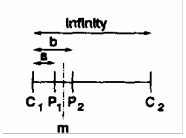
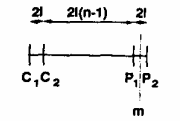
Modern resistivity meters are compact and portable. Portability of equipment is an important factor which affects the logistics of the survey, particularly when traversing.

The location of localized subsurface features such as abandoned mineshafts and cavities often requires the use of less common electrode configurations, in order to increase the sensitivity to lateral changes in resistivity which are of limited lateral extent. The central electrode configuration shown in Table 4.3 generally gives a smoother profile and larger amplitude anomalies over small features than the conventional Wenner or Schlumberger arrays. With the central electrode configuration only the potential electrodes are moved between each measurement, and hence traversing can be carried out rapidly. A 30 m long traverse with a 1 m station interval can be completed in about 30mm. with a two-man team, whereas 60mm. is required when using a Wenner configuration (with a = 2 m). The disadvantage of this electrode configuration is that the distance factor changes for each station making it difficult for the observer to form a picture of the pattern of anomalies directly from the field data. Spurious measurements are therefore too easily missed until the apparent resistivities are calculated.

The Wenner configuration has the disadvantage of producing large flanking anomalies adjacent to anomalies produced by sharp changes in resistivity (Fig. 4.6). This can be a serious limitation, particularly when using the method to locate localized subsurface features, such as sink holes, buried channels or abandoned mineshafts, as the flanking anomalies can mask the main anomaly (Cook and Van Nostrand 1954). The Schlumberger configuration will

in the same situation cause the main anomaly to be enhanced and the flanking anomalies to be reduced.

Table 4.3 Types of electrode configuration commonly used in resistivity surveying

Type of electrode configuration	Sketch of electrode configuration	Distance factor	Comments
Wenner		$2\pi a$	Not suitable for location of narrow steep-sided features, such as fault zones and dissolution features because of flanking anomalies. All four electrodes are moved when traversing.
Schlumberger		$\frac{\pi(L^2 - l^2)}{2l}$	Reduces flanking anomalies. Sensitivity with increased depth, drops less rapidly than with Wenner. All four electrodes are moved when traversing.
Central electrode		$\frac{2\pi}{b(\frac{1}{a(a+b)} + \frac{1}{c(c+b)})}$	Suitable for location of sharp changes in resistivity. Traversing is rapid as current electrodes are not moved.
Pole-dipole		$\frac{2\pi ab}{(b-a)}$	Remote current electrode C2. It is not necessary to have C2 in line with the other electrodes. Configuration permits lateral exploration on radial lines from a fixed position of C2. Suitable for resistivity mapping in the vicinity of a conductor of limited extent.
Double-dipole (dipole-dipole)		$2\pi nl(n-1)(n+1)$	Common configuration used with induced polarization work. n must be less than or equal to 5.

The electrical resistivity method has been used with limited success in the detection of cavities. In view of the time required for a resistivity survey compared with that required for a magnetic survey (discussed above), the resistivity method is not as cost effective in view of the small chances of success. A successful attempt to locate abandoned mineshafts using electrical resistivity is reported by Barker and Worthington (1972).

Resistivity techniques have been used successfully in arid and semi-arid areas in groundwater investigations. Some case histories are given by Martinelli (1978). Krynine and Judd (1957) report the use of the electrical resistivity method to locate and assess the nature of faults and fracture zones suspected of running parallel to a proposed tunnel line. Resistivity methods have also been used to locate and map the extent of fissures in karst dolomite for the foundation design of a dolomite processing plant near the town of Matlock (Early and Dyer 1964).

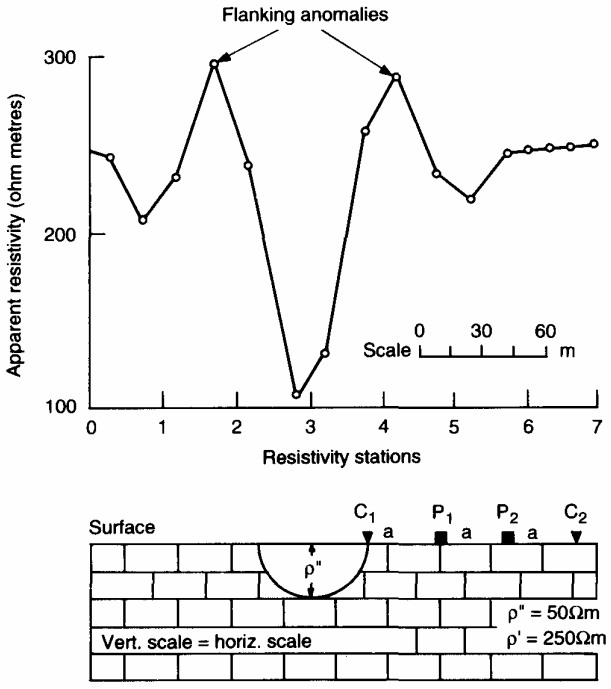


Fig. 4.6 Theoretical flanking anomalies produced over the edges of localised features (after Cook and Van Nostrand 1954).

Gravity methods

Gravity methods involve measuring lateral changes in the Earth's gravitational field. Such variations are associated with near-surface changes in density and hence may be related to changes in soil or rock type. Because gravity methods involve the measurement of a natural force field, ambiguous data (in terms of interpretation) are common and hence the interpretation of field data is qualitative. The effects of rapid near-surface changes in density limit the use of this method in practice to the mapping of large- scale geological) structures. This can be of great value in oil exploration but on the very much smaller scale of engineering site investigations it seriously restricts the use of gravity measuring techniques. Gravity methods may be used for the location of large faults and to find the extent of large buried channels.

As mentioned earlier, density does not vary greatly between different soils and rocks. Thus local variations in gravity (gravity anomalies) will be small compared with the overall gravitational field strength. Gravity measuring instruments must therefore be extremely sensitive. Gravity meters comprise sensitive balances such that small variations are magnified by mechanical or mechanical and optical methods to enable readability. Only the vertical component of gravity is measured with these instruments.

The acceleration of gravity is measured in units of milligals (1 milligal $=10^{-5}$ m/s^2). Gravity meters have a sensitivity of about 1 part in 108 of the Earth's gravitational field which represents a resolution 0.01 milligal. The most sensitive gravity meter has a resolving power in the microgal range.

Measured values of the vertical component of the gravitational field are not only a function of density but also a function of latitude, elevation, local topography, and tidal effects. The effects of these on gravity measurements can be determined and corrections made. Every field measurement must be reduced by applying the following corrections.

1. *Latitude correction*. This is applied because the Earth is not a perfect sphere and hence the gravitational force varies between the poles and the equator.
2. *Elevation correction*. This is applied to reduce all gravity measurements to a common elevation. The correction is in two parts.
    (i) *Free air correction*. This is made for the displacement of the point of observation above the reference level (usually ordnance datum O.D.). This requires the levelling of the gravity meter to within 0.05 m.
    (ii) *Bouguer correction*. This is applied to remove the effect of the material between the gravity meter and the datum level. This requires the density of the material to be known (usually it is estimated).
3. *Terrain correction*. This is applied to remove the effects of the surrounding topography. The correction is obtained from a graphical determination of the gravity effect at the observation point of all hills and valleys. The correction is usually calculated to about 20km out from the station but where relief is low the survey area considered for this correction is reduced (Higginbottom 1976).
4. *Instrument drift and tidal correction*. These corrections are made by returning to a base station and measuring the field strength at frequent intervals during the survey.

The corrected gravity measurements are known collectively as Bouguer anomalies.

A gravity survey requires an accurate topographic survey of the site and surrounding area to be carried out to enable some of the corrections to be applied to the field data. In the British Isles most of the topographic data necessary may be obtained from Ordnance Survey maps and plans. Each observation point must however be accurately levelled. Clearly the acquisition and reduction of gravity data are extremely time- consuming and hence expensive.

The corrected gravity data are normally presented as a contoured gravity map (Isogal map). The contour values may represent Bouguer anomalies or residual gravity values. Residual gravity values are derived from the difference between the regional Bouguer anomaly and the local Bouguer anomaly.

These gravity maps allow anomalies to be readily identified (if of sufficient magnitude) and thus target areas are defined for direct investigation. Figure 4.7 shows examples of the different forms of gravity maps.

To enable detection of subsurface features the amplitude of the gravity anomaly produced must be at least 0.2 mgal. Most features of engineering interest produce anomalies which are much smaller than this and in most cases may be detected more efficiently by other geophysical methods. The main exception according to Higginbottom (1976) is the case of faults with displacements large enough to introduce materials of different density across the fault plane, but where the contrast between other physical properties is slight. A density contrast may also give rise to a P-wave velocity contrast. It would be easier therefore to use a seismic method such as seismic refraction if this were the case.

The gravity method has been used to aid determination of the cross-section of an alluvium filled valley in North Wales (Fig. 4.7, see Griffiths and King (1965), for discussion) and the location of cavities (Colley 1963).

In general, gravity methods are too slow and expensive to be cost effective in conventional site investigations. Only in rare circumstances are the use of gravity methods justified particularly as justification must normally be based on the limited information available at an early stage of the investigation.

In contrast with conventional gravity surveys, which are generally concerned with large-scale geological structures and the identification of different rock types over large areas of ground, ***micro-gravity*** surveys are limited both in extent and objectives. Station spacings may be as close as 1 m, and the normal use of micro-gravity is in the construction industry, to detect subsurface cavities. Here the target has a large density contrast with that of the surrounding ground so that, provided it is sufficiently shallow and there is little 'interference' in the form of complex geometry (for example from the gravitational pull of surrounding buildings, or the effects of nearby embankments, tunnels or basements), the method should be of use in what is otherwise a particularly difficult problem area for geotechnical engineers.

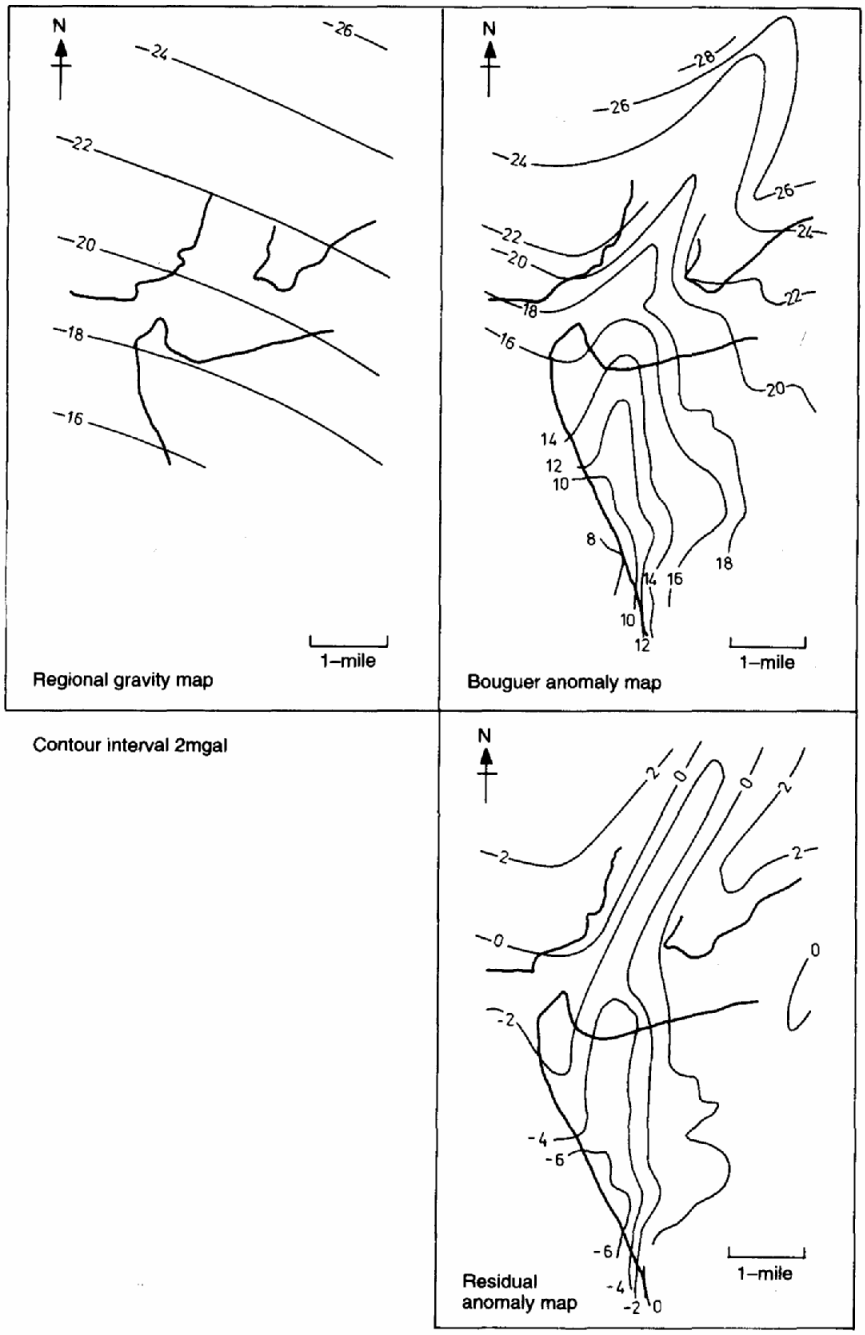


Fig. 4.7 Different types of gravity map for the Harlech, Portmadoc area of North Wales (from Griffiths and King 1965).

Typically, microgravity surveys may involve between 100 and 400 gravity stations with spacings as close as 1 m. The stations are levelled (using precise levelling equipment) and gravity measurements made to a resolution of 1 gal. Typical anomalies associated with shallow (0—l0m) features (e.g. voids, dissolution features and disturbed ground) are between 20 and 100 μgal. Using a portable PC, preliminary data processing may be carried out on site in order to confirm adequate definition of anomalies.

PROFILING

Most ground investigations will not make use of geophysical methods for profiling. This will normally be done by describing the material arising from boreholes, or by carrying out probing tests (Chapter 5). Exceptions may occur when there is need for information from areas between boreholes, or where boreholes are deep, and there is a need to correlate between them. Profiling can be carried out by identifying the characteristics of the material within each bed, or by identifying marker beds which are common to all boreholes.

Whilst electrical resistivity can be used for this purpose, it is not common. Seismic probing is on the increase, and natural-gamma logging has long been used for inter- borehole correlations when deep investigations are being carried out.

Electrical resistivity sounding

Vertical changes in electrical resistivity are measured by progressively moving the electrodes outwards with respect to a fixed central point. The depth of current penetration is thus increased. Any variations in electrical resistivity with depth will be reflected in variations in measured potential difference.

Electrical sounding involves investigating a progressively increasing volume of ground. As the vertical extent of this volume increases so will the lateral extent. Lateral variations in electrical resistivity will therefore introduce errors when determining variations of resistivity with depth. Ideally the lateral dimensions of the volume of ground under consideration should be kept relatively small compared with the vertical dimension. Electrical sounding using the Wenner configuration (Table 4.3) requires that both the current and potential electrode separations are increased between each resistivity measurement. The lateral dimensions are therefore allowed to become large. Thus Wenner sounding is likely to produce an erroneous resistivity/depth relationship because of lateral variations in resistivity.

When electrical sounding with a Schlumberger configuration is used the potential electrode spacing is kept small (potential electrode spacing 0.2 current electrode spacing) and only the current electrode spacing is increased between each resistivity measurement. The potential electrode spacing is only increased when A V becomes very small and in this way the minimum lateral dimension condition is more-or-less satisfied. Thus the results of Schlumberger sounding are less prone to error due to lateral changes in resistivity. Figure 4.8 shows the effect of a lateral change in resistivity on the results of electrical sounding using Wenner and Schlumberger configurations.

When using the Wenner array the errors due to lateral changes in resistivity may be greatly reduced and often eliminated by the use of the Offset Wenner system. The principles of this system may be summarized by considering the ‘signal contributions section’ for a Wenner array shown in Fig. 4.9. For homogeneous ground the positive and negative contributions of high magnitude cancel each other out and the resultant signal originates mainly from depth and not from the region around the electrodes. If, however, a high resistivity body (for example, a boulder) is located in the positive zone, the measured resistance will be greater than would have been measured in the absence of the body; if it is located in a negatively

contributing zone the measured resistance will be lower. In each case the body will cause an error in the resistance measurement. The effect of the body may be reduced by measuring the resistance with the body in the negative zone and then moving the electrode array such that the body is in a positive zone and taking a second resistance measurement. If the two resistance measurements are averaged, the effect of the body is eliminated or at least greatly reduced.

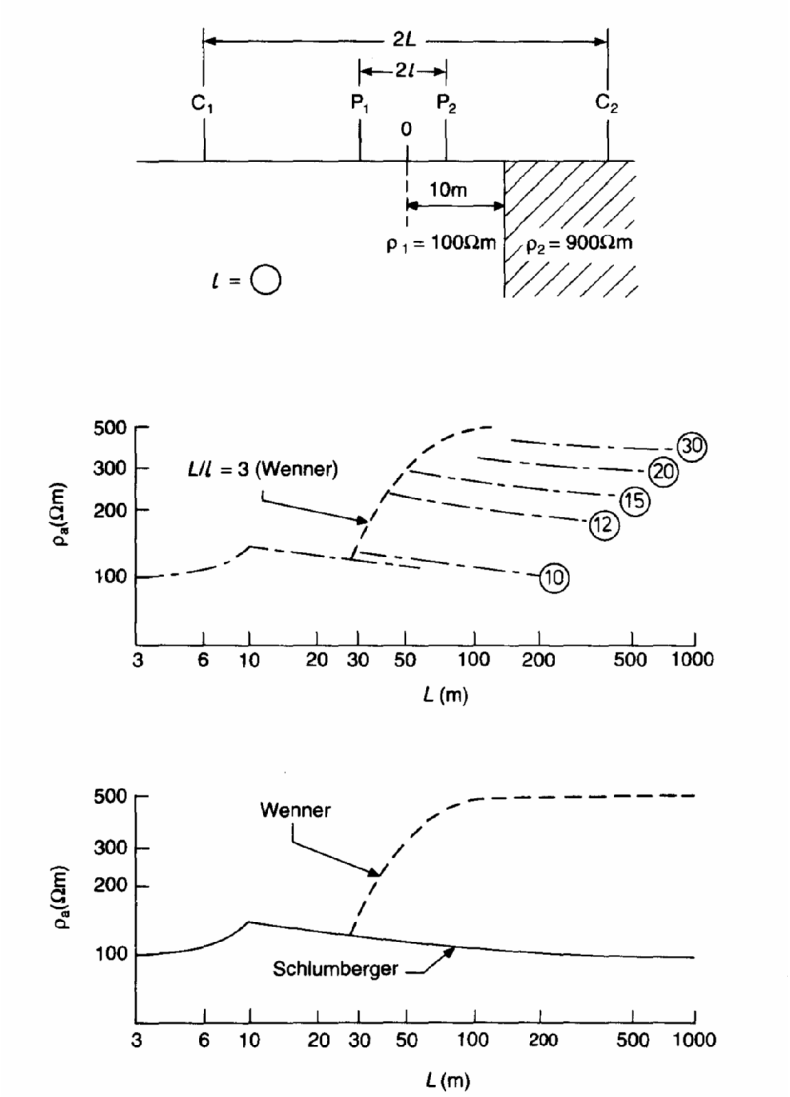


Fig. 4.8 Comparison of data produced by different electrode configurations across a sharp lateral change in resistivity (after Van Zijl 1978).

In practice, Offset Wenner sounding is conducted using a five-electrode array as shown in Fig. 4.10. When using this array the additional electrode remains fixed at the centre of the spread and a switching device is used to change from one adjacent set of four electrodes to the other. Two resistances (R_{D1} and R_{D2}) are measured and averaged to obtain the Offset Wenner resistance RD. If the spacing is increased according to the series a=0.5, 1, 2, 4, 8, 16, 32, 64... 2^n m, so that the potential electrode always falls on a position previously occupied by a current electrode, the necessary data required for depth sounding may be obtained. In practice, sounding data are acquired by placing electrodes at all the electrode positions to be used and attaching them to two multicore cables as shown in Fig. 4.11. The connection points on the multicore cable may be positioned at the necessary spacings to reduce the time spent in

setting out the array. Two resistance readings are taken for each electrode spacing using a switching device and a digital resistivity meter. A portable computer may be used to control the switching device and to store the resistivity measurements. The advantages of the Offset Wenner sounding system include the following.

1. Near-surface lateral resistivity variations are greatly reduced and this results in a sounding curve which is smoother than both the comparative Wenner and Schlumberger curves.
2. The whole system may be carried and operated by one person. Using modern lightweight equipment a sounding may be conducted in much less than one hour.
3. A conventional Wenner apparent resistivity curve is obtained.
4. The magnitude of lateral resistivity effects may be estimated. This estimate is provided by the 'Offset error' defined by the following expression:

$$Offset_error = \frac{R_{D1} - R_{D2}}{R_D} \times 100\%$$

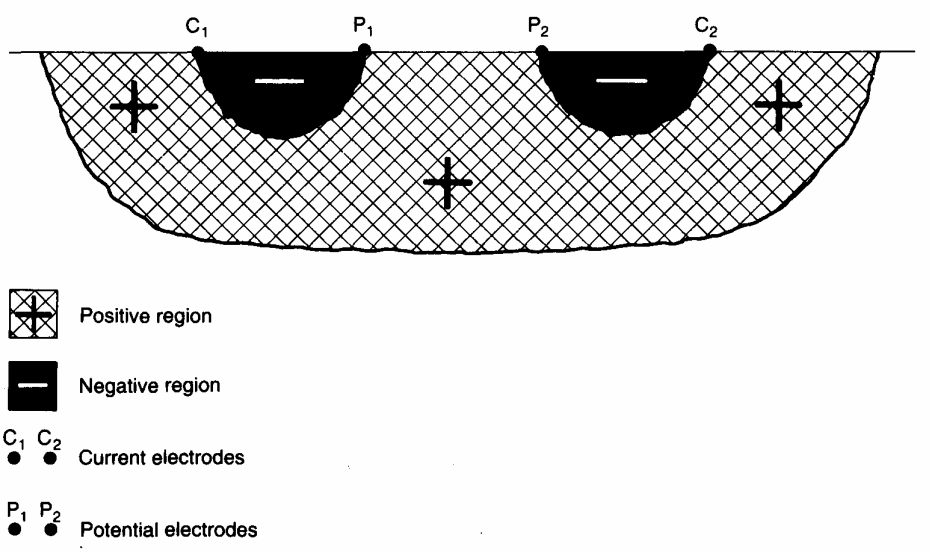


Fig. 4.9 Signal contribution for a Wenner array.

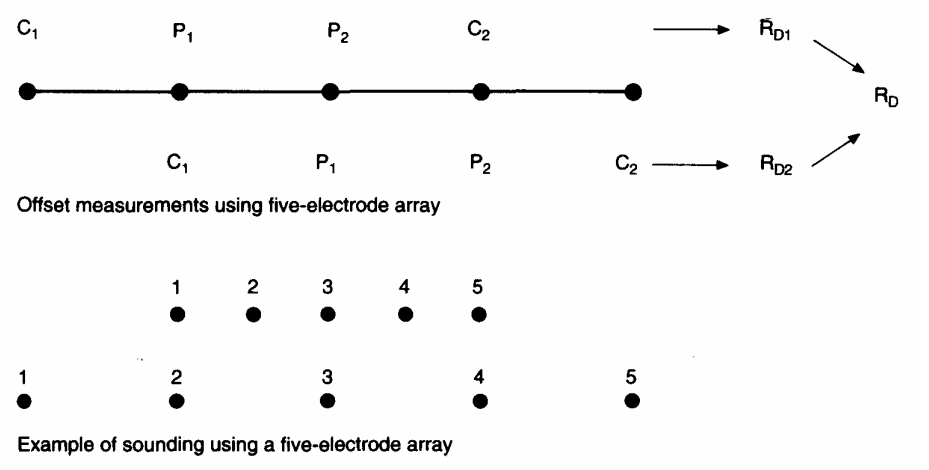


Fig. 4.10 Principles of sounding with Offset Wenner five-electrode array.

If the offset error is systematically greater than 10% the interpretation of the apparent resistivity curve may be significantly in error. Although the Offset technique substantially reduces near-surface lateral effects, deeper large-scale lateral resistivity variations, such as

dipping strata and faults are likely to distort the sounding curve seriously. The Offset error will indicate when such situations are encountered.

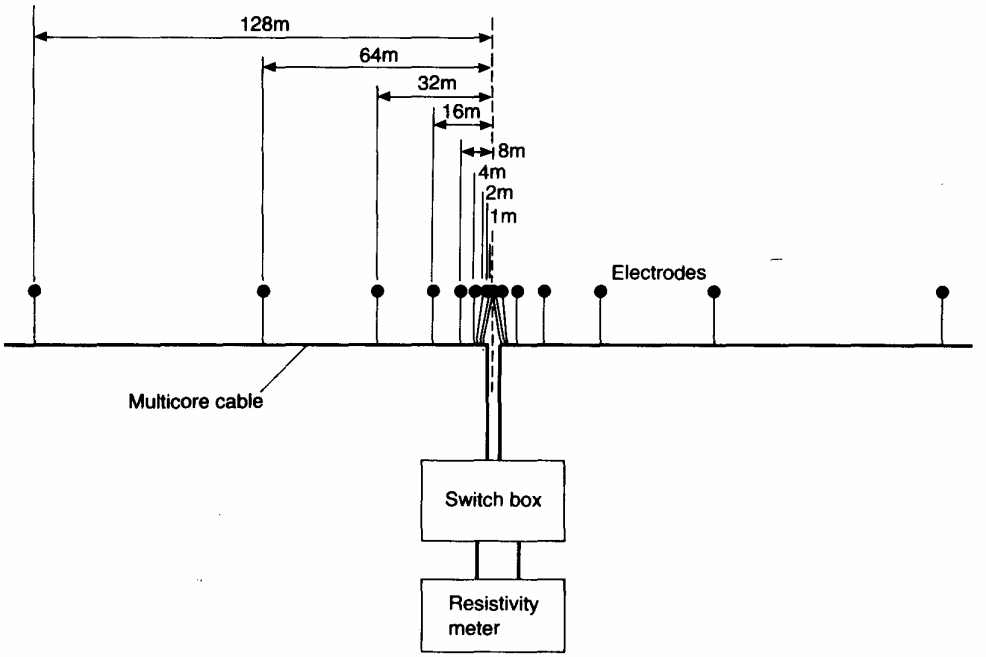


Fig. 4.11 Arrangement for Wenner Offset sounding using multicore cable and switch box for a=0.5, 1,2,4,8, 16, 32 and 64m.

The major potential disadvantage of Offset Wenner and conventional Wenner sounding is the large electrode spacing required to obtain a significant depth of current penetration. For example a typical Offset Wenner spread with a_{max} = 64 m has a total length of 256 m. This is clearly a limitation when using these methods on restricted sites. The Schlumberger configuration requires a smaller current electrode spacing to achieve similar depths of current penetration.

Errors in apparent resistivity measurements can be caused by the following.

1. *Electromagnetic coupling between potential and current electrode cables.* This may be overcome by ensuring that current and potential electrode cables do not cross each other, are not laid very close to each other or are adequately shielded.
2. *Interference from high (or low) tension electrical cables or electrified railway lines.* Sounding or profiling near electrical cables of any sort should be avoided. The interference from overhead high tension cables can be reduced by having the azimuth of the electrode array parallel to the line of the cables.
3. *Highly heterogeneous ground.* If the surface layer is highly heterogeneous due to complex geology, buried services, old foundations, root holes, or waste material, errors may be introduced in sounding data even when a Schlumberger configuration is used.

Quantitative interpretation of electrical sounding data is possible using various curve matching techniques. The measured values of apparent resistivity are plotted as a function of current electrode spacing. For Wenner configurations ρ_a is plotted as a function of a (1/3 current electrode spacing) and for Schlumberger configurations ρ_a is plotted as a function of L (1/2 current electrode spacing). Logarithmic scales are normally employed as this facilitates direct curve matching techniques since there are no problems in comparison of data due to the

use of different scales. The pattern of current distribution through a stratified media can be derived theoretically. Thus theoretical curves (master curves) of apparent resistivity against electrode spacing can be produced (on logarithmic scales). Such curves have been computed for strata parallel to the ground surface by Mooney and Wetzel (1956) and Mooney and Orellana (1966). A full set of master curves for strata parallel to the ground surface is published by La Compagnie Générale de Géophysique (1963). An example of one set of master curves is given in Fig. 4.12. These master curves are matched with the field curves. The master curve which best matches the field curve can be used to determine the apparent resistivities and the depths of each layer detected in the field. Figure 4.13 shows some field data on which a best-fit master curve has been superimposed. The field curve will never exactly match the master curve because of undulating or dipping interfaces which cannot be represented in a finite set of master curves. Thus a subjective element is introduced in the curve matching process and the choice of physical model must be aided by a knowledge of the likely geological succession and groundwater conditions. Preliminary electrical soundings are therefore carried out where possible close to the site of a borehole or near an exposure to provide a control over interpreting subsequent electrical soundings. Data processing techniques now allow a more objective approach to be made in finding a physical model which best represents the field prototype. Most sets of published master curves are for strata which are parallel to the ground surface and it is therefore necessary to have prior knowledge of the local dip and strike of the strata beneath the site. It is most unlikely that the ground surface will have the same inclination (in both magnitude and direction) as the dip of the strata beneath it and it is therefore necessary to attempt to orientate the azimuth of the electrode array along the strike of the dipping strata. This is not always possible as the ground may not be horizontal in this direction. Clearly this is a serious limitation of the electrical sounding method. Master curves for dipping strata have been computed by Maeda (1955), but the number of curves involved tends to over-complicate interpretation by curve matching alone. The dipping strata problem tends to lend itself to the more modern data processing techniques.

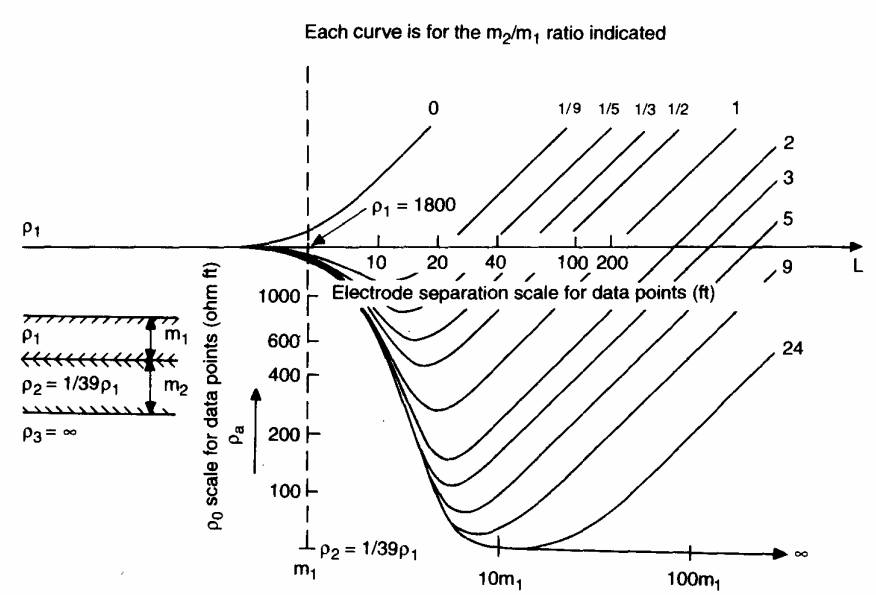


Fig. 4.12 Typical set of master curves for Schlumberger sounding (Compagnie Générale de Géophysique 1963).

The computation of master curves is based on the assumption that each layer considered is isotropic and homogeneous in terms of resistivity. This is rarely true in soils and rocks and can lead to ambiguities in interpretation and hence serious errors in depth determinations. In practice, measurements taken at the ground surface do not allow for any differentiation to be

made between isotropic and anisotropic layers. The degree of resistivity anisotropy in anisotropic layers is generally greater than unity and hence depths are overestimated. Anisotropy of the clay overlying the chalk in Fig. 4.13 is likely to be the reason why the depth of the top of the chalk is overestimated by the interpretation of the resistivity data.

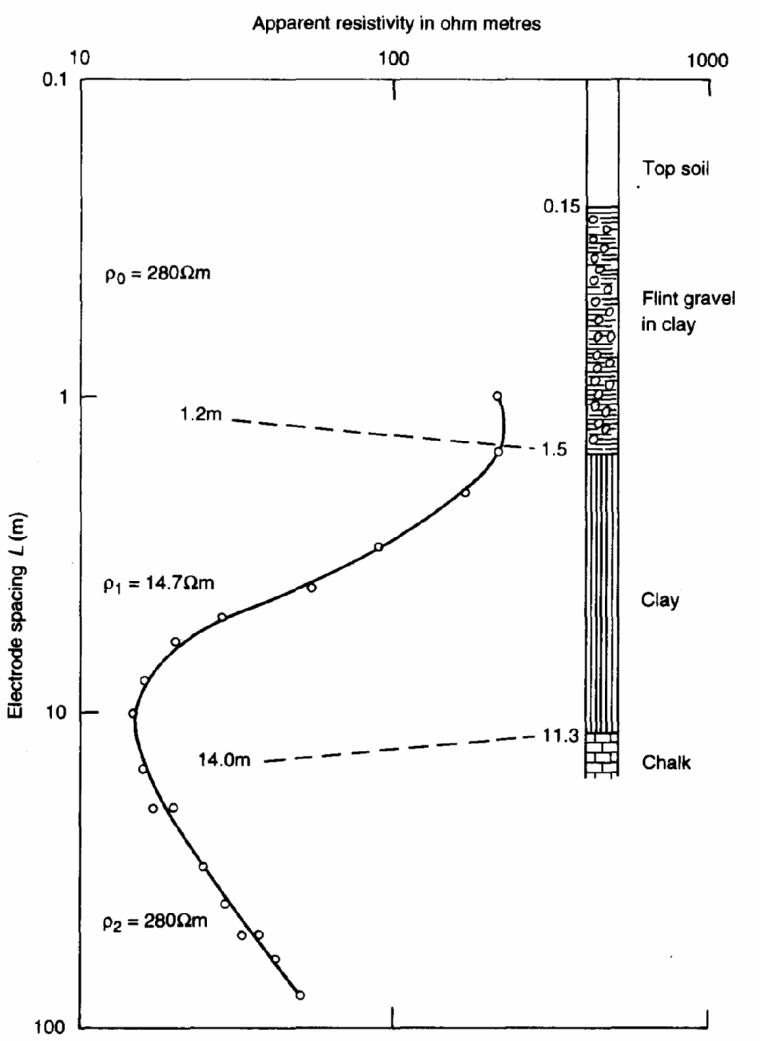


Fig. 4.13 Interpretation of electrical resistivity sounding data.

Seismic profiling

A relatively quick method of obtaining a profile which will approximate to ground stiffness is to drive a probe or penetrometer into the ground, using it either as a seismic source, or as a receiver if geophones are incorporated into it.

The seismic cone is one of a large family of cone penetrometers (see Chapter 5) which are widely used in geotechnical investigation, particularly in relatively soft ground. Essentially the device consists of a solid metal shaft, 10cm2 in cross-sectional area and with a 60° cone end, which is pushed hydraulically into the ground. By mounting geophones in the cone, the arrival of seismic waves can be detected, and these can be interpreted in terms of travel times, and hence seismic velocities and ground stiffnesses.

Figure 4.14 shows the principle of the test method. A seismic piezocone is pushed progressively into the ground. Each metre or so it is stopped. A hammer is used at the surface, to produce seismic waves. For shallow saturated soil the compressional (P) wave velocity is normally primarily a function of the bulk modulus of the pore water, and is not sensitive to changes in the stiffness of the soil skeleton. Shear waves are usually used, and are typically generated by placing a wooden sleeper under the wheel of the penetrometer truck and striking it horizontally with a large hammer.

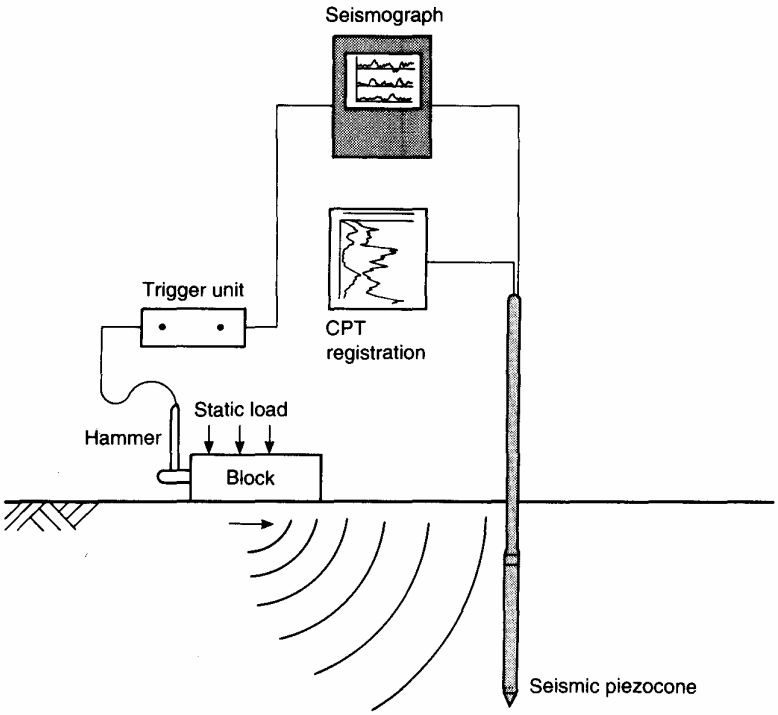


Fig. 4.14 Principle of seismic shear wave profiling using the cone penetrometer (courtesy Fugro).

The striking of the block with the hammer triggers an armed seismograph, and this records the arrival of the seismic waves at the cone tip. If the site is noisy, the signal- to-noise ratio can be improved by repeating the process and stacking the signals. The travel time from the surface to each cone position is determined from the seismograph traces, and the time taken for the wave to travel between each cone position is determined by subtraction. The so-called 'interval velocity' is then determined by dividing this difference in travel time by the distance between the two cone positions. Better results can be obtained by using a cone with two sets of geophones, mounted a metre or so apart, in it. Finally, the very small strain shear modulus of the soil can be determined from the equation:

$$G_{\circ} = \rho V_s^{\,2} \tag{4.5}$$

where ρ = soil density, which can either be estimated or determined from samples, and V_s = seismic shear wave velocity. Figure 4.15 shows a comparison of shear wave velocities determined both by the seismic cone and by a cross-hole test (see later), compared with the cone resistance itself.

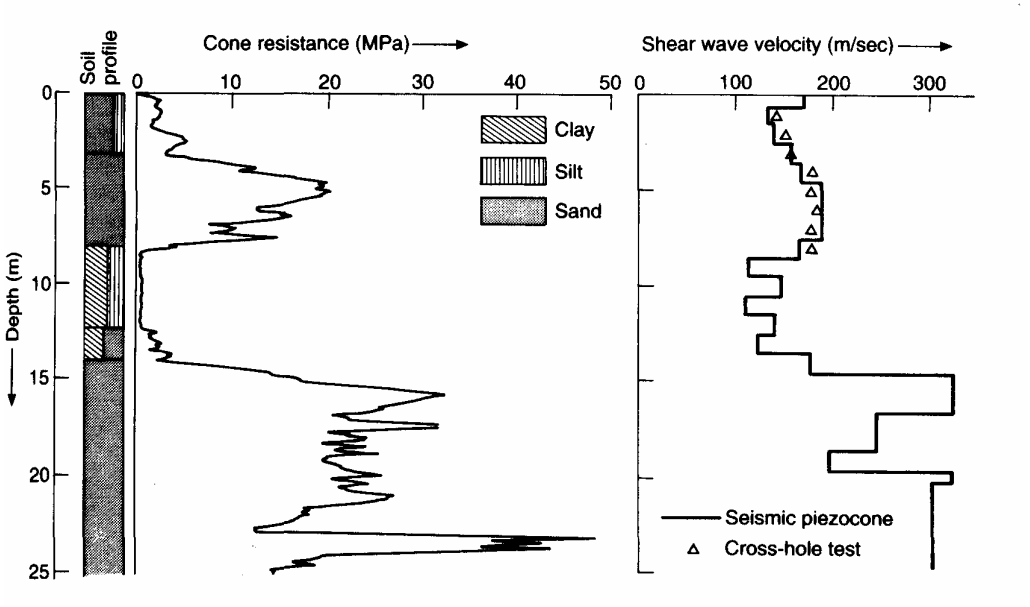


Fig. 4.15 Results of a seismic cone test (courtesy Fugro).

A further possible method of obtaining a profile is to use a conventional penetrometer as a shear-wave source, and place geophones at the surface. This is an inversion of the scheme described above. It has been found the Standard Penetration Test (described in Chapter 9) produces seismic waves which are rich in shear wave energy. Any penetrometer may be used as a source, provided that its tip is oversized so that the energy is input into the ground only at its base. This method will allow dynamic penetrometers and probes to be used directly to obtain information on ground stiffness, but it has the added complication that the component of travel time due to the wave travelling down the steel rod to the penetrometer tip must be subtracted from the travel time before the interval velocity is calculated.

Natural gamma logging

Geophysical logging of boreholes is most widely used where coring has not been carried out, for example in oil exploration, to obtain information on:

1. the geological formations through which the borehole is drilled (Rider 1986);
2. the borehole fluid; and
3. the construction and condition of the borehole.

Telford *et al.* (1990) have argued that, given the relatively low cost of borehole logging, it should be carried out on all deep boreholes. However, many of the available techniques do not give information of use for geotechnical purposes. In addition, many logs that are produced are qualitative in nature.

Borehole logging is carried out by lowering an instrument to the bottom of the hole, and raising it progressively whilst making measurements. The logging equipment comprises four parts: the instrument for making measurements (termed a 'sonde'), the cable and winch, power and processing modules, and a data recording unit (Fig. 4.16).

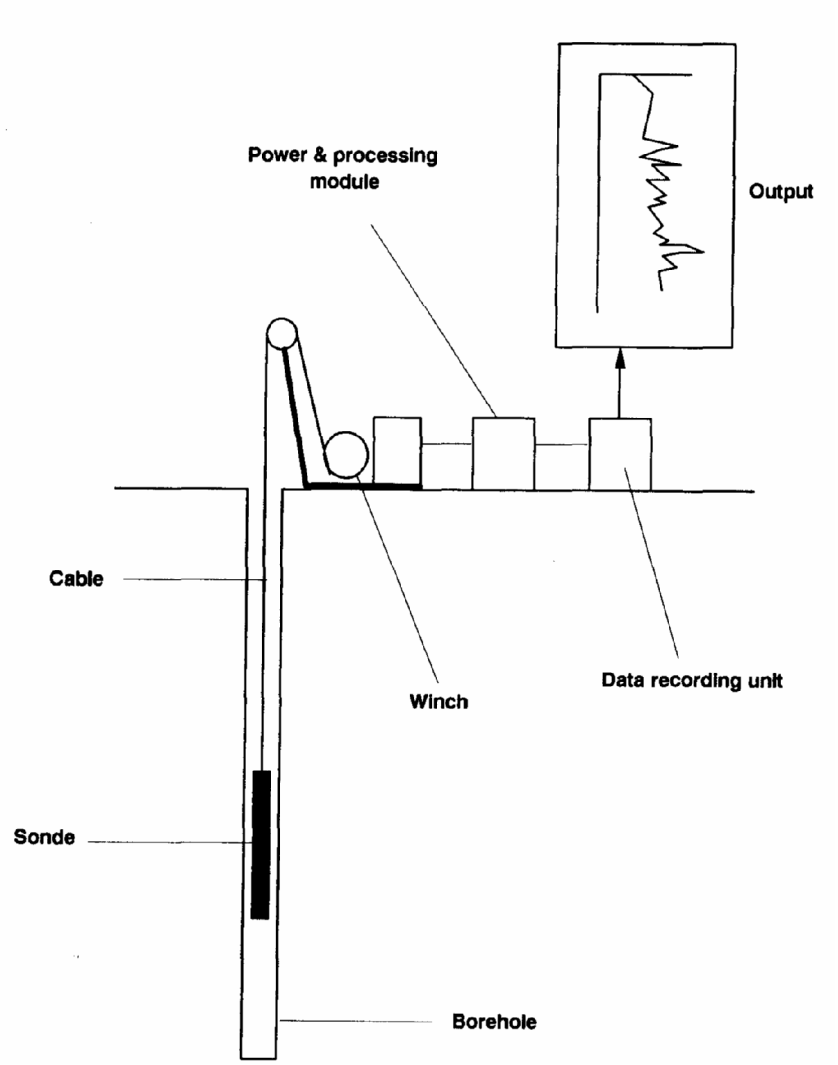


Fig. 4.16 Elements of a geophysical borehole logging system.

There are a large number of types of sonde available, each measuring a specific property. These are summarized in Table 4.4. Many of the sondes that are available are unsuitable for use in ground investigation, not least because of the need to use steel casing to support unstable ground. Resistivity and sonic logs cannot work in cased holes. Sonic logs produce results which are related to P-wave velocity, and this is of little relevance in near-surface deposits. There is little interest in the borehole fluid during ground investigations, since this is often a product of the drilling process.

Borehole construction logs are of limited value, with the exception of CCTV, and this will not function if the borehole fluid is cloudy.

The remaining tools use radiometric methods. Of these the natural gamma sonde has been the most used, because it is suitable for inter-borehole correlation. Individual sections of geophysical logs may have distinctive shapes, or characteristic signatures, which can be mapped with the same features on logs from adjacent boreholes, so confirming the lateral extent of beds within the formation. Figure 4.17 shows an example.

The sonde for natural gamma-ray logging consists of a scintillation counter, normally a sodium iodide crystal, a photomultiplier and associated electronics. The recorded gamma-ray

emissions are a function of the sampling time (i.e. the rate at which the sonde is moved up the hole) and the hole diameter. The presence of casing also affects the sensitivity of the log to variations in the formation.

Table 4.4 Borehole logging sondes

Type of measurement	Sonde type	Property measured
Electrical	Spontaneous potential	Potentials caused by electrochemical differences between the salinity of fluids or the mineralogy of the ground.
	Single-point resistivity	Electrical resistivity of the ground around the borehole.
	Normal resistivity	As above.
	Focused electric	As above.
	Micro-resistivity	As above, but for a limited zone around the hole.
	Induction	Electrical conductivity of the ground around the borehole.
Radiometric	Natural gamma ray	Natural radiation emitted as a result of the disintegration of uranium, thorium and potassium, which occur mainly in clays, marls and shales.
	Spectral gamma	As above.
	Gamma - gamma (density)	Attenuation of back-scattered radiation as a function of electron density of the ground surrounding the borehole.
	Neutron (porosity)	Hydrogen content of the formation.
Sonic	Sonic	Travel time of compressional waves over a fixed interval of ground adjacent to the borehole.
Fluid	Temperature	Temperature of borehole fluid.
	Conductivity	Electrical conductivity of the borehole fluid.
	Flowmeter	Upwards or downwards velocity of the borehole fluid.
Borehole construction	Caliper	Borehole diameter.
	Cement bond	Amplitude and received acoustic signal.
	Closed circuit TV	Visual record of borehole.

SECTIONING

Some geophysical methods can be used to produce cross-sections of the ground. Given that it is extremely difficult to find certain targets (for example, buried valleys, old mineworkings, and dissolution features) with borehole investigations, this ability makes geophysics particularly attractive for some ground investigations. Three techniques that are either widely used or show promise are:

- ground-probing radar;
- seismic reflection; and
- seismic tomography.

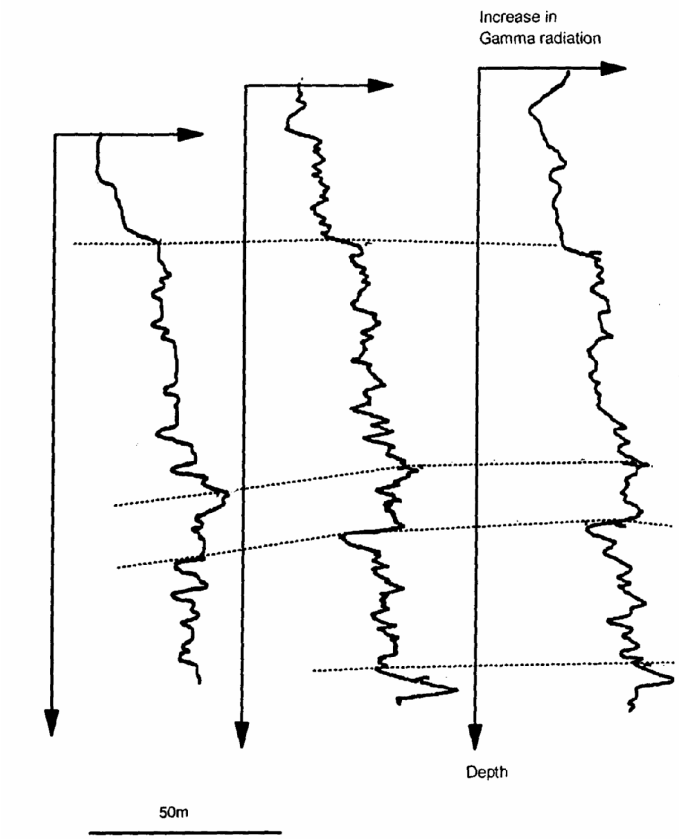


Fig. 4.17 Correlation between three boreholes using natural gamma-ray logs (BSI, 1987).

Ground penetrating radar

Radar (RAdio Detection And Ranging) was initially developed as a means of using microwaves to detect the presence of objects, typically aircraft and ships, and to derive their range from the transmitter. This process was achieved by transmitting pulses of radiation and recording the reflection. Advances in radar technology have seen the development of systems capable of providing images of the ground surface from aircraft or from space, and systems that can penetrate the ground enabling subsurface features to be mapped. This latter system is known as ground penetrating radar. It differs from those systems used to detect aircraft and shipping and to provide images of the ground surface in the power of the transmitter and the wavelength of the signal it produces. Typically, radar systems used in remote sensing operate with frequencies of about 1 GHz and have wavelengths between 8 and 300mm. Electromagnetic energy transmitted at such high frequencies will only penetrate the ground to a depth of a few hundred millimetres.

Ground penetrating radar operates at frequencies between 1 and 2500 MHz and is capable of penetrating the ground to depths of more than 30m. However, the depth of penetration is very sensitive to the electrical properties of the ground and in the case of ground with a relatively high conductivity (for example, saturated clay) the depth of penetration may be reduced to less than 1 m. Ground penetrating radar was first used to map the thickness of ice sheets in the Arctic and Antarctic and glaciers. From the early 1970s it began to be used in non-ice environments and today its application spans most types of ground. It is now being used in site investigations to map subsurface features such as rockhead, groundwater table, voids, fractures in rock and the extent of contaminated ground. It is also used to provide sections of road pavement structure for highway maintenance (Greenman 1992), to detect voids under

concrete pavements (Moore *et al.* 1980; Steinway *et al.* 1981), to check the integrity of tunnel linings and to locate reinforcement bars in reinforced concrete structures.

The radar unit produces a pulsed electromagnetic wave which travels through the ground at a velocity controlled by the electrical properties of the ground. Differences in relative permittivity (dielectric constant) or electrical conductivity resulting from changes in soil type or groundwater chemistry will result in the waves being reflected. The signals reflected from subsurface interfaces or buried objects are received by the same antenna for transmission (see Fig. 4.18). The receiving electronics amplifies and digitizes the reflected signals which are stored on disk or tape for complete post- processing. The radar record is similar to a seismic record in that it consists of a wave form in the time domain. Thus once the return signal is received by the antenna the radar system acts in a similar manner to a seismograph in providing an accurate timebase for storing and displaying the radar record. One major difference between a seismograph and a radar system is in the time-base resolution. For a radar system the resolution is measured in tens of picoseconds, whereas the resolution for a seismograph may be several hundred nanoseconds.

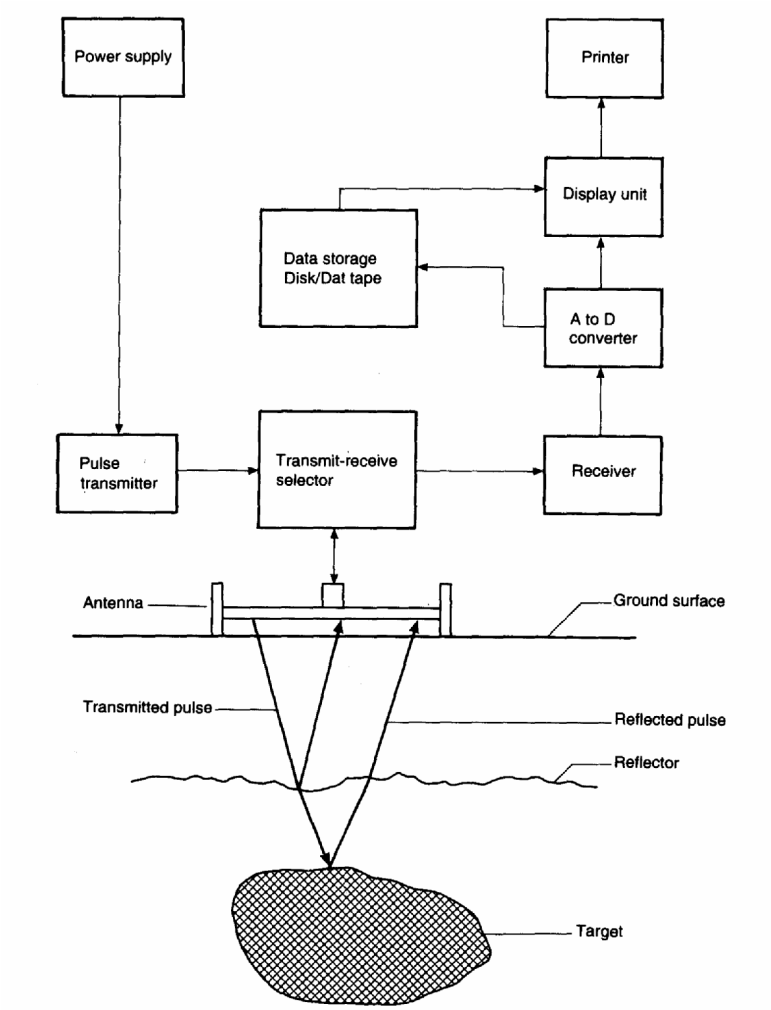


Fig. 4.18 Schematic diagram showing the operation of a ground penetrating radar (GPR) system.

Immediate on-site results may be viewed on a graphics display. Modern radar systems such as the SIR System-10 manufactured by GSSI (Geophysical Survey Systems Inc.) may be controlled from a PC and have a colour display which aids interpretation. Such systems also allow signal enhancement by algebraic addition of successive records taken at the same location. This process, known as stacking, improves the signal-to-noise ratio. It is a feature that is also found on most modern seismographs.

By moving the radar antenna over the ground surface a continuous real-time geological section (pseudo-section) is built up by arranging each radar record next to each other. A typical pseudo-section is shown in Fig. 4.19. The horizontal axis of the section represents distance and the vertical axis represents the two-way travel times of reflections in nanoseconds. In order to transform this into distance the velocity must be known. The lines shown on the pseudo-section represent reflectors and are made up from coalescing wiggle traces from individual radar records.

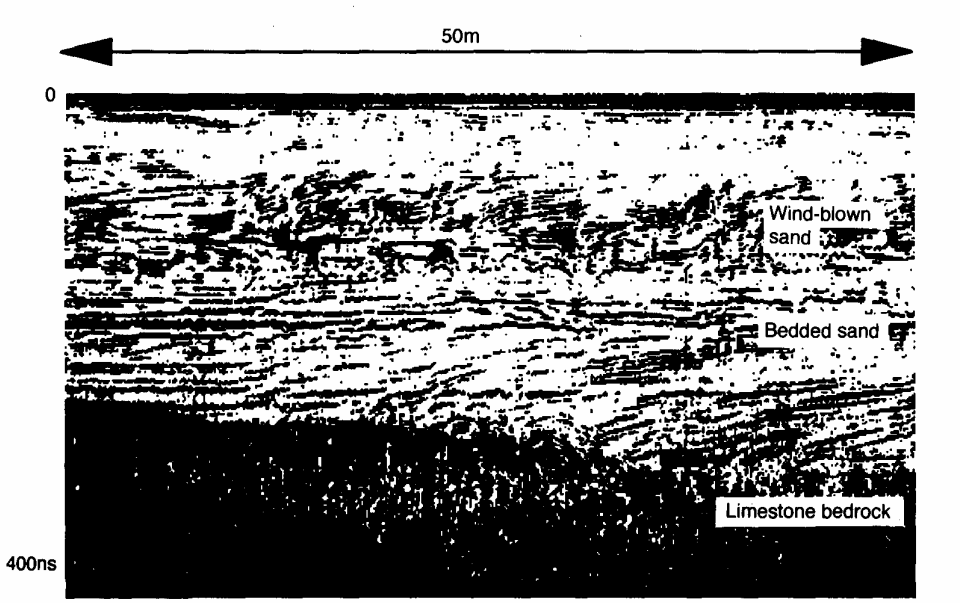


Fig. 4.19 Radar pseudo-section showing sands overlying limestone (after Ulriksen 1983).

The section shown in Fig. 4.19 depicts sands overlying limestone. The diagram shows the unique capacity of the radar to display the stratification in soils. The upper part of the section consists of wind-blown sands which are characterized by some disorder. The underlying sand is more regularly stratified indicating deposition under water. The limestone in the bottom of the section appears dark.

In order to interpret a radar pseudo-section it is necessary to know how the section is derived. Figure 4.20 illustrates schematically the typical format of a radar reflection section. The physical model used in Fig. 4.20 depicts a horizontal reflector overlying an isolated target. Such a model may be represented in the field by a cavity in, say, limestone which is overlain by soil. The transmitted pulse from the radar antenna does not travel vertically downward but spreads out in a cone. This means that the isolated target is ‘seen’ before the antenna is directly over it. A relatively small void or isolated reflector will act like a point target which produces a characteristic hyperbolic anomaly.

Measurements of travel time on the hyperbolic tails of this anomaly may be used to calculate the depth of the feature at the point when the antenna is vertically above it. Unless the buried feature is made of conducting material, such as metal the electromagnetic waves will pass

through the feature and be reflected from the bottom as well as the top as shown in Fig. 4.20. These reflections may permit the size of the buried feature to be determined. Figure 4.21 shows a radar pseudo-section of cavities in limestone.

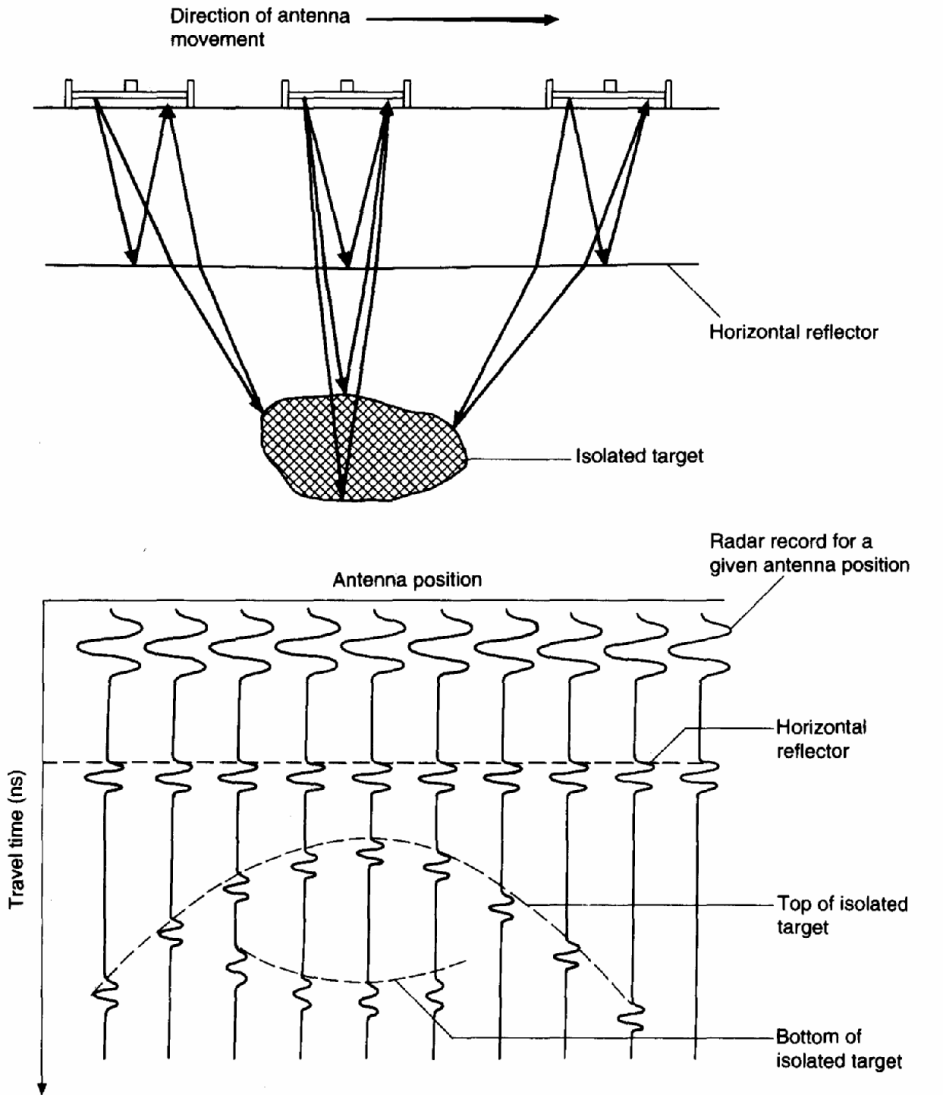


Fig. 4.20 Schematic illustration of the format of a GPR reflection section.

The horizontal reflector shown in Fig. 4.20 appears as a linear anomaly in the pseudo-section. The depth of this reflector can only be determined if the velocity of the material above it is known. Velocity analysis can be done by physically sampling the material and performing the appropriate laboratory tests. However, this approach is time consuming and expensive. One of the simplest and most common methods used for velocity determination is common-mid-point (CMP) sounding which is used in reflection seismic. There are two requirements for this method to be applicable:

1. there must be a planar reflector extended for some distance along the profile;
2. there must be two antennas, one transmitting and one receiving.

The arrangement is shown in Fig. 4.22. By moving both antennas equal distances away from the initial centre point the same reflection point will be maintained. The depth to the reflector can now be calculated using the expression:

$$z = \sqrt{\left[\frac{t_2^{\,2} x_1^{\,2} - t_1^{\,2} x_2^{\,2}}{4(t_1^{\,2} - t_2^{\,2})}\right]} \tag{4.6}$$

With the depth known, the average velocity in the material is calculated using the travel time when the antenna separation was zero.

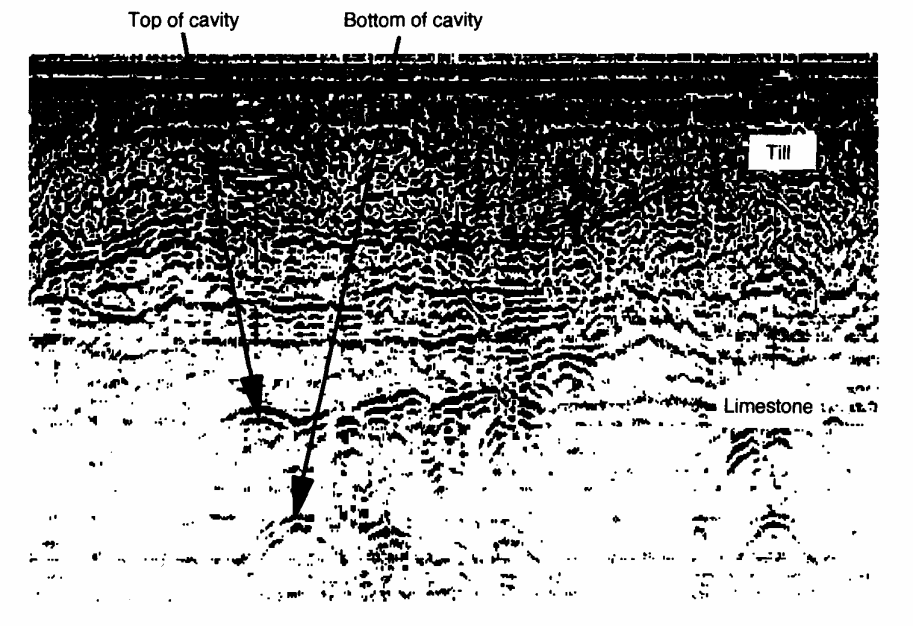


Fig. 4.21 Radar profile showing cavities in limestone (after Ulriksen 1983).

The radar pseudo-sections have a compressed x-axis, which makes them look more dramatic than the recorded structures actually are. Some inclined reflectors may be lost from the section as the dip exceeds a critical value. The maximum recordable slope is identical to the radiation angle of the antenna. This is not a fixed measure but rather a matter of detection level in the receiver. For practical purposes it may be regarded as an angle. Ulriksen (1983) found the maximum recordable slope angle for an 80 MHz antenna (used as both transmitter and receiver) in peat was 36°.

Near-surface soil may contain a great number of minor reflectors which cause scattering of the return signal and makes the sections difficult to-interpret. This problem is overcome by using two separated antennas. One antenna is used as a transmitter and the other is used as a receiver. The near-surface echoes will be returned to the transmitter rather than to the receiver antenna. Echoes from greater depths will thus be enhanced.

There are a number of similarities between the theory of electromagnetic and elastic body (seismic) wave propagation (Ursin 1983). Both radar and acoustic pulses propagate with finite velocities that depend on the material properties and each are reflected and diffracted by local changes in the medium. The dynamic behaviours are different (with regard to amplitude and dispersion), but the kinematic behaviours are the same. This similarity results from the fact that displacement currents dominate conductive currents at frequencies where GPR is effective. Under these conditions, an electromagnetic pulse propagates with virtually no dispersion and has a velocity controlled by the dielectric properties of the material alone. For equal wavelengths there is considerably less attenuation of electromagnetic energy than of mechanical energy. At low frequencies or in high conductivity environments (for example, sand saturated with saline water) where conductivity currents dominate, GPR cannot be used

effectively. In such cases electromagnetic fields diffuse into the ground and electromagnetic induction methods (for example, transient EM) are more appropriate as a means of investigation.

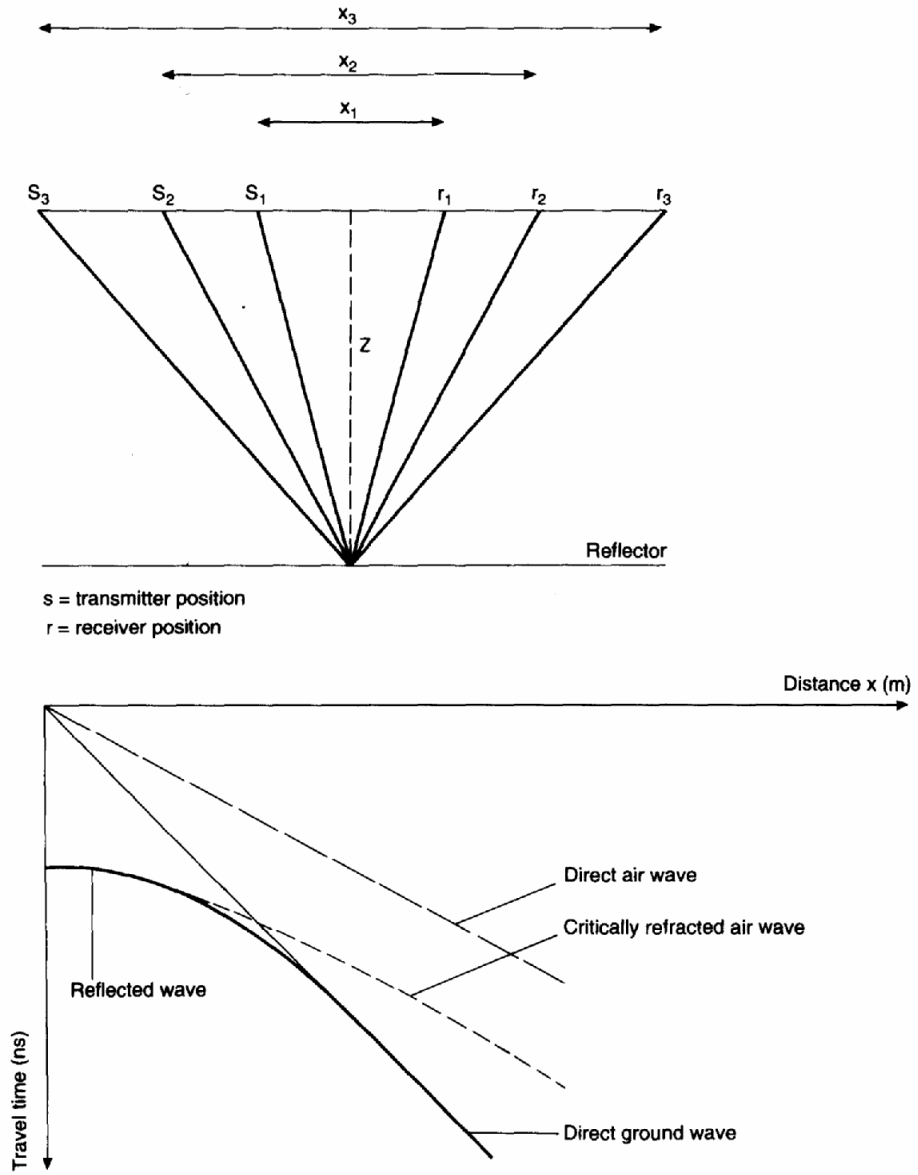


Fig. 4.22 Illustration of CMP sounding.

The similarities between radar and reflection seismics have resulted in data-processing techniques that were developed for the latter (such as convolution and migration) being used for processing radar data.
The major factors that affect the performance of ground penetrating radar are:

1. the contrast in relative permittivity (dialectric constant) between the target and the surrounding ground;
2. the conductivity of the ground;
3. the shape of the target and its orientation with respect to the radar antenna; and
4. the density of scattering bodies within the ground that produce reflection similar to those from the target.

The penetration depth of the radar energy is dependent upon the conductivity of the materials being probed which, in turn, is primarily governed by the water content and the amount of salts in solution. Typical values of conductivity are shown in Table 4.5.

Table 4.5 Approximate electromagnetic parameters of typical rocks and soils at a frequency of 100MHz (data from Morey (1974) and Ulriksen (1983))

Material	Conductivity, σ (mho/m or S/m)	Relative permittivity, K	Attenuation, α(dB/m)	Velocity, C (cm/ns)
Air	0	1	0	30
Fresh water	10^{-3}	81	0.18	3.3
Sea water	4.0	81	10^{3}	3.3
Granite (dry)	10^{-8}	5	10^{-5}	13
Granite (wet)	10^{-3}	7	0.6	11
Basalt (wet)	10^{-2}	8	5.6	11
Shale (wet)	10^{-1}	7	45	11
Sandstone(wet)	4×10^{-2}	6	24	12
Limestone (wet)	2.5×10^{-2}	8	14	11
Sandy soil (dry)	1.5×10^{-4}	3	0.14	17
Sandy soil (wet)	7×10^{-3}	25	2.3	6
Clayey soil (dry)	2.5×10^{-4}	3	0.28	17
Clayey soil (wet)	5×10^{-2}	15	20	7.8

The ability of the radar to detect a target will depend to a large extent on contrast in dielectric properties. These properties may be considered in terms of relative permittivity *(K)* which compares the dielectric constant of the ground with that of free space. Typical values of *K* are shown in Table 4.5.

Both conductivity and relative permittivity are frequency dependent. In general, the relative permittivity will decrease with increasing frequency whereas conductivity will increase. It will be seen from Table 4.5 that the range of relative permittivity is between 1 and 81 with the lowest value for air and the greatest for saline water (seawater). Most rocks and soils are made up from minerals which are essentially insulators. Thus the range of relative permittivity for dry rocks and soils are generally small (2 to 12). However the addition of water causes the relative permittivity to rise. The greatest changes are seen in soils as a result of their much greater porosity in most cases. Relative permittivity is sensitive to moisture content and pore water chemistry. Saline pore water will result in a much higher relative permittivity than fresh -pore water. Conductivity varies in a much wider range, about twenty orders of magnitude, and is sensitive to moisture content and pore water chemistry.

In assessing whether GPR is the most appropriate method for a particular investigation the following factors should be considered.

1. *Contrast in electrical properties.* A subsurface feature that displays little or no contrast in electrical properties with the surrounding ground will not be detected by GPR. The degree by which features will reflect electromagnetic waves may be estimated using the following expression for power reflectivity:

$$P_r = \left(\frac{\sqrt{K_h} - \sqrt{K_t}}{\sqrt{K_h} + \sqrt{K_t}} \right)^2 \qquad (4.7)$$

where K_h = relative permittivity of host medium, and K_t = relative permittivity of target

In order to assess whether sufficient energy will be reflected Annan and Cosway (1992) suggest the following rules-of-thumb.

- Pr>0.01.
- The ratio of target depth to the smallest lateral target dimension should not exceed 10:1.

A metal target will have a relative permittivity that approaches infinity resulting in a value of P_r which is approximately unity. The result is that very little or no electromagnetic energy will penetrate a metal object. For example, in the case of a tunnel lined with metal mesh to prevent rock falls, all the radar signal will be reflected at the tunnel wall and none would penetrate the tunnel wall. This makes it impossible to determine the size of the tunnel and map features of interest directly below it.

2. *Penetration.* A subsurface feature may display a significant contrast of electrical properties with the surrounding ground and yet remain undetected as a result of signal attenuation. This may be considered in terms of a detection range which is dependent upon the electrical properties of the ground as well as the efficiency of the antenna. Figure 4.23 shows the relationship between conductivity and effective depth of penetration for a given transmitter frequency and sensitivity of the radar set. It will be seen from Fig. 4.23 that for soils and rocks with a conductivity greater than i0 mho/m penetration will be less than 10 m, and at conductivities greater than 10^{-2} *mho/m* the penetration is negligible. In the UK most soils encountered will be clay rich and thus the effective depth of penetration of GPR may be somewhat limited.

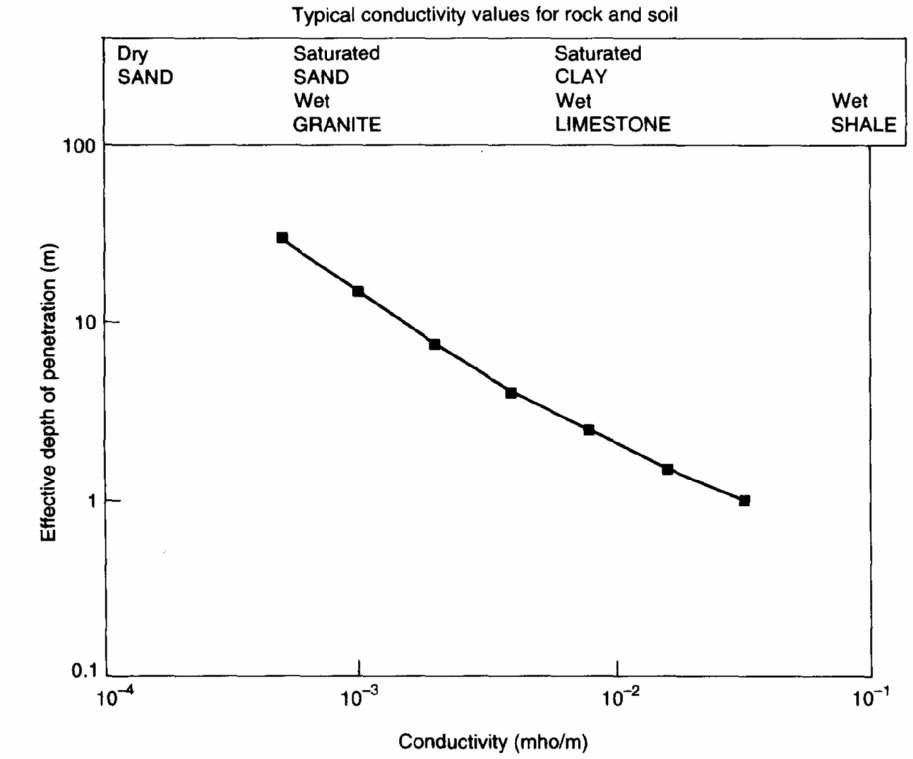


Fig. 4.23 Relationship between effective depth of penetration for GRP and ground conductivity (data from Darracott and Lake, 1981).

As a rough guide to estimating the effective penetration depth *(Lmax)* Annan and Cosway (1992) suggest the following rule-of-thumb:

$$L_{\max} < \frac{30}{\alpha} or L_{\max} < \frac{35}{\sigma} \quad (4.8)$$

where α= attenuation in *dB/m,* and σ = conductivity in mS/m.

These equations are generally applicable where attenuation is moderate to high (>0.1 dB/m) which is typical of most geological settings. The values of L_{max} vary according to whether conductivity or attenuation is used in the calculation. For example L_{max} for a wet clayey soil will be 0.7m based on conductivity, and 1.5m based on attenuation.

3. *Resolution.* The spatial resolution of GPR is related to the wavelength of the electromagnetic waves it produces. As the wavelength is increased, the resolution of the radar is reduced. Ideally the wavelength should be short in relation to the illuminated objects. The operating frequency of radar together with the electrical properties of the ground will determine the wavelength of electromagnetic waves passing through it. For example, an 80MHz antenna has a wavelength of *3.45m* in air and 0.42m in freshwater.

   For a given transmitter power rating the depth of penetration will be influenced by the operating frequency of the radar. The depth of penetration may be increased by reducing the operational frequency at the expense of resolution. Annan and Cosway (1992) provide a simple guide for determining penetration depth for a given frequency based on the assumption that the spatial resolution required is about *25%* of the target depth. This guide is shown in Table 4.6.

   In cases where maximum resolution is required near the ground surface as well as at depth it will be necessary to carry out separate surveys using antennas with different operating frequencies.
4. *Signal-to-noise ratio.* The GPR method is sensitive to the environment in which the survey is carried out. Metal structures, radio transmitters, overhead or underground electrical power lines may all produce sufficient noise to saturate a sensitive receiver.

Table 4.6 Guide for determining penetration depth for a given frequency

Depth (m)	Centre frequency (MHz)
0.5	1000
1.0	500
2.0	200
5.0	100
10.0	50
30.0	25
50.0	10

In general, GPR methods can be used more cost effectively in situations when searching for a known target (for example, buried pipe, subsurface cavities, rock ledge, buried valley, etc.) rather than as a general reconnaissance tool, where broad conclusions about subsurface conditions are sought. A comprehensive review of the application of ground penetrating radar to civil engineering is given by Ulriksen (1983). Case histories illustrating the use of GPR in the UK are given by Darracot and Lake (1981) and Leggo and Leech (1983).

Seismic reflection

Seismic reflection methods have been used on offshore site investigations since the 1950s. Traditionally this method has not been used for conventional, onshore investigations because of the relatively low cost of boreholes compared with those offshore, and because the method has in the past been difficult to apply to shallow investigations. Recent research has extended the method to shallow depths, but it has yet to find application in onshore civil engineering ground investigations.

The seismic reflection method is used in engineering investigations primarily for accurate profiling of geological structures. The method has the unique advantage of permitting mapping of many horizons from a single shot, and the precision of mapping does not decrease significantly with depth in favourable conditions. This is not the case with other geophysical methods. Both quantitative and qualitative interpretations of reflection data are possible.

The seismic reflection method relies on measuring travel times of P waves which have been reflected back to the surface by boundaries separating materials with different characteristic P-wave velocities.

The proportion of energy reflected by a velocity interface is defined by the reflection coefficient of that interface. According to Dohr (1975) the reflection coefficient of an interface for a normally incident (i.e. the angle of incidence is zero) wave is:

$$R_{0,1} = \frac{A_r}{A_d} = \frac{\rho_1 V_1 - \rho_0 V_0}{\rho_1 V_1 + \rho_0 V_0} \tag{4.9}$$

where A_r = amplitude of the reflected wave, A_d = amplitude of the normally incident wave, ρ_o = density of upper layer, ρ_1 = density of lower layer, V_0= P-wave velocity of upper layer, and V_1 = P-wave velocity of lower layer.

Clearly the greater the reflection coefficient, the stronger the reflections are from the interface and hence more easily identified from the seismic record. As the density of rocks and soils is not very variable, velocity contrast must be the controlling factor with respect to the amplitude of reflected events. Telford *et al.* (1990) give the following values of ***R:***

Interface	***R***
Sandstone on limestone	0.040
Soft ocean bottom	0.11
Base of weathering	0.46

Velocity inversions give rise to a negative reflection coefficient. This means that reflected waves will be 1800 out of phase with the incident waves. Complete phase reversals of this kind can be detected on the seismic record. Theoretically, therefore, it is possible to overcome the blind-zone problem in refraction surveying by careful examination of reflected events on the seismic record. In practice, however, such events may be difficult to identify at shallow depths.

Interpretation of seismic reflection data can lead to accurate determinations of the depth and dip of the strata identified from the seismic record. This is only possible if the P-wave velocity of each layer is known. The P-wave velocity for each layer encountered may be determined by borehole methods or by analytical methods using travel-time data from the survey itself.

The most common use of the seismic reflection method in engineering is in site investigations over water, particularly offshore investigations (for example, investigations for offshore platforms for the oil industry). The technique commonly used in such investigations is continuous seismic (reflection) profiling. The continuous seismic profiling method (CSP) is merely an extension of echo sounding which is used by most sea-going vessels to profile the sea bottom. The method involves transmitting a brief acoustic pulse from just below the water surface and detecting the reflected pulses with a pressure-sensitive detector (hydrophone) which is mounted near the energy source. The signal from the hydrophone is amplified and presented in a graphic form by a suitable recording device. The vessel beneath which the energy source and detector are mounted moves along a set traverse line while the acoustic pulses are generated. The result is that the reflection data are presented in a graphical format representing a real time geological section (Fig. 4.24). In general, the depth of penetration of the seismic (acoustic) pulse into the sea bed is inversely proportional to the frequency of the pulse. This has given rise to a number of CSP systems designed for a whole range of requirements, from deep investigations for offshore tunnel projects, to detailed shallow investigations for submarine pipelines. Common CSP energy sources include the following.

1. *Sparkers*. Shock waves are produced by the explosive formation of steam bubbles, resulting from electrical discharge between two electrodes.
2. *Boomers*. Shock waves are produced by explosive repulsion of a metal plate, spring loaded against the force of an insulating coil. The repulsion of the plate is triggered by passing a high voltage through the coil.
3. *Pingers*. These produce an acoustic pulse using piezoelectric or magnetostrictive transducers.
4. *Air guns*. Shock waves are produced by explosive release of high pressure air from an immersed pressure chamber.

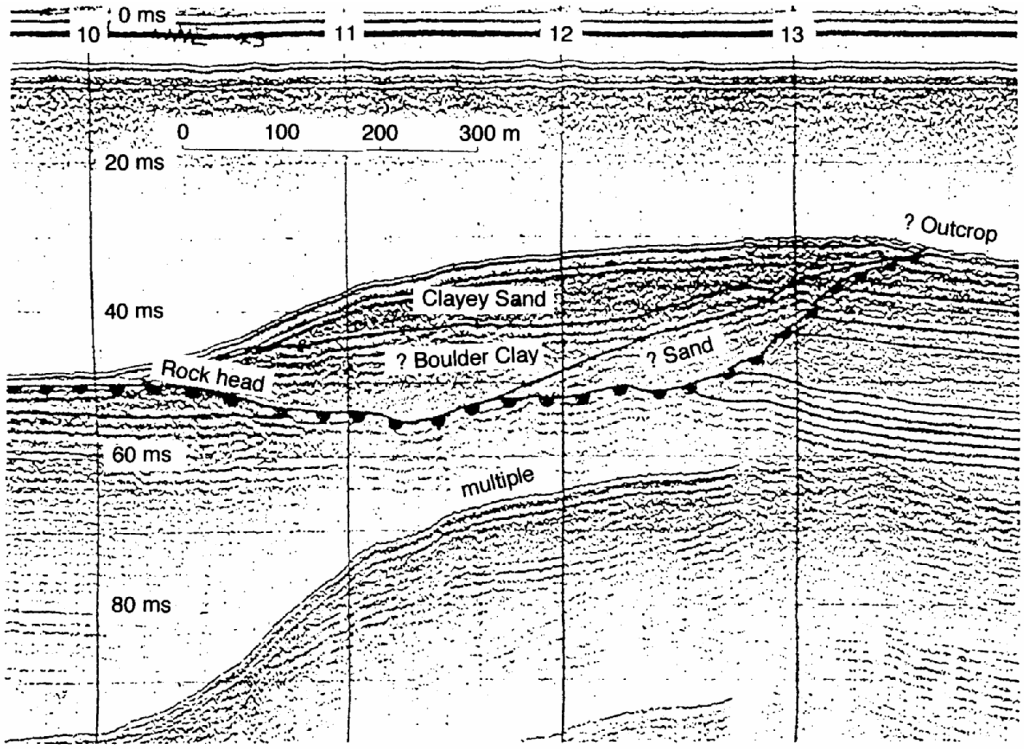


Fig. 4.24 Continuous seismic profiling record (McCann and Taylor-Smith 1973).

Table 4.7 gives details of the currently available energy emission systems used in CSP work, and the type of survey for which each is best suited.

Table 4.7 Continuous seismic profiling systems currently available and their characteristics

Description	Type of survey	Frequency band (Hz)	Pulse length (ms)	Comments
Sparker	Deep reflection (and shallow reflection through gravels and boulder clay)	50-000 Light duty sparker 50-0000	8-2 3-5	Penetration good. Useful in investigation of bedrock structure. Unsuitable for detailed shallow sub- bottom studies. However, a light duty sparker has been used with success in a wide variety of shallow surveys, including harbour and pipeline route surveys. Equipment is compact, and requires only a small power supply.
Boomer	Deep reflection	200-3000 400-14 000*	3-5	Depth resolution similar to that of the light duty sparker, but penetration is superior. Boomers have been used for similar types of investigation as sparkers but are preferred for water depths >80-l00m. Disadvantages: heavy and cumbersome and require large power supply.
Pinger	Shallow reflection (penetration through clayey soil only)	1000-12000	0.2-1	Depth resolution excellent, but penetration varies in the inverse ratio to the sea bed sediment grain size.
Side scan sonar	Sea floor profiling	48000-105000		Resolution good, equipment compact and requires small power supply.
Conventional echo sounding	Sea floor profiling	30 000-40 000		As above.
Air gun	Deep and shallow reflection	2-25000	1-4	Capable of deep penetration in shallow to moderate depths of water. Moderate penetration in deep water.

* Precision boomer.

The success of both land-based and marine seismic reflection methods depends on the velocity contrast between different materials. If the velocity contrast is low, then most of the energy will be refracted, and hence the reflected energy will be small. In general only about 1% of the incident energy is reflected back, except in exceptional circumstances, such as reflections from the sea floor where the reflection coefficient is normally very high. One advantage of reflection methods over refraction methods is that interfaces with low velocity contrasts arc more readily identifiable by increasing the amount of energy put into the ground. This may, however, create problems with unwanted multiple reflections from strong reflectors.

Seismic reflection methods are used mainly to profile subsurface or sub-bottom (in marine investigations) layered systems. Current research, however, shows that the data acquired by

such methods are capable of a more detailed interpretation to obtain geotechnical parameters. McCann and Taylor-Smith (1973) review the application of seismic reflection together with other geophysical techniques in the determination of geotechnical parameters of sea-floor sediments.

Seismic tomography

Tomography (from the Greek, tomos: a slice or section) is a method whereby an image of the distribution of a physical property within an enclosed region is produced without direct access to the zone. Seismic velocity tomography is a geophysical sectioning technique which determines the spatial seismic velocity distribution within a given area of interest.

Seismic velocity tomography is potentially extremely useful in geotechnical ground investigations, for two reasons. First, tomography can give information on the general variability (i.e. of seismic velocity) beneath a site, and by inference allow a qualitative assessment of the variability of properties such as stiffness and strength. Features such as voids, fractures, rock layers and soft spots are often difficult to detect using more conventional (boring and drilling) techniques, because only a minute proportion of the sub-soil is sampled and tested by direct methods of ground investigation. Geophysical techniques may be helpful in detecting such features.

Secondly, the technique can be used to provide values of the 'very small strain' stiffness (G_o, or G_{max}) of the ground, since this is uniquely linked to seismic shear wave velocity (V_s) through the equation:

$$G_\circ = V_s^{\,2}\rho \tag{4.10}$$

where ρ = bulk density of the soil. Depending upon the ground conditions and the strain levels imposed by construction, this stiffness may be more or less relevant to the operational stiffness required for the analysis of geotechnical displacement problems. In many ground conditions (for example fractured rock or granular soils) the alternative methods of estimating stiffness may be sufficiently expensive or inaccurate as to make seismic methods attractive.

The form of tomography that is, perhaps, most familiar is the technique of computer assisted tomography (CAT), as used in diagnostic medicine. The method of seismic tomography has been in use for some years: amongst the first applications of geotomography was a survey by Bois *et al*. (1971) in an oil field. In seismic velocity tomography, an image of the distribution of the seismic propagation velocity properties within a region of the ground is deduced from measurements of the transit times of artificially induced seismic waves crossing the zone. The process can be divided into a number of activities, each of which requires careful attention if useful results are to be achieved:

- data acquisition and reduction;
- reconstruction of velocities; and
- interpretation.

The field geophysical method involves the input of seismic energy (from mechanical hammers, or sparkers) at discrete points down boreholes, and possibly along the ground surface, and the acquisition of seismic records of the incoming wave energy (via geophones or hydrophones) at as many other discrete points around the area of interest as is feasible. The seismic traces are then inspected to determine the first arrival of the wave type of interest. In many rock investigations this may be the onset of primary, compressional (P) waves, but in weaker sediments such as saturated soils, it may be necessary to determine the arrival time of secondary, shear (S) waves. The data required from the field, before reconstruction processing can commence, are the travel times from each source to every receiver, and the co-ordinates of the source and receiver positions.

Tomographic reconstruction (as applied here) is the mathematical process by which velocities at different points within the ground are calculated. The principles of tomographic reconstruction are well-established (Radon 1917). In general, the region to be imaged is divided into discrete rectangular areas or cells (Fig. 4.25). Let υ_j be the seismic propagation velocity within the *jth* of *n* reconstruction cells. This velocity value applies uniformly across the full area of the cell. The travel time, t_i for the *ith* ray across the grid of cells is given by:

$$t_i = \sum_{j=1}^{n} \left(\frac{d_{ij}}{v_j} \right) \tag{4.11}$$

where d_{ij} = length of the *ith* ray within the *jth* cell. In a survey which incorporates *m* travel times, acquired for rays at various positions and orientations across the region, there will be *m* such summations. These can be expressed in matrix form as:

$$t = Dw \tag{4.12}$$

where **D** is an $m \times n$ array having elements of the form d_{ij}; $\boldsymbol{w}$ is an *n*-element column vector containing the current estimate of the reciprocal velocities ('slownesses'); and $\boldsymbol{t}$ is a column vector of the m travel times calculated across the discretized velocity field.

Let $\boldsymbol{p}$ be the column vector of the *m* observed travel times from a field survey. For reasons to be discussed in detail later, it is usually not possible to find a velocity field which is exactly consistent with the measured travel times. Thus the process of reconstructing a velocity field is equivalent to the minimization of a residual vector, e, defined by the equation:

$$e = p - Dw \tag{4.13}$$

Commonly used strategies for computerized geotomographic reconstruction include the damped least squares method (Bois et at. 1971), the Back Projection Technique, BPT (Kuhl and Edwards 1963; Neumann-Denzau 1984), the Algebraic Reconstruction Technique, ART (Kaczmarz 1934; Peterson *et al.* 1985), and the Simultaneous Iterative Reconstruction Technique, SIRT (Dines and Lytle 1979; Gilbert 1972), although many others are available (for example, Gordon and Herman (1974)).

The acquisition diagram in Fig. 4.25 shows straight rays. In practice, rays may deviate from straight paths as waves undergo, for example, refraction in an heterogeneous velocity field. Therefore the path followed by a ray, between a source and receiver, is not known. D is a function of w: the elements of w and D are unknown. Thus reconstruction involves not only the distribution of errors along ray paths, but also the determination of an appropriate position for that ray path.

The product of reconstruction is a single velocity for each cell. These velocities may be contoured and displayed as colour or grey scales in a tomogram. A seismic velocity tomogram is, necessarily, an approximation. It is a two-dimensional, discrete estimate of a continuously varying, three-dimensional function — that is, the distribution of the seismic velocity properties of the ground. The accuracy of a tomographic estimate of the subsurface and, hence, its usefulness as a predictive tool is influenced by many diverse factors. These affect the ability of the technique to determine the size, form and seismic velocity (and hence stiffness) of features in the ground.

Idealizations of wave behaviour

Seismic energy travels through the ground as waves. There are a number of different physical theories associated with the description of wave behaviour. Each particular theory or idealization may be useful in one context, but it is often necessary to appeal to more than one theory to understand fully how seismic energy can be transmitted through the ground. For example, a source of seismic energy at a point in a homogeneous isotropic medium will produce a set of spherical wavefronts and these can, as a simplification, be represented as a set of rays emanating from the source (Fig. 4.26a). The ray approximation is convenient for

use in geotomographic reconstruction because of the line integral relation between propagation velocity and travel time assumed in eqn 4.11. According to Huygen (who first put forward a wave theory for light), each point on a wavefront can be regarded as a possible secondary source. This concept leads directly to Fermat's principle of stationary travel times, which identifies the ray path as that giving the minimum travel time. A corollary of Fermat's principle is Snell's law of refraction, which governs the deviation of a ray at a velocity interface. Snell's law is restrictive because it only permits energy propagation normal to a wavefront. Ray paths for reconstruction determined using Fermat's principle will allow for reflection, refraction, diffraction, and also head waves.

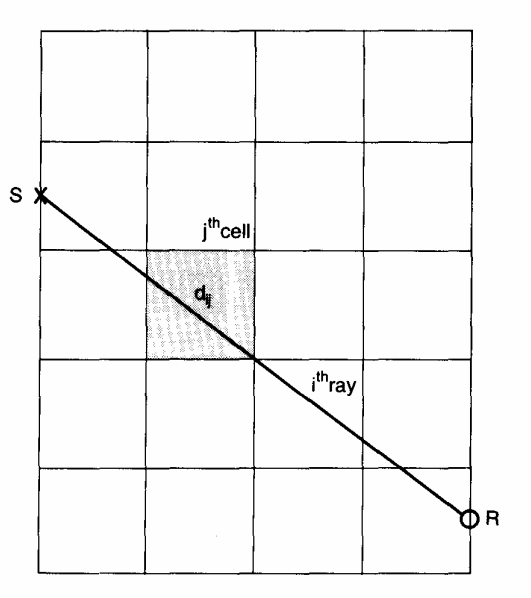


Fig. 4.25 Definition of notation for reconstruction.

Figure 4.25, with its straight ray paths, gives an oversimplified picture of energy passing from seismic source to receiver. In situations where there is not much variation in seismic velocity in the region of interest, the straight ray assumption may be reasonable. ISRM (1988) suggests that this is the case when velocity contrasts are less than 20%.

When there are greater velocity contrasts, straight-ray reconstruction may lead to unacceptably inaccurate tomograms. Wave energy may be refracted (Fig. 4.26b) or diffracted (Fig. 4.26c) around obstacles, and head waves may form along velocity interfaces (Fig. 4.26d). Attempts to reconstruct the distribution of seismic velocities will be doomed to failure if the mathematical model being used assumes that rays can only bend according to Snell's Law, but diffraction or head waves dominate.

To illustrate these points, consider the two tomographic surveys outlined below. The first was designed to locate a tunnel crossing the tomographic plane. If ray paths are assumed to be those permitted by Snellian refraction then there should be receivers which are unable to detect any energy emitted by the source (Fig. 4.27a). In practice, all receivers detected incoming wave energy, albeit at various reduced amplitudes. The presence of the water table produced a higher velocity layer a short distance below the tunnel and gave rise to head waves which provided a route around the void (Fig. 4.27b). Even without diffraction and head waves, if refraction is invoked then many rays will travel around the anomaly (Fig. 4.27c). As might be expected from the discussion above, a P-wave survey failed to show any signs of the tunnel (Fig. 4.27d).

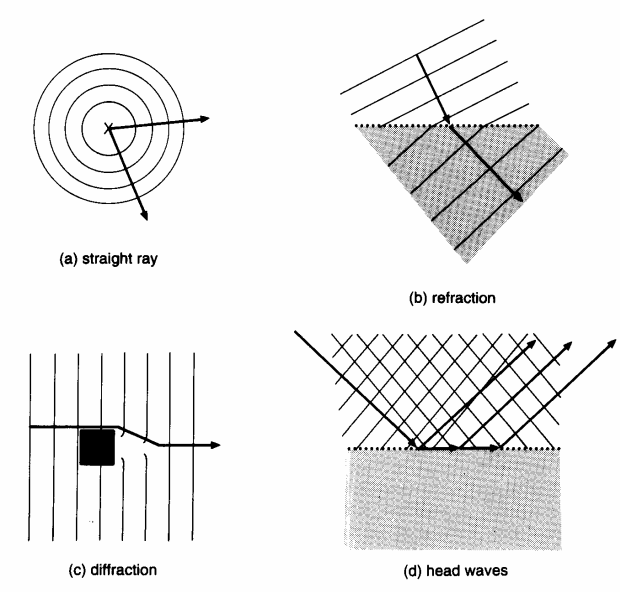


Fig. 4.26 Wave propagation.

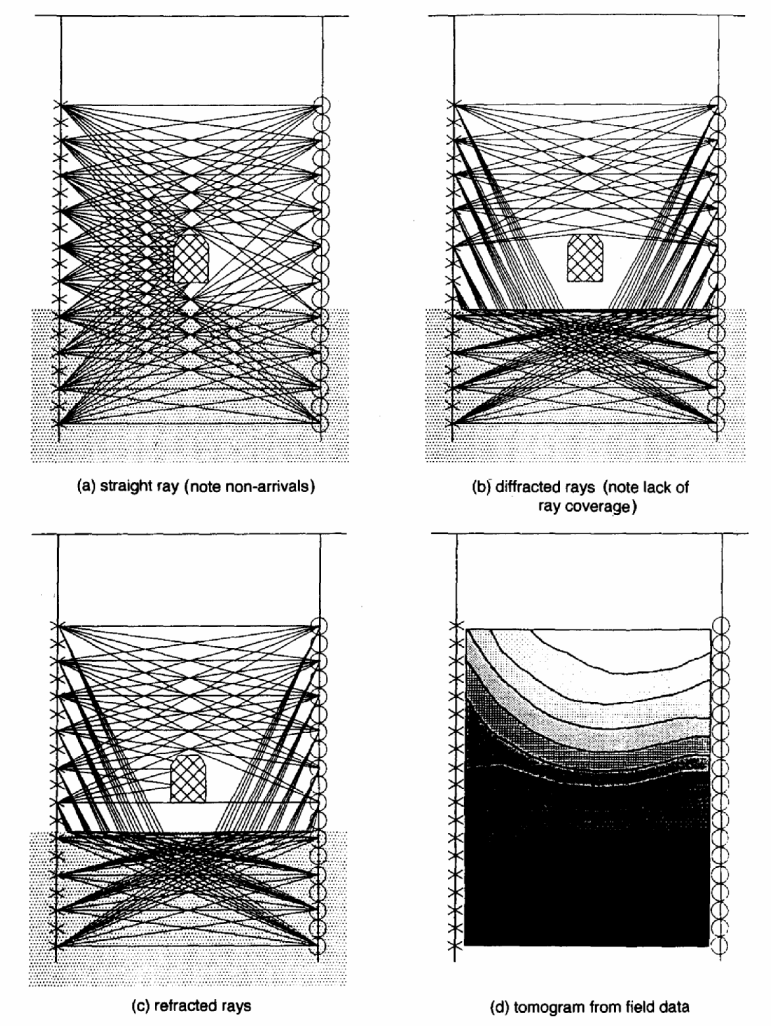


Fig. 4.27 Unsuccessful tunnel location.

An idealization of tunnel geometry is shown in Fig. 4.28. Simple mathematics can demonstrate that for any given size ratio (a/L) there is a critical velocity ratio (V_2/V_1) below which an increase in velocity contrast has no effect on travel time. At lower velocity ratios the velocity (and hence stiffness) of the inclusion (V_2) cannot be recovered correctly during reconstruction, because first arrival energy does not travel through the lower velocity material. For the simple geometry shown, it is obvious that the maximum effect that a low-velocity inclusion may have is to double the travel time. When the inclusion occupies as much as 30% of the distance between the source and receiver, the travel time will increase by no more than 6% of the value when no low-velocity inclusion is present, regardless of the velocity contrast. The detectability of high-velocity inclusions will not he influenced by this effect.

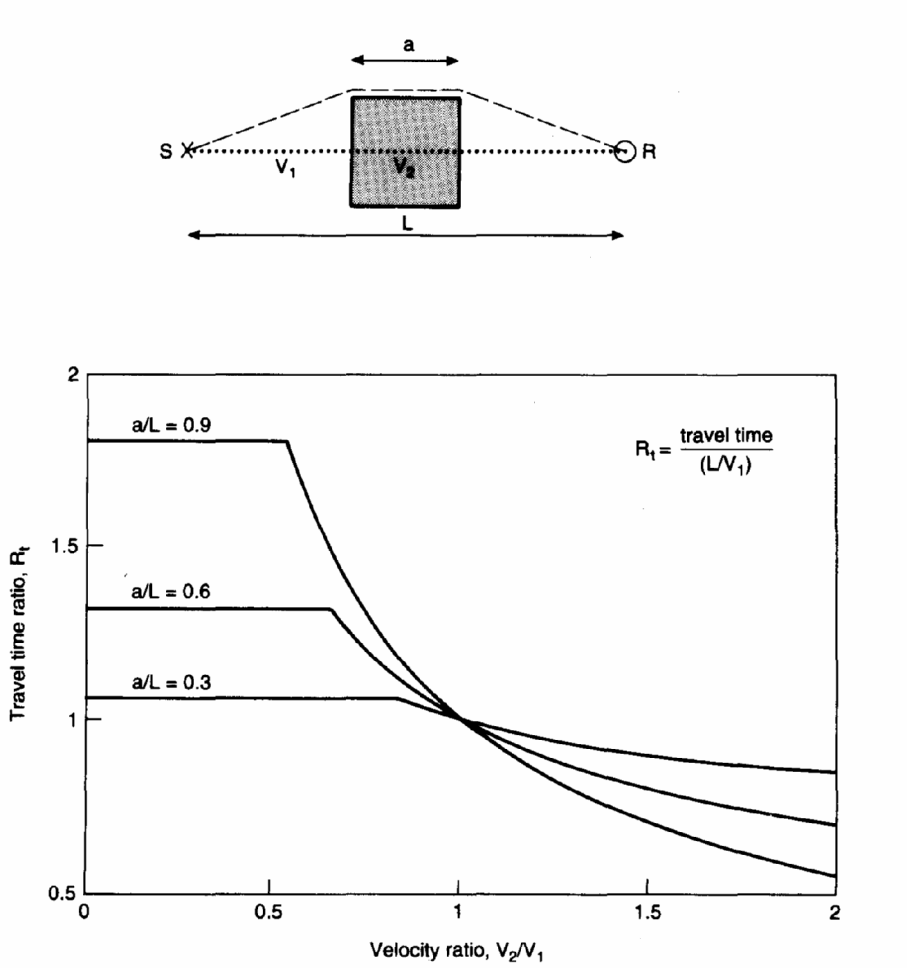


Fig. 4.28 Normalized travel time ratio as a function of velocity contrast.

As a second example of problematic wave phenomena, Fig. 4.29 shows two adjacent shear-wave tomograms that were obtained from the London clay. The diagonal features which appear in these images result from picking first-arrival shear-wave energy arising from tube waves propagating in the water-filled source boreholes. It is emphasized that these are not a feature of the ground. Virtually identical features can be synthesized by assuming that a compressive wave propagates down the hole in the borehole fluid, generating a shear wave upon its impact with the base.

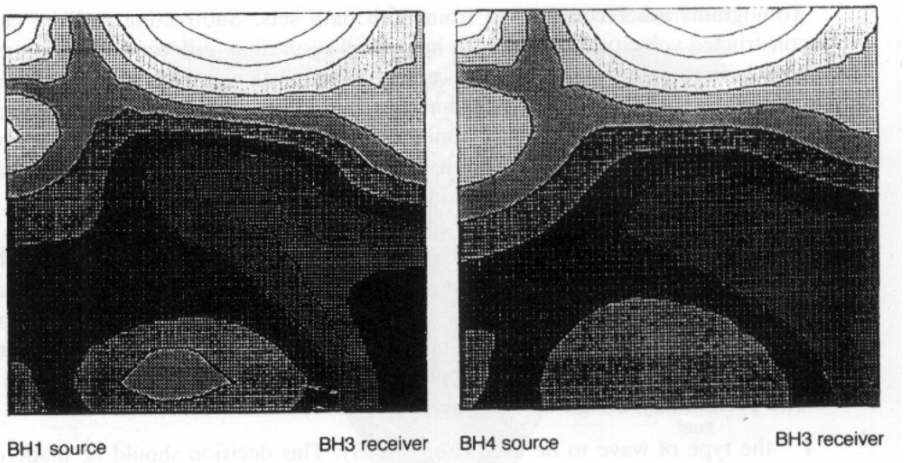


Fig. 4.29 Atefacts produced by tube-wave propagation.

Finally, there are other aspects of wave behaviour that are important. For example, most tomographic reconstruction is based on the assumption of plane structures in the ground. It should always be remembered that seismic waves are three-dimensional in form and that first arriving seismic events may have followed paths outside the plane of the boreholes. In addition, features in the ground which are smaller in dimension than about one wavelength of the signal will not be capable of being resolved by tomographic reconstruction. In our experience, P waves typically will be expected to have wavelengths of between 1 and 5 m, while for S waves the wavelength is of the order of 1 to 3m.

Influence of data errors

Data errors can arise because of inaccurate seismograph triggering, as a result of mis-picking travel times, and also from locating the seismic source and receiver stations incorrectly. The latter class of error will usually be negligible in cases where a borehole deviation survey is available.

Any estimate of travel time between source and receiver will involve some level of error. This error is a function of the accuracy with which the source trigger time is known, and. the certainty with which first arrival events can be identified on the seismic records. The latter is largely a function of signal-to-noise ratio.

The influence of observational errors in the travel time data set, $\boldsymbol{p}$, on the spatial resolution of a survey can be quantified as follows. Let t_{err} be the empirically determined uncertainty on these data. An inclusion would be imperceptible if the increase in travel time due to its presence were less than the travel time error. Thus the following condition must be satisfied:

$$\frac{t_{err}}{t} < R_t \tag{4.14}$$

where t is the observed travel time for the wave and R_t is the ratio of travel times for the feature, for example, as defined in Fig. 4.28. R_t is a function of the geometry of the problem and the velocity ratio; values for a simple square inclusion can be obtained from Fig. 4.28. If eqn 4.14 is not satisfied, data errors will effectively mask the presence of the feature. In practice, estimates of t_{err} can be based on previous field experience, allowing for signal degradation caused by ambient noise levels and attenuation due to increased borehole spacing. For example, in a survey across a 15 m span of London clay, the authors have observed travel time and errors of the order of 10 ±0.2ms (t_{err}/t=2%) and 75 ±0.8ms (t_{err}/t=1%) for P and S waves, respectively. These values suggest that, to be detectable, a void must have dimensions greater than about 13—18% of the borehole spacing.

As has been indicated, in a typical geotomographic survey, the matrix **D** does not have full rank. Furthermore, **D** is ill-conditioned. Ill-conditioned systems tend to magnify the effects of data errors (Jackson 1972). This problem can be reduced by identifying and excluding outliers from the data set and also by 'damping out' the influence of the smaller eigenvalues in the tomographic system. Nevertheless, to some degree, observational errors will, unavoidably, affect a tomogram that is derived from field data. An effective empirical method of assessing the influence of travel time errors on a reconstructed image is given by the following procedure. A numerical model of the suspected velocity field is simulated. Using a suitable ray-tracing algorithm, a set of travel times for theoretical rays across the field are calculated. A second data set is generated by adding random 'error' values, limited in magnitude by terr, to the travel times in the first set.

Tomograms are reconstructed from both data sets. Subtracting, cell by cell, the reconstructed velocity values in each image will result in a 'difference' tomogram. This image indicates the effect of data errors in a particular tomographic system, as processed by the chosen reconstruction algorithm. The image can be interpreted thus:
if cell velocities in the difference tomogram show a deviation of, say, 5% from the velocities within adjacent cells, then such fluctuations in a field tomogram should, perhaps, be attributed to observational error rather than genuine variations in the properties of the surveyed region.

As with many new and complex techniques, there is a danger that tomography will become discredited as a result of thoughtless misuse. As we have demonstrated above, there are potentially many reasons why the technique might be expected not to succeed, given particular site conditions. If tomography is to be successful then a number of basic criteria should be applied during the planning and design of the work.

The preliminary design of a tomographic survey should consider:

1. the type of wave to be used (i.e. P or S). This decision should be made on the basis of the expected velocity contrasts in each case, and the predicted wavelength in relation to any target;
2. the expected signal-to-noise ratio, and the repeatability with which the seismograph can be triggered by the source; these will influence the travel time errors; and
3. the size and geometry of the required tomogram, based on the amount of ground to be investigated, and the expected size of the target.

It is essential that shallow tomographic surveys are planned on the basis of a reasonable knowledge of the likely range of ground stiffnesses (and hence seismic velocities), and the information required from the survey. If tomography is intended to detect a 'target' (for example, a fault or a cavity) then the possible orientations and sizes of the target should be estimated. Following this, synthetic travel times should be generated for a number of possible survey geometries (i.e. borehole separation, and the down-hole source and receiver spacings) and processed using a range of reconstruction techniques. In this way, the design of the survey may be optimized and the viability of the technique assessed.

On the basis of our experience to date, the following may be stated.

1. Low-velocity anomalies (such as cavities) appear more difficult to detect than high-velocity anomalies (for example, hard inclusions).
2. The absolute values of seismic velocity recovered from a survey may be regarded as indicative of ground stiffness variations, but should not be used in an absolute way in engineering calculations.
3. Velocity variations are likely to be most reliably reconstructed when velocity contrasts are low.

4. Planar, or approximately planar features (such as faults), can relatively successfully be located, provided that they strike normal to the tomographic plane. An example of the successful reconstruction of real data, over a fault in an oil storage cavern floor, is shown in Fig. 4.30.

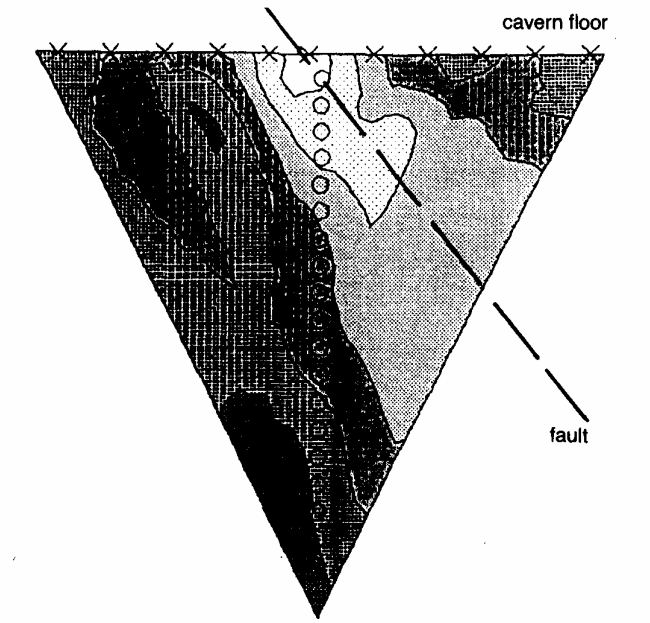


Fig. 4.30 Successful application of geotomography: detection of a fault in weak rock.

Seismic tomographic surveys

Tmographic surveys are commonly carried out between two boreholes. In the simplest case an energy source is placed in one borehole (transmitter borehole) and an array of receivers placed in the second borehole (receiver borehole) as shown in Fig. 4.31. The survey is performed by keeping the energy source stationary in the transmitter borehole and moving the receiver array up or down the receiver borehole taking a record for each receiver array position. The energy source is then moved to a new position and the process of moving the receiver array is repeated. When the energy source has traversed the section of interest the survey is complete.

In cases where the energy source used may give rise to errors in travel time determination because of triggering problems the survey may be carried out using two receiver boreholes.

In some cases only one borehole is used. Here the energy source is placed on the surface at different distances from the borehole and the receiver array is run in the borehole. This arrangement has proved very useful on restricted sites or in cases where tomography was not considered in the original design of the site investigation. The tomogram produced by this arrangement has a characteristic triangular shape. Figure 4.30 is an example of a tomogram produced using this arrangement.

Borehole preparation

When carrying out seismic tomographic surveys with either P waves or S waves the boreholes must be lined with plastic (ABS) casing. The use of plastic casing is critical to the success of the survey because metal casing presents significant velocity contrast with the surrounding ground. The casing is capped at the base and inserted into a grout-filled borehole. The grout is

necessary to provide acoustic coupling between the borehole and the ground. A bentonite or cement-bentonite grout is generally suitable.

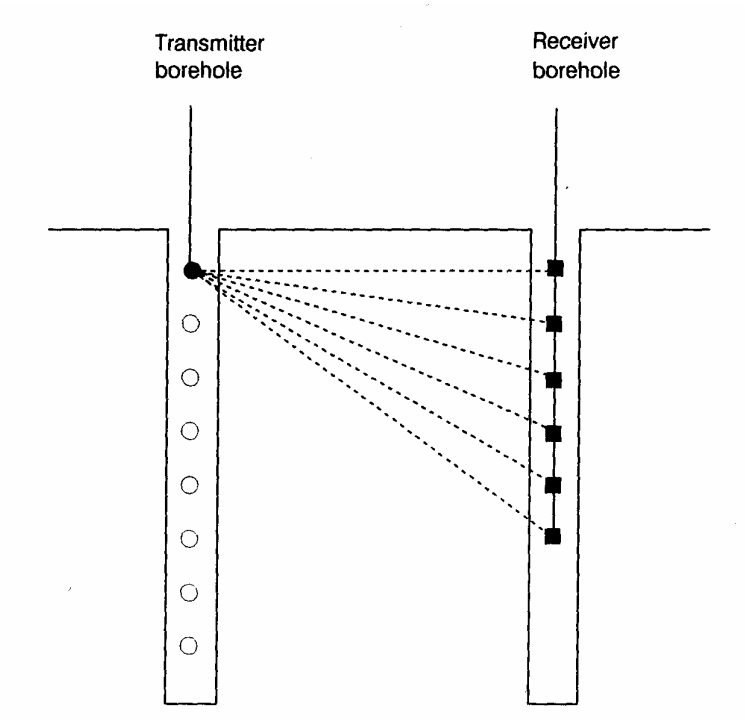


Fig. 4.31 Acquisition geometry for a typical seismic tomographic survey.

In order to minimize errors in tomographic reconstruction it is necessary to define with reasonable accuracy the three-dimensional geometry of the survey section. In order to achieve this borehole verticality surveys must be carried out as part of the tomographic survey.

Equipment and field techniques

The basic equipment required for most tomographic surveys includes:

- energy source (P wave or S wave);
- receiver array;
- compressed air for use with clamped sources and receivers;
- winches for lowering tools down the boreholes; and
- seismograph.

The equipment in terms of energy source and receivers will depend to a large extent upon the type of seismic wave being employed for the survey. Table 4.8 outlines the equipment commonly used for P- and S-wave surveys.

The borehole sparker operates in a similar manner to that used for continuous seismic profiling over water (see Table 4.7). An electrical discharge of some 4kV is generated at a known depth in the transmitter borehole. With such high voltages being used it is important that site personnel are kept clear of the high tension cables crossing the site, and that only suitably trained personnel are permitted to operate the equipment. The signal produced by the sparker has a sharp leading edge and is highly repeatable which aids picking travel times. Since the energy source is electrically operated, an electrical signal can be sent to trigger the seismograph the same instant as the electrical discharge occurs in the borehole. This means

that timing errors associated with triggering are almost eliminated and surveys may be carried out using only two boreholes.

Table 4.8 Equipment commonly used for P- and S-wave cross-hole tomographic surveys

Type of seismic wave	Energy source	Type of receiver	Number of boreholes required	Comments
P	Borehole sparker (freely suspended)	Hydrophone (freely suspended)	2	Borehole sparker is a very repeatable energy source with negligible trigger timing errors.
S	Clamped shear wave hammer	Clamped three-component geophones	3	For most commercially available mechanically operated shear wave hammers two receiver boreholes are necessary for accurate travel time measurements.

When carrying out P-wave surveys using a borehole sparker it is common practice to use an array of hydrophones in a water-filled receiver borehole. Because shear waves cannot travel through water, only P waves will be received at the hydrophones. The number of hydrophones in the array will depend upon the number of channels available on the seismograph. Typically arrays will comprise 10 or 20 hydrophones. The spacing between hydrophones may be changed easily on site, and because the array is freely suspended within the borehole moving it from one location to the next may be done very rapidly.

Shear waves are generally generated using a mechanically operated hammer striking an anvil which is clamped to the borehole wall. A typical hammer used for generating vertically polarized shear waves in boreholes is the Bison hammer shown in Fig. 4.32. Coupling between the hammer and the ground is provided by clamping the anvil to the borehole casing. The clamps are operated pneumatically and retracted using springs. As a result of the mechanical operation of the hammer, the seismograph must be triggered by placing a piezoelectric transducer or a hydrophone in the transmitter borehole close to the hammer. The triggering system often produces significant timing errors. In order to avoid such timing errors it is common practice to use two receiver boreholes such that travel times are measured between them rather than between transmitter and receiver.

The receiver array for shear wave surveys comprises a series of three-component geophones. A three-component geophone consists of three geophones orientated in three mutually perpendicular directions (one vertical and two horizontal). The operation of geophones is described later. A typical receiver array includes four three-component geophones each of which is clamped independently to the casing using the same system as for the hammer. In order to ensure that the alignment of each set of geophones remains fixed within the borehole the array is contained within a flexible (normally rubber) tube. The tube is lowered down the borehole using a set of rods so that the alignment may be controlled such that one set faces in the optimum direction to maximize the amplitude of the incoming signal. This arrangement makes it impossible to change the spacing of the receivers on site.

The borehole spacing and measurement interval used in tomographic surveys are similar to those used in conventional cross-hole seismic surveys which are discussed later.
The time taken to conduct a tomographic survey will depend upon:

1. the length of the section being surveyed;
2. the station interval;

3. the number of receivers in the array;
4. whether the source and receiver array are clamped or freely suspended; and
5. the amount of stacking required to reduce the signal-to-noise ratio to an acceptable level.

A 40 m section may be surveyed in a day using a borehole sparker and ten hydrophones with a station interval (i.e. transmitter interval and hydrophone spacing) of 2 m and between 4 and 8 stacks per record. The same section may take at least two days using a clamped source and receiver.

The field work required for even a minor tomographic survey is more than that required for most conventional cross-hole or surface seismic surveys. However, the major cost in tomographic surveys is in the data handling and processing. Generally several hundred travel times must be picked for each section surveyed. These must be checked and entered into the computer for use by ray tracing algorithms.

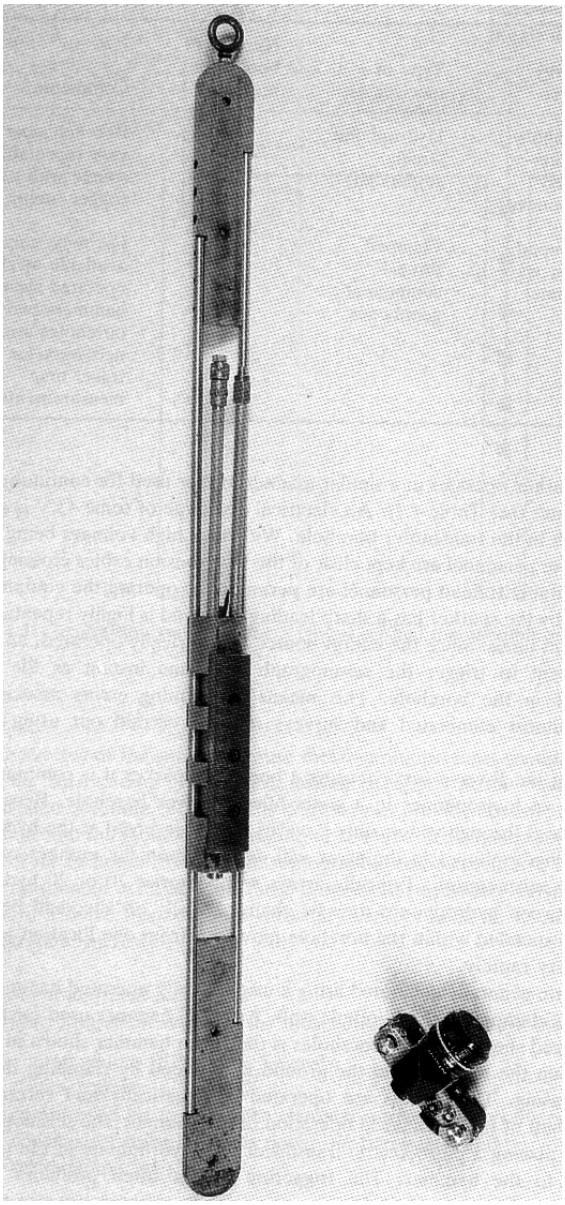


Fig. 4.32 Bison shear wave hammer.

DETERMINATION OF PROPERTIES

Geophysics is generally of very little use in providing parameters for geotechnical analysis and design. There is, however, one important exception. As already noted above, several seismic methods can be used to obtain information on the stiffness of the ground.

Traditionally, the seismic methods preferred by geophysicists have been based upon methods used for deep mineral exploration, and they have therefore relied upon P waves. Compressional (or primary) waves are convenient for these purposes because they can be readily generated (for example using hammer blows or explosives), and their identification on the seismic record in simple — they are the first arrivals. Deep rocks have high skeletal stiffnesses, and therefore the P-wave velocity of the rock is usually much greater than that of its interstitial fluid.

In geotechnics, however, the materials to be tested are generally of low effective stiffness (i.e. their skeletal stiffness is low), and are often fissured and fractured.

Furthermore, in most temperate countries the groundwater table is close to ground level. Therefore the P-wave velocity of near surface soils and rocks is normally close to that of water (about 1500 m/s), and P-wave velocities cannot be used to distinguish between the different types of ground or to provide data on the stiffness of the ground (rather than the stiffness of its pore water). Recent work has therefore concentrated upon the use of seismic shear waves to characterize soils and weak rocks. Shear waves are more difficult to detect, since they occur as secondary events, arriving some time after the primary waves. But they have two helpful characteristics — their amplitude is often large, particularly if a special shear-wave source (which is designed to produce energy which is rich in shear waves) is used, and their sense can be reversed. It is standard practice to reverse shear-wave inputs in order to help the identification of first arrivals, since compressional waves do not reverse.

As was noted above, the very small strain stiffness of the soil or rock mass is linked to seismic shear-wave velocity by the simple equation:

$$G_\circ = V_s^{\,2}\rho \tag{4.10}$$

and because the density of soils and rocks can usually be estimated with reasonable accuracy, the shear modulus (G_o) can readily be determined from the shear-wave velocity.

Until recently, it was thought that, because of the highly non-linear behaviour of most soils (Fig. 4.33), the very small strain stiffness determined from geophysics was so much greater than the values required for design that it was of no use in geotechnical design. It is now known that in many cases this is not so. Finite element analyses of the deformations around civil engineering structures have repeatedly shown that strain levels at working loads are very small, of the order of 10^{-2}-10^{-1}% strain, and this has led to a realization that stiffnesses obtained at higher strains during conventional laboratory triaxial tests are far too low. At the same time that special laboratory techniques have been introduced there has been a rapid growth in the use of seismic shear-wave geophysics.

Comparisons of the shear moduli obtained from geophysics with those obtained from the back-analysis of observed deformations around full-scale structures has shown that while geophysics does overestimate the stiffness of the ground at engineering strain levels, it does so by a relatively low margin. In weak rocks and strongly cemented soils G_o may approximately equal the stiffness at engineering strain levels, while in loose granular media and heavily overconsolidated clays it may be between two and three times greater. Given the relatively large uncertainties associated with stiffness determined in other ways, whether in

the laboratory or using *in situ* testing, shear-wave geophysics must be regarded as a very useful site investigation technique.

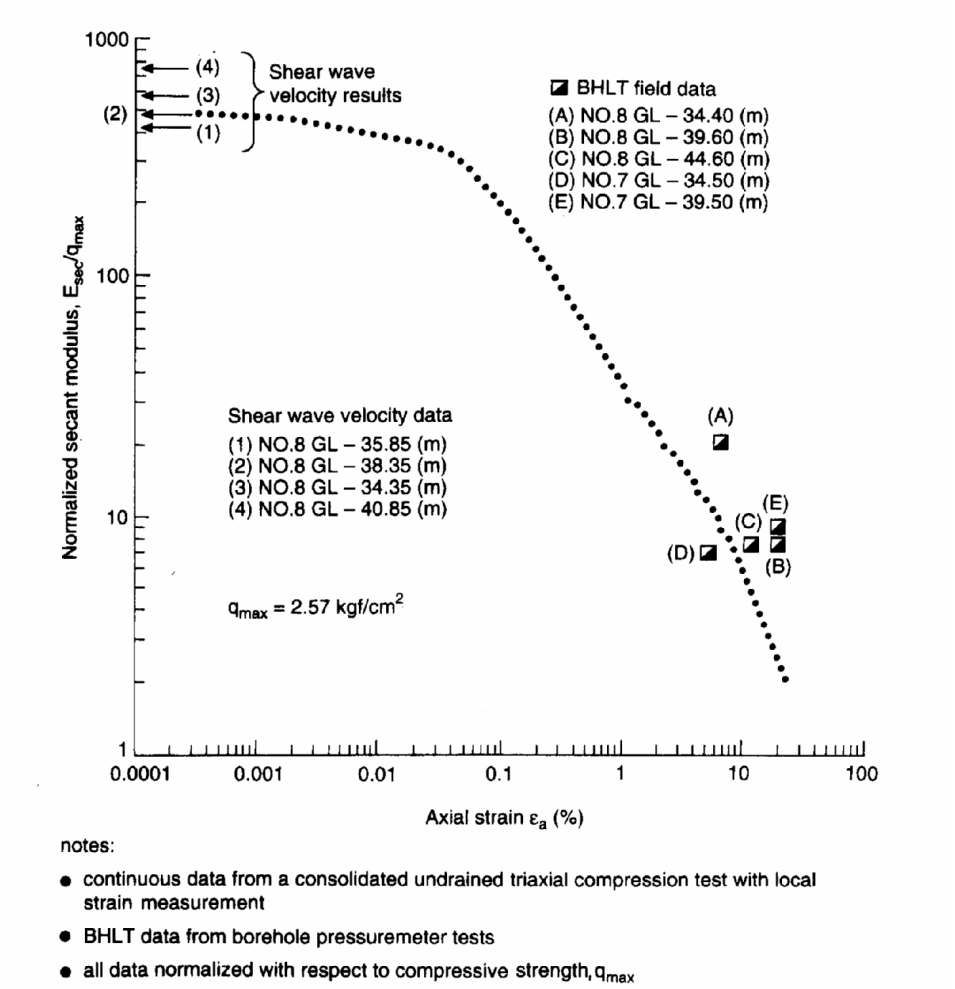


Fig. 4.33 Stiffness degradation curve for stiff clay from Tokyo Bay (Mukabi *et al.* 1991).

Laboratory measurements of small-strain stiffness have frequently been reported in the literature, and are now commonplace in UK site investigation practice, and there are also well-established more conventional *in situ* testing techniques which can be used to determine the stiffness of the ground (for example, plate tests and pressuremeter tests), so that it might be thought that there is little point in carrying out further (very small strain) stiffness measurements by geophysics. However, laboratory tests are subject to sample disturbance, and may not be possible in some soils, because sampling is impossible. And both laboratory and in situ tests are generally carried out on small elements of soil, and may not give stiffnesses which are representative of the mass, inclusive of fracturing and the full range of particle sizes present.

Below we describe three techniques which are relatively well established, and under normal circumstances (in the absence of severe background noise) can be expected to yield good results.

Seismic refraction

The refraction method is based on the critical refraction of seismic waves at the boundaries between materials with different characteristic seismic wave velocities (Fig. 4.34). Snell's Law governs the refraction of seismic waves across a velocity interface. From Fig. 4.34:

$$\frac{\sin i}{\sin r} = \frac{V_o}{V_1} \tag{4.15}$$

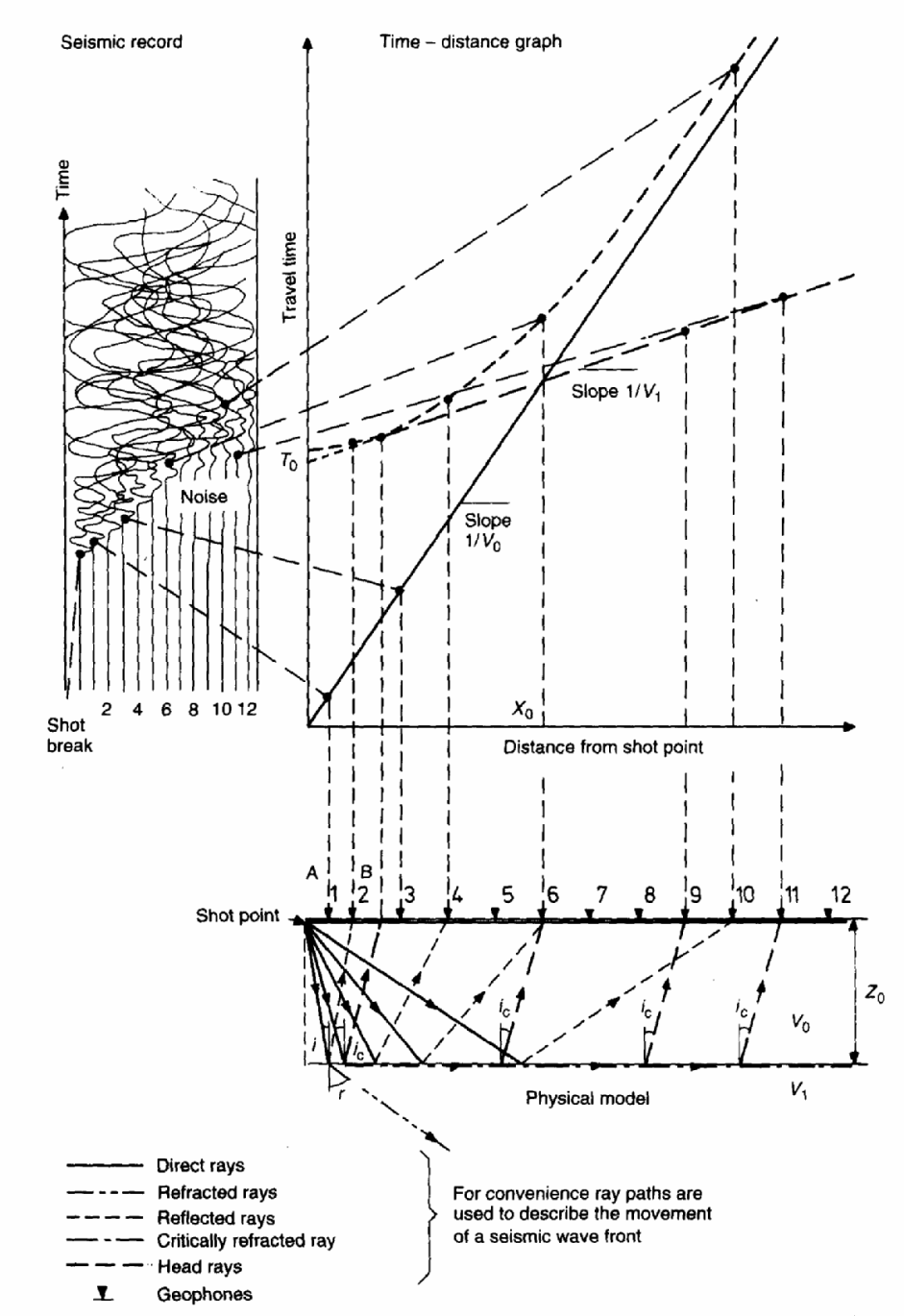


Fig. 4.34 Seismic refraction using P-wave events (first arrivals).

For critical refraction, sin r = 1 (i.e. r = 900). Hence:

$$\sin i_c = \frac{V_o}{V_1} \tag{4.16}$$

Critical refraction therefore will only occur if $V_1 > V_0$, and the rays meet the interface at the critical angle of incidence, i_c. If $V_1 < V_0$, rays will be refracted towards the normal to the interface making critical refraction impossible. This imposes a limitation on the method and is discussed in more detail later. If a ray meets the interface at an angle less than the critical angle, refraction takes place and the ray continues downwards (Fig. 4.34). Also some of the energy is reflected at the interface. If the angle of incidence exceeds the critical angle, total reflection occurs.

The critically refracted ray travels along the interface in the lower (higher velocity) medium. This produces an oscillatory motion parallel to, and immediately below, the interface. Because there is no slippage along the interface, the upper medium is forced to move with the lower medium in a zone adjacent to the interface. This causes the generation of continuous new disturbances which are analogous to sonic booms produced by aircraft flying at supersonic speeds. The shock waves produced are known as head waves. It can be shown that the resultant head wave fronts emerge at the critical angle 4, (Dix 1939). It is the detection of the head wave by the geophones that enables data to be obtained for layers other than the surface layer.

The seismic record produced by a geophone placed at a given distance from the energy source will show a combination of direct events,[2] refracted events (head waves), reflected events and 'ground roll' (surface waves). The reflected waves will always arrive later than the direct waves or the refracted waves. In one special case, the refracted wave and the reflected wave arrive simultaneously. The energy source to geophone distance at which this occurs for a simple two layer case (Fig. 4.34) is given by:

$$x = 2Z_0 \tan i_c \tag{4.17}$$

where x = distance between energy source and geophone, Z_0 = depth to first refractor and i_c = critical angle.

At distances less than $2Z_0 \tan i_c$ the head wave does not exist and hence no refracted events appear on the seismic record.

In Fig. 4.34 the geophones nearest the energy source (shot point) will record the direct wave as a first arrival event. However, because the critically refracted wave travels at a greater velocity than the direct wave, eventually the head wave produced at the interface will arrive at the geophones before the direct wave. The distance from the shot point at which the direct wave and the head wave arrive at the ground surface simultaneously is termed the critical distance X_0. The critical distance is used in the interpretation of seismic refraction data. Other critical distances may be defined for multilayer systems for the simultaneous arrival of refracted waves from different layers.

The equipment required for refraction surveying includes an energy source, geophones, a take-out cable, and a signal-enhancement seismograph.

The fundamental requirement for an energy source for seismic surveying is that it should be capable of delivering to the ground an impulse which has a sharp leading edge, so that first-arrival events can be accurately picked from the geophysical records. Traditionally seismic refraction has been used primarily to determine sub-soil geometry, and has used compressional wave arrivals. For this purpose, and for investigation of shallow depths, it has been sufficient to use a sledge hammer striking a metal plate placed on the ground or, for greater depths of penetration, detonators placed in shallow holes. Figure 4.35 contrasts this

[2] The signal produced by a geophone in response to a wave front is termed an event.

with the techniques required to produce shear waves. For shear waves two sets of data are required at each source position, in order to give reversals (Fig. 4.36). Traces can be stacked, as with P-wave surveys, but only for shear-wave energy input in the same direction. A commonly used borehole shear- wave source is the Bison hammer, shown in Fig. 4.32.

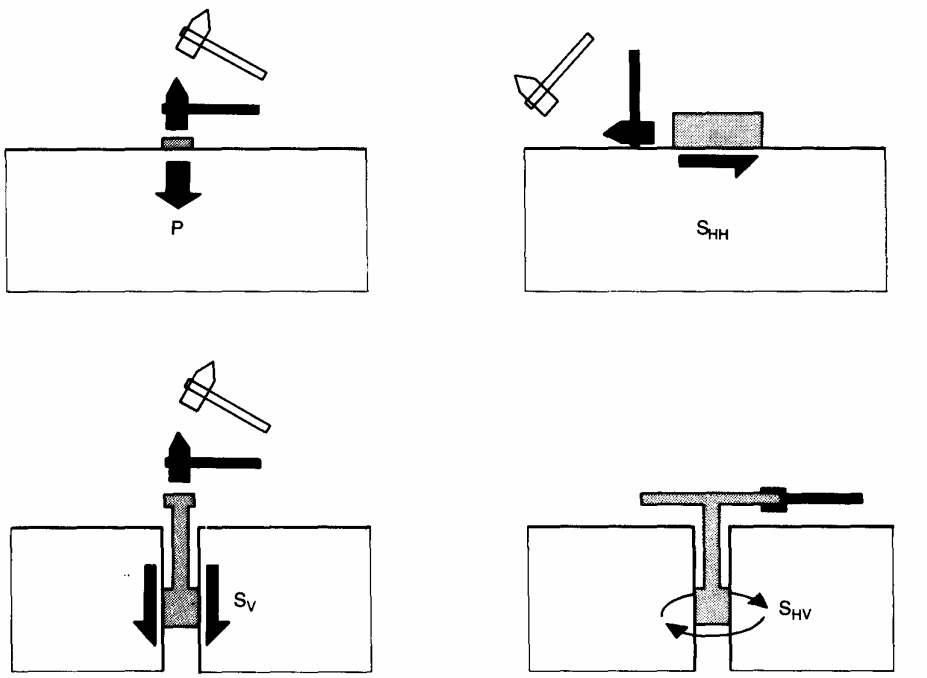


Fig. 4.35 Simple methods of producing shear-wave energy for shallow seismic surveys.

The ground motion induced by the passage of the seismic wave is detected by a small electro-magnetic transducer, termed a geophone. Typically either 12 or 24 geophones are used. A geophone consists of a coil of wire, suspended between the poles of a magnet which is fixed to its outer casing. The coil acts as an inertia element, such that any vibration causes the coil to oscillate within the magnetic field and generate an output voltage. The output of the geophone is therefore proportional to the velocity of the ground on which it is placed. Geophones can be mounted either horizontally or vertically, and for shear-wave surveys should be orientated in the direction of the incoming shear-wave energy, in order to maximize the signal.

The seismograph is connected to the geophones via the take-out cable. Signal enhancement seismographs have the ability to stack repeated geophone traces, and can normally accept up to 12 or 24 geophones. They amplify the small electrical signals that the geophones produce as a result of ground vibrations, and use a precise common time standard against which to record the trace produced by each of them. After amplification the signal is digitized, either using a fixed gain (for example in the ABEM Terraloc) or with 'automatic gain' as in Digital Instantaneous Floating Point (DIFP) machines such as the Bison 9000 series seismographs. When using machines with a relatively small dynamic range (the Terraloc Mk II has only eight-bit resolution) it is necessary to set the gain carefully, and to use different records to capture compressional and shear-wave arrivals. DIFP seismographs overcome these problems.

Geophones are normally laid out at regular intervals in a line (at points 1 to 12 in Fig. 4.34), termed a 'spread', and are orientated in the best direction to receive maximum incoming shear-wave energy. The shot point is normally colinear with the spread. A minimum of two

shot points should be used, one at each end of the spread, and at each shot point the direction of energy input must be reversed.

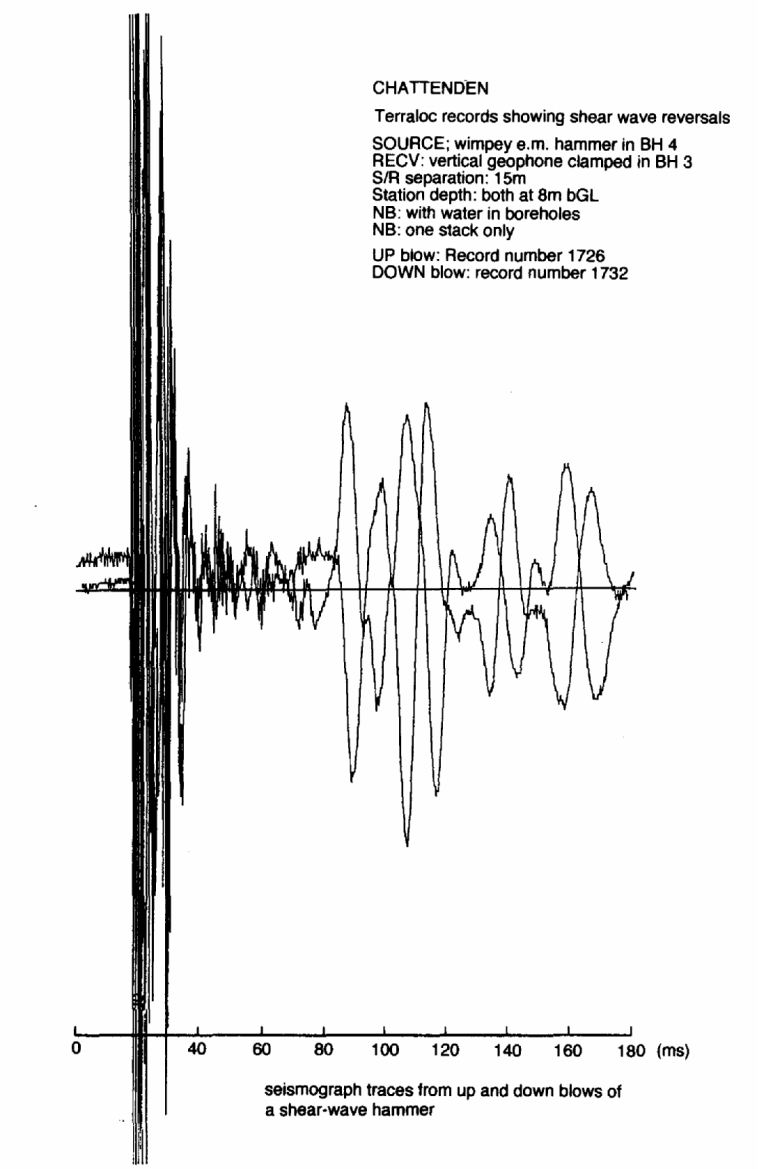


Fig. 4.36 Shear wave reversals on two traces recorded by a signal-enhancement seismograph.

In order to determine the seismic velocities of different materials it is necessary to determine the travel time from the shot position to each geophone for the relevant type of wave. When the shot is made, either by hammer blow or explosives, the seismograph is triggered and starts recording for a short predetermined period, It is advisable to place a geophone next to the shot point in order to provide a reference time, as well as a check on triggering accuracy. The traces that are obtained may be viewed on screen or printed, immediately after the shot, in order to guarantee that good quality data have been obtained. They are best processed by computer. For P-wave surveys the first break is picked, normally by eye. For shear-wave surveys two traces, with the same shot point but with the energy input in opposite directions, are superimposed. The traces are shifted with respect to time to obtain a match on the rise of the trace at the reference geophone. The trace is then searched for the point at which reversals

first occur, and this time is then picked. A time-distance graph is plotted for either the P- or S-wave arrivals, or for both.

For multi-layered ground the velocities and depths of the various layers are interpreted as follows. The time—distance graph is split into straight-line portions, as in Fig. 4.37. The gradients of these lines give the velocities of the various layers. The line passing through the origin is produced by direct waves and the line which when extrapolated intersects the time axis (at time T_0) is produced by critical refraction in the second layer. The intersection of these two lines defines the distance from the shot at which the direct and refracted waves arrive at the surface simultaneously. This has been previously defined as the critical distance X_0. The point at which the extrapolation of the second line (refracted events) intersects the time axis is the intercept time T_0. This has no real meaning with respect to the physical model since it is impossible for head waves to emerge at the shot point. It will be seen from Fig. 4.34 that no head waves emerge at the surface between the shot point A and a point B which is $2Z_0 \tan i_c$ distant from A. The critical distance and intercept time are used together with the characteristic P-wave velocities for each layer to calculate the perpendicular distance Z_0 between the shot point and the top of the second layer (Fig. 4.34). The distance Z_0 may be determined using critical distance by:

$$Z_\circ = \frac{X_\circ}{2}\sqrt{\left(\frac{V_1 - V_0}{V_1 + V_0}\right)} \qquad (4.18)$$

It is, however, easier to determine the intercept time from the time-distance graph, and hence depth determinations are normally made using this parameter from the expression:

$$Z_\circ = \frac{T_\circ}{2}\frac{V_1 V_0}{\sqrt{(V_1^{\,2} - V_0^{\,2})}} \qquad (4.19)$$

However, when experimental errors which affect the determination of V_1 are considered (Steinhart and Meyer 1961), it is preferable to use the critical distance in depth determinations.

Figure 4.37 shows a time-distance graph for a multi-layer case, Intercept times and critical distances (simultaneous arrival of two refracted events for layers other than the surface layer) may be determined for each layer. The above equations can be extended to allow the depth of each layer to be calculated.

The accuracy of velocity and depth determinations together with the chances of actually detecting different strata (or other geological bodies) are very much dependent on velocity contrast between different media. Providing that the velocities of the layers increase with depth, in general the greater the velocity contrast the greater the confidence in identifying different strata and the greater the accuracy of depth determinations. Clearly it is important to have at least some idea of the expected materials before making the choice of using the refraction method, as this method may not be suitable.

If the velocity contrast is low (e.g. V_1: V_o <3:1) the critical distance may be difficult to determine, particularly if the boundary between the two media is gradational (i.e. there is a continuous change in velocity) or irregular. The advantage of using intercept times in depth determinations becomes obvious in such cases. Errors in depth determinations are still possible when intercept times are used, because of errors in velocity determinations owing to a scatter of data defining a velocity segment of the time— distance graph resulting from an irregular refractor. There are techniques available which allow more accurate velocity determinations in such conditions. Where the velocity contrast is large (e.g. V_1: V_o >3:1) the head wave inclination (with respect to the normal to the interface) is very small. This causes the velocity term in the depth determination (i.e. $V_1V_o/\sqrt{(V_1^2-V_o^2)}$) to be a minimum, and hence accuracy is increased by minimizing errors due to erroneous velocity determinations.

Bullock (1978) states that the accuracy of depth determinations is expected to be within ±15% of the true depth over a range of depths of interest between 3m and 100 m.

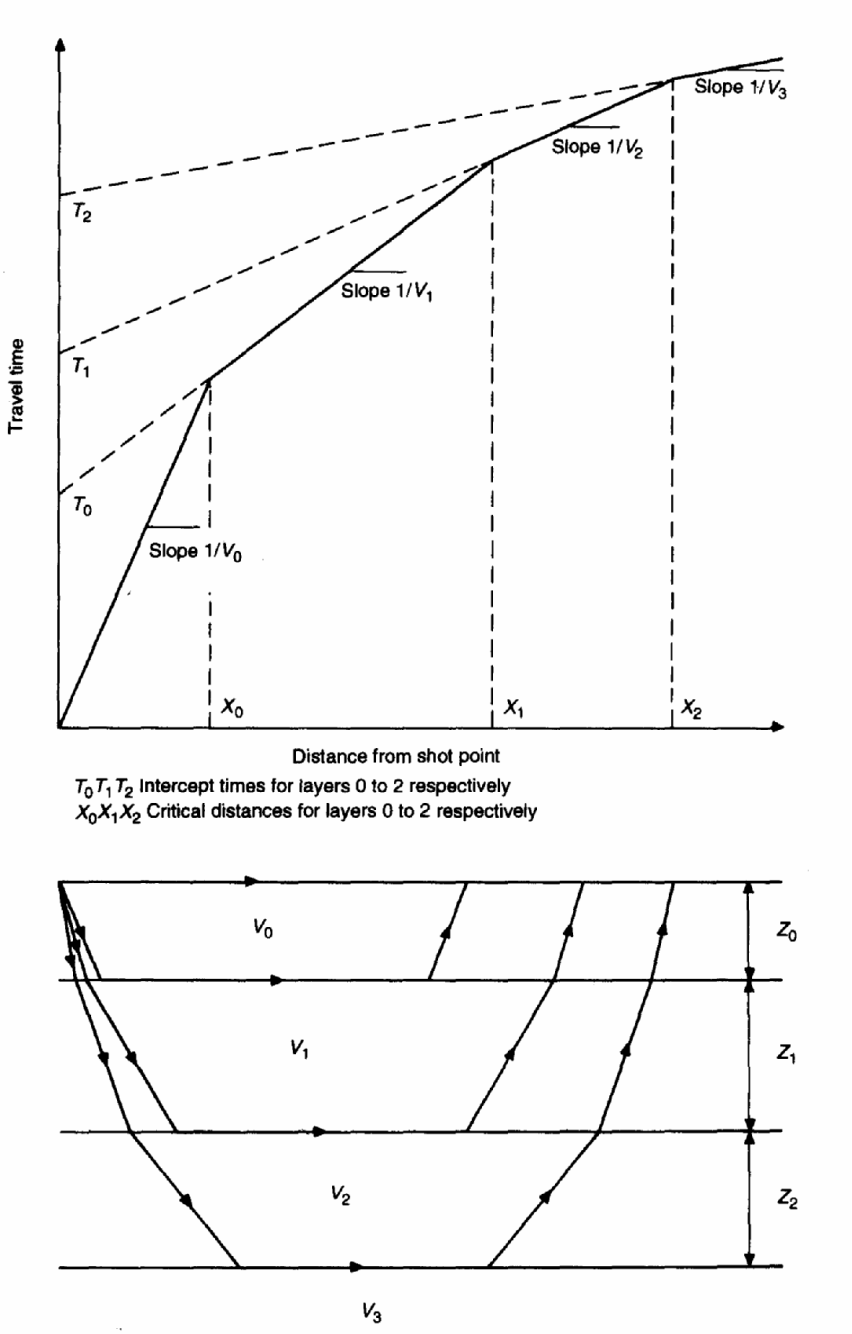


Fig. 4.37 Time—distance graph for a simple multi-layer case.

The interpretation of the time—distance graphs shown in Figs 4.34 and 4.37 is straightforward. In practice, however, the interpretation is often more complicated. The features which commonly give rise to a complicated time—distance graph include:

1. dipping interface;
2. irregular interface and buried channels;
3. lateral changes in velocity;

4. buried step faults; and
5. lateral changes in stratum thickness.

These features may be identified if sufficient data are obtained from each geophone spread. This involves using more than one shot point for each spread in order to maximize the amount of data. It will not normally be worthwhile, during geotechnical investigations, to carry out the additional work necessary for this.

In many near-surface situations the shear-wave velocity of the ground increases approximately linearly with depth, as weathering of the ground reduces. Seismic refraction techniques can then be used to get reasonable approximations to the very small strain stiffness of the soil or rock beneath the surface. Whilst, as we have seen, the customary method of interpreting seismic refraction data is to assume that the ground is layered, each layer having a constant velocity, an alternative treatment is to fit the data to an inverse sinh function, corresponding to a linear increase of velocity with depth (Dobrin 1960). In this case the refraction paths are arcs of circles (Fig. 4.38) and from the geometry it can be shown that the travel time T is:

$$T = \frac{2}{k}\sinh^{-1}\left(\frac{kx}{2V_o}\right) \tag{4.20}$$

where V_0 = velocity at the surface, x = horizontal distance from the shot point to the geophone, k = increase in velocity with depth, and V = velocity at any depth, given by:

$$V = V_o + kz \tag{4.21}$$

Abbiss (1979) fitted the above equations to data obtained from P-wave seismic refraction surveys on the fractured chalk at Mundford by Grainger *et al.* (1973), and using the relationship:

$$z = \frac{V_o}{k}\left\{\left[1+\left(\frac{kx}{2V_o}\right)\right]^{1/2} - 1\right\} \tag{4.22}$$

where z = depth reached by the survey and x = distance from the shot point to the geophone, obtained good correspondence between the velocities from this method and those from the layer method. Abbiss calculated the dynamic moduli obtained from this method of interpretation, using:

$$E_d = G2(1+\nu) = V_s^{\,2}\rho 2(1+\nu) \tag{4.23}$$

Given that:

$$\frac{V_s}{V_p} = \frac{(1-2\nu)}{2(1-\nu)} \tag{4.24}$$

$$E_d = V_p^{\,2}\rho\frac{(1-2\nu)(1+\nu)}{(1-\nu)}$$

As this demonstrates, if compressional (P) waves are used, then Poisson's ratio must be known. Although this is not normally the case, in this instance there had been both laboratory and field measurements which gave a value of Poisson's ratio (v) of 0.24 (Burland and Lord 1970; Burland et at. 1973). The dynamic moduli obtained from the seismic method were about twice those backfigured from the movements below a large instrumented tank test.

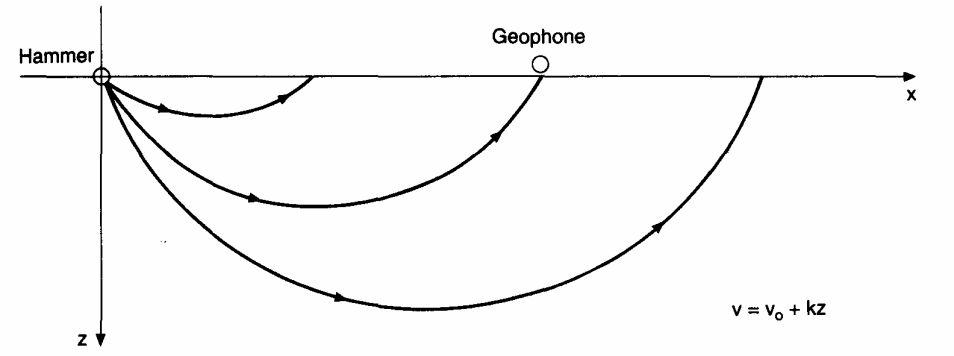


Fig. 4.38 Seismic ray paths for a linear increase of velocity with depth (Abbiss 1979).

Cross-hole and down-hole seismic surveys

The down-hole seismic method, used in conjunction with the in situ cone test, has been described under 'Profiling', earlier in this chapter. Cross-hole surveys are described in detail in Ballard *et al.* (1983). They are typically carried out using three parallel in-line boreholes, with plastic lining (typically about 100mm internal diameter) grouted into them. Horizontal borehole spacing is typically 5—7 m. The closest spacing possible is desirable, in order to achieve a high signal-to-noise ratio, but this consideration must be tempered by the need to record accurate travel times between the boreholes. A vertically-polarized shear-wave source, such as the Bison hammer, is lowered into one of the end holes. Seismic waves are generated by raising or dropping the frame of the hammer against the central, clamped shuttle. In the standard hammer, wave initiation is detected by a piezoelectric transducer, and this is used to trigger a seismograph. Three-component geophones are clamped at the same level as the hammer in the other two boreholes. The seismograph records the incoming waves at the two holes, and the travel time between them is obtained by subtraction. All three boreholes are surveyed for verticality, so that their relative positions are known precisely at each test depth. The shear and compressional wave velocities are then calculated by dividing the appropriate travel time by the distance between the receiver boreholes at that depth. Measurements are made at depth intervals of between 1 m and 5m, depending upon the total depth to be surveyed. It is better to take measurements at intervals of 1 m even in deep boreholes, because the data so-obtained can then be averaged using a rolling average method, so reducing the impact of any 'rogue' measurements. As is usual when picking on first breaks, about 10—15 traces are stacked during data acquisition, to improve the signal-to-noise ratio.

Boreholes constructed, for cross-hole surveys can also be used for up-hole and down-hole surveys. In 911e case of down-hole surveys the speed of surveying can be greatly increased by using strings of three-component geophones. Typically three or four geophones are used in each string, with three orthogonally orientated geophones at each level. The sources of P and S waves are deployed at the surface, and typically consist of vertical hammer blows on a metal plate (for P waves) and horizontal hammer blows on a loaded plank (for S waves). As with the cross-hole survey, the interval velocity is determined from the difference in travel time and the distance between pairs of geophones, but in this case the source is located within a metre or so of the top of the borehole, and the time intervals are determined between adjacent geophones in a single borehole. In both types of survey geophones should be orientated in the borehole so that one set faces in the optimum direction to maximize the amplitude of the incoming signal.

Figure 4.39 shows the results of cross-hole and down-hole seismic surveys in Mercia Mudstone. The material at this site is strongly layered, containing evaporite materials in the form of veins or bands of gypsum, and layers and scattered nodules of gypsum, dolerite and anhydrite. It can be seen that the cross-hole velocities are consistently higher than those from

the down-hole survey, where the geophones were at 6.4 m centres. It might be thought that this was related solely to anisotropy, but in layered ground such as this it is a product, at least partly, of the method of test. Both methods use first arrival events in order to determine the velocity of the ground. But as Fig. 4.40 shows, the cross-hole travel times are, on average, faster because the seismic energy being recorded as first breaks travels through the faster layers, in the form of head waves. If the purpose of the surveying is to determine the *average* ground stiffness, via the seismic velocities, then the down-hole velocities will provide a more reliable estimate than the cross-hole values in horizontally layered ground, since they average the properties of the materials between the geophones. On the other hand, cross-hole velocities are more suited to identifying layering in the ground. This makes it clear that cross-hole and down-hole seismic results should only be interpreted in conjunction with good borehole records, and that they should be regarded as complementary methods.

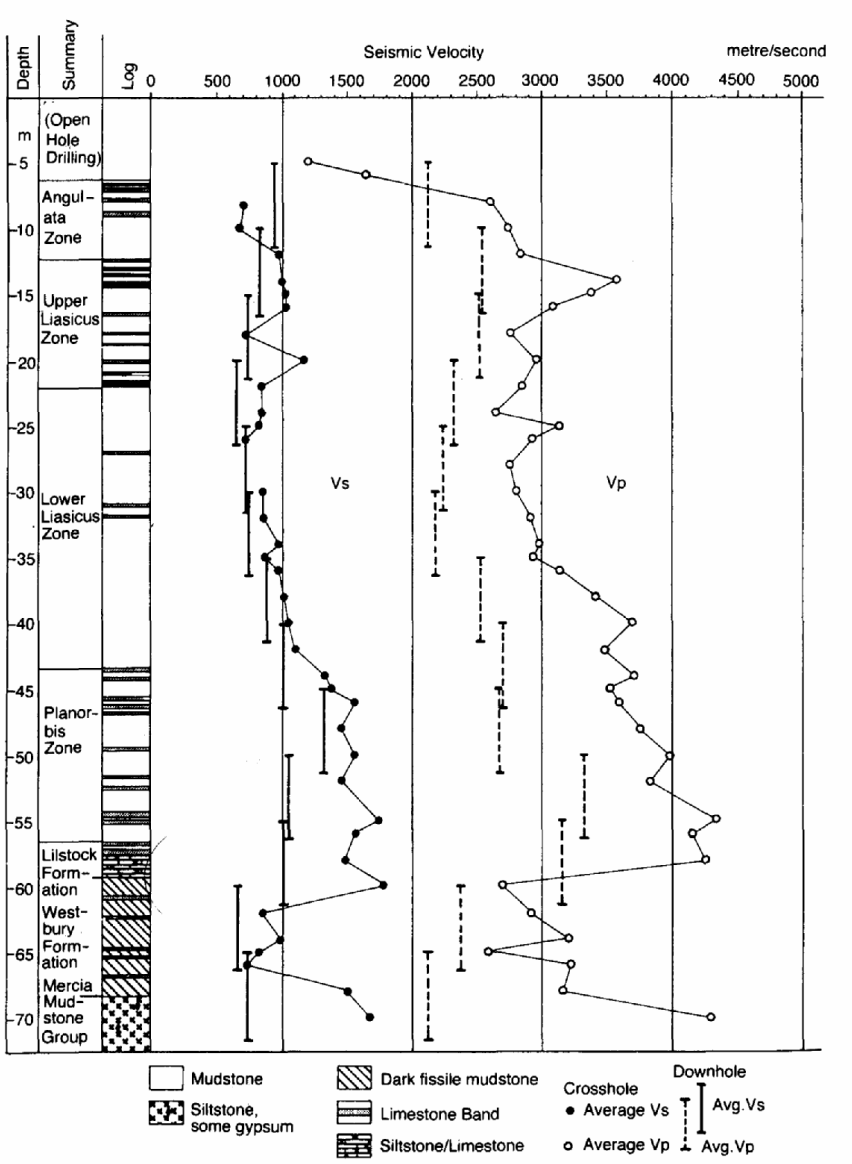


Fig. 4.39 Seismic velocities (V_s and V_p) for Mercia Mudstone, determined from cross-hole and down-hole measurements (Pinches and Thompson 1990).

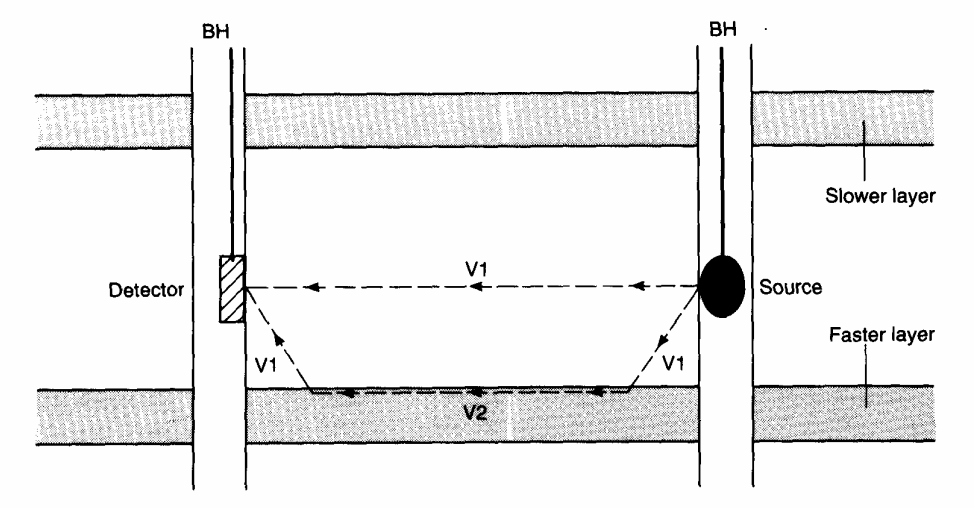


Fig. 4.40 Biassing towards higher seismic velocities in cross-hole seismic tests, as a result of head waves (Pinches and Thompson 1990).

The surface wave technique

As noted at the start of this chapter, seismic energy travels through the ground as both body waves (longitudinal compressional (P) and shear (S) waves) and as surface waves (Love waves and Rayleigh waves). Energy sources used in seismic work are not generally rich in Love waves, but near-surface measurements can be affected by Rayleigh waves, which travel at a similar (although slightly slower) velocity to shear waves. Rayleigh waves are associated with a particle motion which is elliptical in the vertical plane, and which attenuates rapidly with depth and with distance from the source.

A frequency controlled vibrator, which may be relatively lightweight (<20 kg) for a shallow survey, but may weigh several hundred kilograms for a deep survey, is placed on the ground surface, and geophones are placed in a line, or lines, radiating away from it. For small surveys a lightweight electromagnetic vibrator can be used, and the geophones may only extend up to about 2 m away from the vibrator (Fig. 4.41). The geophones may be connected to a seismograph, a phase meter, or (if only two geophones are in use) a spectrum analyser. The vibrator is connected to a power oscillator and amplifier, and a sinusoidal input is used.

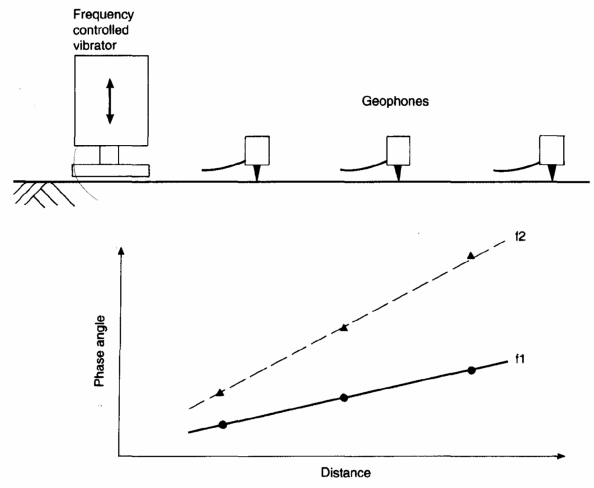


Fig. 4.41 The surface wave technique.

For each frequency, the phase angle of the surface wave at each geophone position is measured. This may be achieved directly with a phase meter or a spectrum analyser. When a seismograph is used, the trace for each geophone is captured in the time domain, and is then converted to the frequency domain using fast Fourier transform analysis. The phase angle/distance from vibrator relationship should be a straight line for each input frequency.

The wavelengths of the surface waves travelling past the geophones can be obtained from:

$$\lambda = 2\pi / \frac{\delta\phi}{\delta d} \tag{4.25}$$

where λ = wavelength, φ = phase angle, and d = distance from the vibrator. The Rayleigh wave velocity is determined from the wavelength:

$$V_r = f\lambda \tag{4.26}$$

where f= frequency, and for an isotropic elastic medium with Poisson's ratio = 0.25 (typical of soils and rocks):

$$V_r = 0.92V_s \tag{4.27}$$

The depth to which each stiffness is attributed is assumed to be λ/3. The shear modulus at that depth is therefore:

$$G_o = \rho\left(\frac{V_r}{0.92}\right)^2 \tag{4.28}$$

Experience has shown that the depth of penetration of Rayleigh wave testing can be quite small, of the order of 8m, in cohesive soils such as the London clay, when using lightweight vibrators and conventional geophones. Figure 4.42 shows results of Rayleigh wave testing from the Building Research Station's London clay site at Chattenden in Kent, and compares these with results obtained from cross-hole surveys and from back-analysis of deformations around excavations and buildings on the London clay in central London. There is good agreement between the geophysical methods down to a depth of about 7 m.

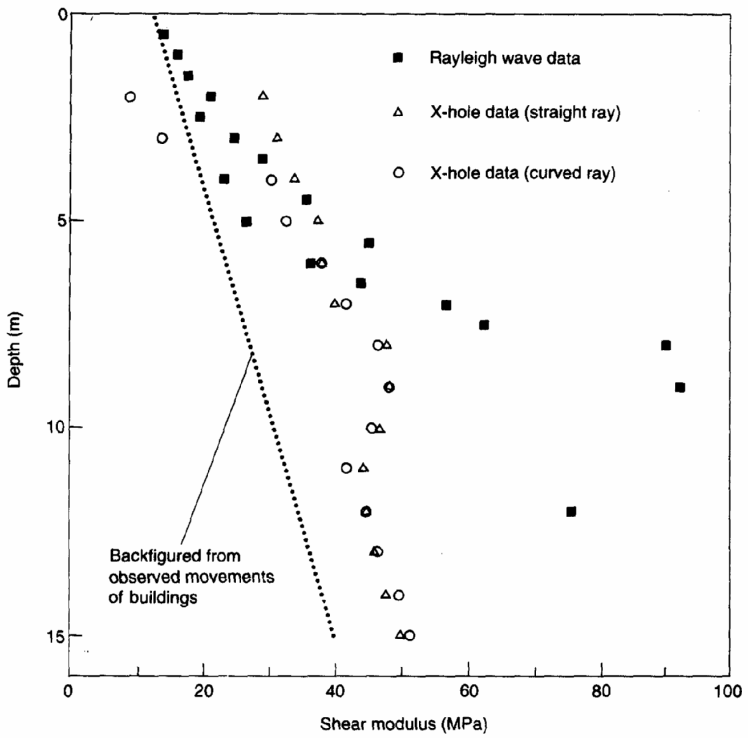


Fig. 4.42 Results of surface wave and cross-hole testing in the London clay.

In fractured weak rocks, where the modulus of the ground is greater, the depth of penetration is also greater. Figure 4.43 shows surface-wave derived values of shear modulus for two near-surface chalks. The low-porosity material was highly fractured, and in a very loose state. The high-porosity material, despite being very weak, had tight joints. It showed no increase in penetration with depth. Comparison of the data from these and other sites with stiffnesses obtained from 1.8 m diameter plate tests has shown that they are comparable, with the geophysical method generally overpredicting the stiffness of the ground by up to a factor of two (Matthews 1993). Yet the surface wave tests were conducted in about two hours each, gave immediate results, and cost approximately 1/30th of the cost of the plate loading tests. Since the fractured rock could not be sampled, laboratory tests were not possible. The only other method of obtaining stiffnesses for foundation design, by using the SPT, is known to be much less accurate, and involves the drilling of investigation holes.

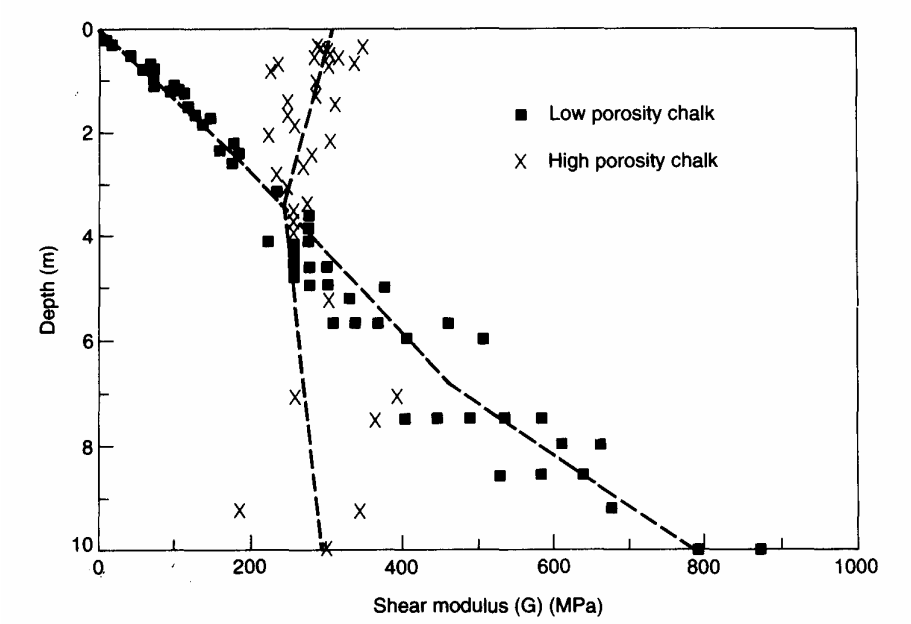


Fig. 4.43 Results of surface wave testing in high- and low-porosity fractured chalk.

The surface wave technique is in its early stages of development in the UK, despite having been developed in the 1930s. It shows considerable promise, and it is likely that its usefulness will increase as equipment is improved. For the moment, however, it must be borne in mind that, unless large and specialist equipment is deployed, the technique can only supply ground stiffnesses for a relatively shallow depth below the base of the vibrator, and that the depth of penetration is a function, at least partly, of the stiffness of the ground being tested.

Chapter 5

Subsurface exploration: boring, drilling, probing and trial pitting

INTRODUCTION

Chapter 4 considered the various indirect methods by which the ground can be investigated, using geophysical techniques. Whilst these methods can be extremely valuable for ground investigation purposes, they are not in everyday use. The bulk of ground investigation is carried out using the direct methods of investigation described in this chapter, coupled with *in situ* or laboratory tests.

The primary functions of any ground investigation process will be one of the following:

1. locating specific 'targets', such as dissolution features or abandoned mineworkings
2. determining the lateral variability of the ground;
3. profiling, including the determination of groundwater conditions;
4. index testing;
5. classification;
6. parameter determination.

Geophysical methods can be very good at giving information on the location of specific targets, and investigating the lateral variability of the ground, but their results are often more qualitative than is preferred by design engineers. Parameters for engineering design most commonly are derived from *in situ* tests carried out in boreholes or from self-penetrating probes although, as was noted at the end of Chapter 4, seismic geophysics methods can give valuable information on the stiffness of the ground. Most profiling is done on the basis of soil and rock descriptions, carried out either on samples obtained from boreholes, or on the faces of trial pits or shafts. And the majority of classification and index testing is carried out on samples taken from boreholes and trial pits.

Therefore the direct methods of testing described in this chapter are at the centre of routine ground investigation. They provide the opportunity to obtain samples for visual description and index testing, which are the primary ways in which the strata at a site are recognized, and for sampling and much of the *in situ* testing needed for parameter determination, as well as allowing the installation of instrumentation such as piezometers.

Boring is carried out in the relatively soft and uncemented ground (engineering 'soil') which is normally found close to ground surface. The techniques used vary widely across the world. The most common methods are augering, washboring and (in the UK) light percussion drilling. This latter technique is well adapted to stoney soils, and has its origin in water well drilling techniques.

Drilling has traditionally been used in the more competent and cemented, deeper deposits (engineering 'rock'). It is now also widely used to obtain high-quality samples of heavily overconsolidated clays, for specialist laboratory testing. Both of the above methods can produce holes to great depths, which can be used for *in situ* tests as well as for sampling, and can allow the installation of instrumentation (for example, to measure groundwater pressures).

Probing is increasingly being used as a cheap alternative to boring and drilling. It is used as a qualitative guide to the variation of ground conditions, and is particularly valuable for profiling. The

techniques used are often fast, and are generally cheaper than boring and drilling, but they cannot be used to obtain samples or to install instruments.

Finally, examination *in situ*, by trial pits and shafts, provides by far the best method of recording both the vertical and lateral ground conditions. Borehole methods generally only take restricted samples, perhaps at every metre or so of depth, for engineering description. Rotary coring normally attempts to recover continuous core, but cannot give a guide to lateral variability, and gives only restricted information on discontinuity patterns in the rock. But trial pitting allows continuous description of soil conditions over the entire face of the pit or shaft, allows measurements of discontinuities in rock, and in addition permits very high quality samples to be obtained.

An understanding of these techniques is important not only because they represent the major element of cost in a ground investigation, and must therefore be used with care, but also because the way in which they are selected and used can have a great effect on the quality of site investigation.

BORING

A large number of methods are available for advancing boreholes to obtain samples or details of soil strata. The particular methods used any country will tend to be restricted, based on their suitability for local ground conditions. The principal methods used worldwide are:

- light percussion drilling;
- power augering; and
- washboring.

Light percussion drilling

Often called ‘shell and auger’ drilling, this method is more properly termed light percussion drilling since the barrel auger is now rarely used with this type of equipment. The drilling rig (Fig. 5.1) consists of:

1. a collapsible ‘A’ frame, with a pulley at its top;
2. a diesel engine; connected via a hand-operated friction clutch (based on a brake drum system) to
3. a winch drum which provides pulling power to the rig rope and can be held still with a friction brake which is foot-operated.

The rope from the winch drum passes over the pulley at the top of the ‘A’ frame and is used to raise and lower a series of weighted tools on to the soil being drilled. The rig is very light, and can be readily towed with a four-wheel drive vehicle. It is also very easy to erect, and on a level site can be ready to drill in about 15 mm. Where access is very limited, it can be dismantled, and rebuilt on the other side of an obstruction such as a doorway.

In clays, progress is made by dropping a steel tube known as a ‘claycutter’ into the soil (see Fig. 5.2). This is slowly pulled out of the borehole and is then generally found to have soil wedged inside it. The claycutter normally has a solid or slotted weight, called a sinker bar, attached to its upper end, the top of which is connected to the winch rope. When the claycutter is withdrawn from the top of the hole, the soil is removed with a metal bar which is driven into it through the open slot in the claycutter side.

In granular materials, such as sands or gravels, a shell is used. At least 2 m of water is put in the bottom of the borehole, and the shell is then surged, moving about 300mm up and down every second or so. Surging the shell upwards causes water to be drawn into the bottom of the hole, and this water loosens the soil at the base of the hole and forces it to go into suspension. As the shell is dropped on the bottom of the hole the mixture of soil and water passes up the tube of the shell, past the simple

non- return valve (sometimes called a ‘clack’). As the shell is raised, the clack closes and retains the soil, which precipitates above it.

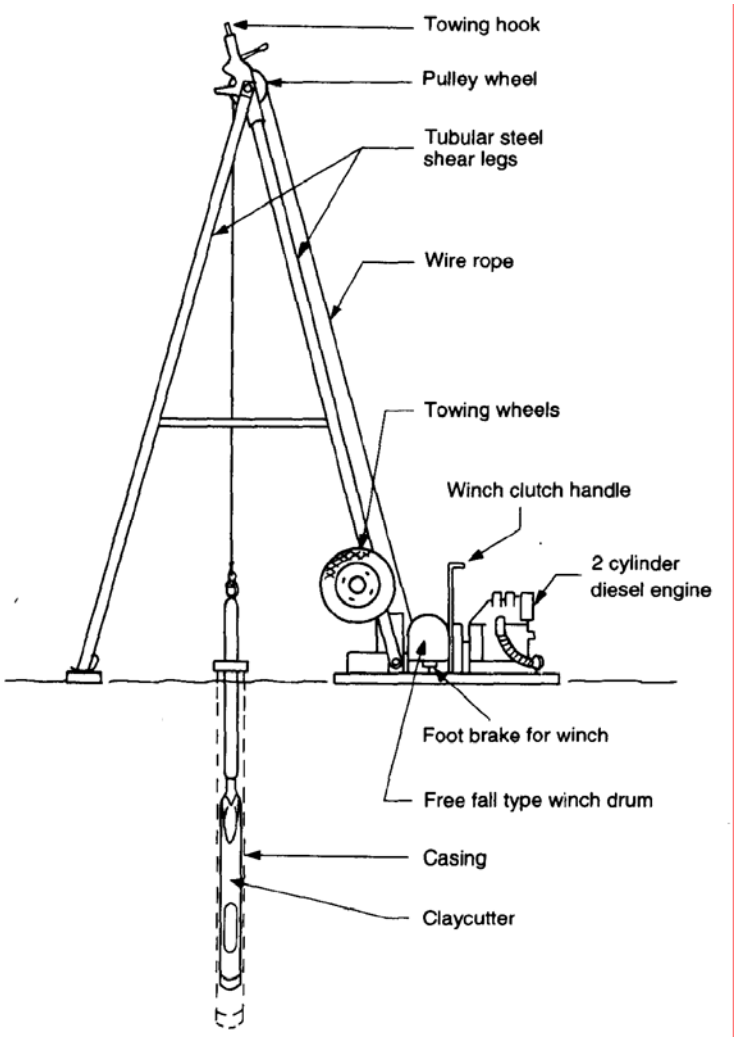


Fig. 5.1 Light percussion drilling rig (Pilcon Engineering Ltd).

By repeatedly surging the shell up and down at the base of the hole, soil can be collected and removed from the hole. The casing should either be allowed to follow the hole down (if it is loose) or should be driven so that it is just above the base of the hole, otherwise progress will be slow, and either large cavities will be formed on the outside of the casing or the soil will be loosened for a considerable distance around the hole. Of course, casing is nearly always used with the shell, because most granular soils will not stand vertically if unsupported in the presence of water.

Casing is not only used when drilling in granular soils, but is also necessary when drilling in very soft soils or when drilling in clays, to seal off groundwater after it is encountered. The presence of water in the base of the hole will allow samples to swell, but the reason that most drillers seal off water is more basic: stiff plastic clays become difficult to recover with the claycutter if large quantities of water are present and if this water cannot be controlled the driller will usually be forced to drill more slowly using the shell.

In the UK, where large parts of the South-east have stiff clays which provide ideal drilling conditions, the light percussion rig normally has 1000—1500kg capacity and most commonly uses 150—200mm dia. casing and tools. It will have little difficulty in boring to 45 m depth in a very stiff clay such as the London clay, but in sandy soils more casing sizes will often be needed to reduce friction.

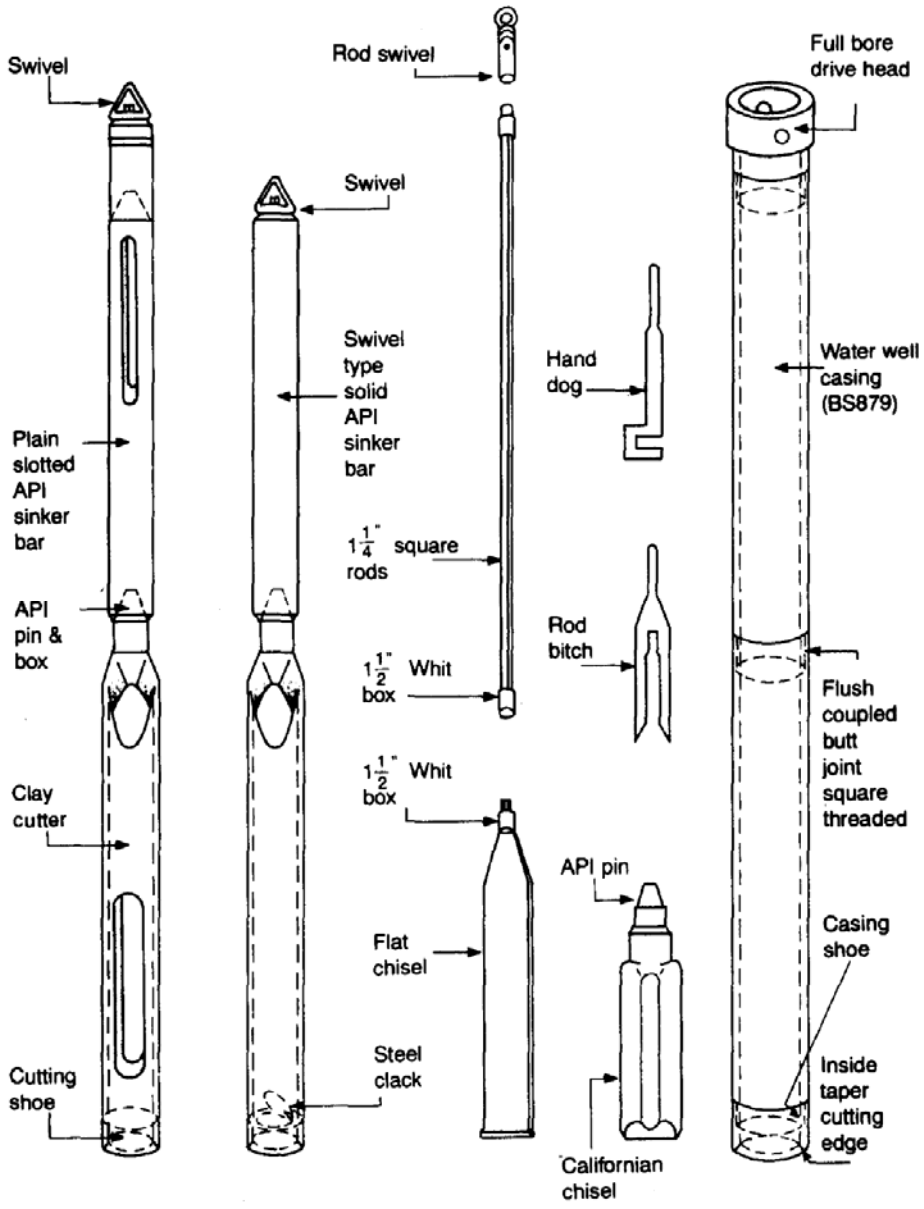


Fig. 5.2 Light percussion drilling tools.

The friction transmitted by sand or chalk to the outside of casing will often be too great to allow the rig to pull more than 10—20m of casing out of the ground without the use of short-stroke hydraulic jacks. Under these conditions strings of casing of different diameters are used to allow a greater depth of drilling. As an example, if a borehole were to be advanced to 50m in sand, the driller might start the boring using 300 mm dia. casing and tools and drill until the rig began to have problems pulling the casing, which might occur at 15 m depth. At this stage the driller would insert a string of 250 mm dia. casing and pull back the larger casing 1 m or so to make sure that it would still be loose at the end of boring the hole. The inner 250 mm dia. casing, of course, would receive no friction on the upper 14m of its length, and the hole could now be advanced until its second string became tight, when a 200mm string of casing would be inserted at, say 30 m below ground level (GL). At the end of boring the hole might be cased with four different sizes, as in Table 5.1.

The minimum casing size possible in Britain is 150mm dia., because this is the smallest size allowing the use of the British Standard General Purpose 100mm dia. sampler (BS 5930). The casing used in the UK is square threaded and flush coupled, in contrast to the drive pipe' in use in the USA which is

coupled via a threaded external collar. This type of coupling can be particularly troublesome in sands, where the coupling considerably increases the difficulty in extracting casing at the end of drilling.

Table 5.1 Example of casing for 50m borehole

Depth (m)	Casing (mm dia.)
GL—14	300
GL—29	250
GL—41	200
GL—50	150

Augering

Augers may be classified as either bucket augers (Fig. 5.3) or flight augers. Bucket augers are similar in construction to the flat-bottomed Sprague and Henwood barrel auger. They consist of an open-topped cylinder which has a base plate with one or two slots reinforced with cutting teeth, which break up the soil and allow it to enter the bucket as it is rotated. The top of the bucket is connected to a rod which transmits the torque and downward pressure from the rig at ground level to the base of the hole: this rod is termed a 'Kelly'. Bucket augers are used for subsurface exploration in the USA, but are rarely used for this purpose in the UK. This is probably because they require a rotary table rig, or crane-mounted auger piling rig for operation, and this is usually expensive to run. Casing also provides some problems, since a single rig cannot drill in cohesionless soil beneath the water table.

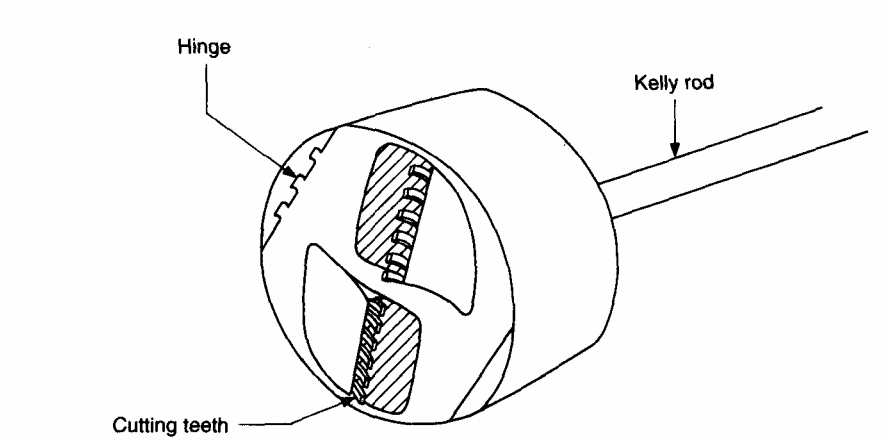


Fig. 5.3 Bucket auger.

Flight augers may be classified as short-flight augers (Fig. 5.4) or continuous- or conveyor-flight augers. Short augers consist of only a few turns of flight above cutting teeth or a hardened steel edge. A high-spiral auger may contain three or four turns of flight. The hole is made by forcing the auger downwards at the bottom of the hole, while rotating it. The cutting teeth break up the soil or rock, which is then transferred up the auger flights. When the flights become full, or when the auger has been advanced for the height of the flights, the auger is raised to the top of the hole and the soil flung clear by rapidly rotating it. Once again, the auger is supported by a Kelly rod which transmits the torque and downward thrust from the drill rig to the auger.

The principal limitation of short augering is that the hole depth is restricted to the length of Kelly rod which the rig can handle. For many of the rigs commonly in use this is only 3—6m. The use of a crane-mounted auger piling rig will allow holes to be drilled to 20—30m if a telescopic Kelly rod is fitted, but as already noted such rigs are very expensive.

The problems of deep drilling with short augers are largely overcome by the use of continuous or conveyor augers. Continuous augers can be classed as: (i) solid stem continuous-flight augers (Fig. 5.5); or (ii) hollow stem continuous-flight augers.

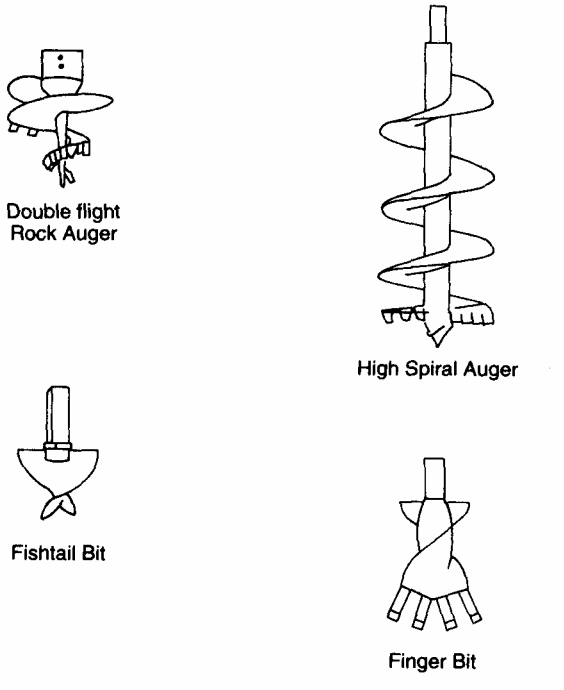


Fig. 5.4 Short-flight augers and auger bits.

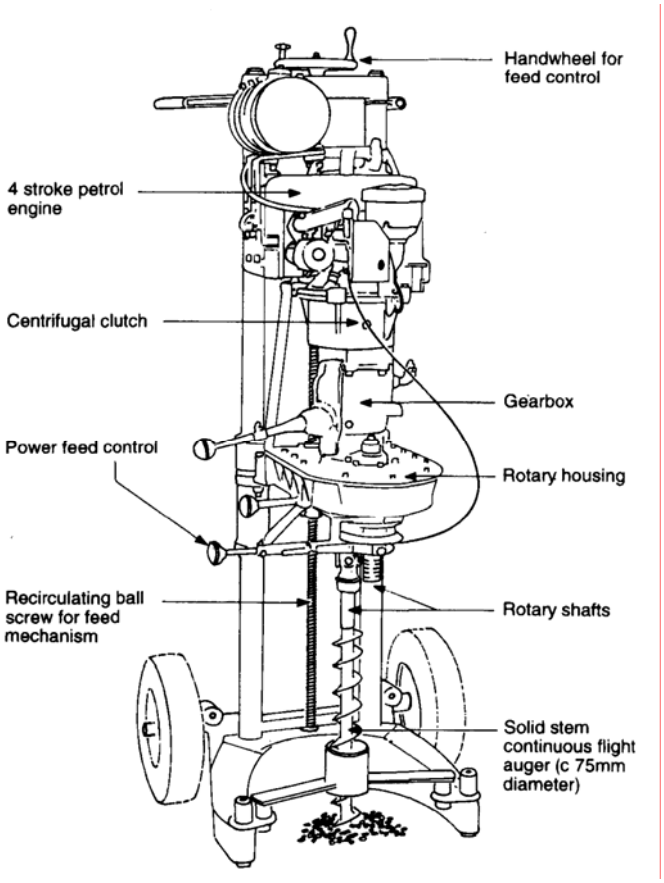


Fig. 5.5 Mobile Minuteman' small diameter solid stem continuous-flight auger rig.

Solid-stem continuous-flight augers allow much deeper holes to be drilled with fewer problems. With this type of auger the Kelly never enters the borehole, as the auger flights extend to above ground level. As the auger is rotated and pushed downwards the soil removed from the base of the hole travels up the flights and emerges at the ground surface. Although this type of auger apparently overcomes the problems found in drilling deep holes with short-flight or bucket augers, it presents a serious problem in site investigation because soil moving up from the base of the hole is free to mix with the soil at higher levels on the edge of the borehole. Thus while auger tailings from short-flight augers or bucket augers may be fairly representative (even if highly remoulded), the soil emerging from the top of a continuous-flight auger will be of no use. In addition, in common with all the auger methods above, the need for casing in granular or other collapsing soils presents a problem. In fine-grained soils a casing can be inserted when collapsing soil is encountered, and can sometimes be advanced by jetting; but in coarse gravels the continuous-flight auger is unusable because it must be removed each time a sample or *in-situ* test is to be carried out. At this stage the hole will collapse.

Hollow-stem augers (Fig. 5.6) consist of an outer spiral continuous flight with a separate inner rod which blocks off the base of the hole when the auger is being advanced. Both the outer flights and the centre plug are furnished with a bit at the base. The auger is forced into the ground in the same way as a solid-stem auger, with the inner and outer sections rotating together. When samples are required, the inner rods and plug are removed and samples can be taken from the material below the base of the auger. Hollow-stem auger drilling would at first sight seem to be the ideal method of producing site investigation holes, because it is often fast and reliable. There are, however, several problems which should be considered.

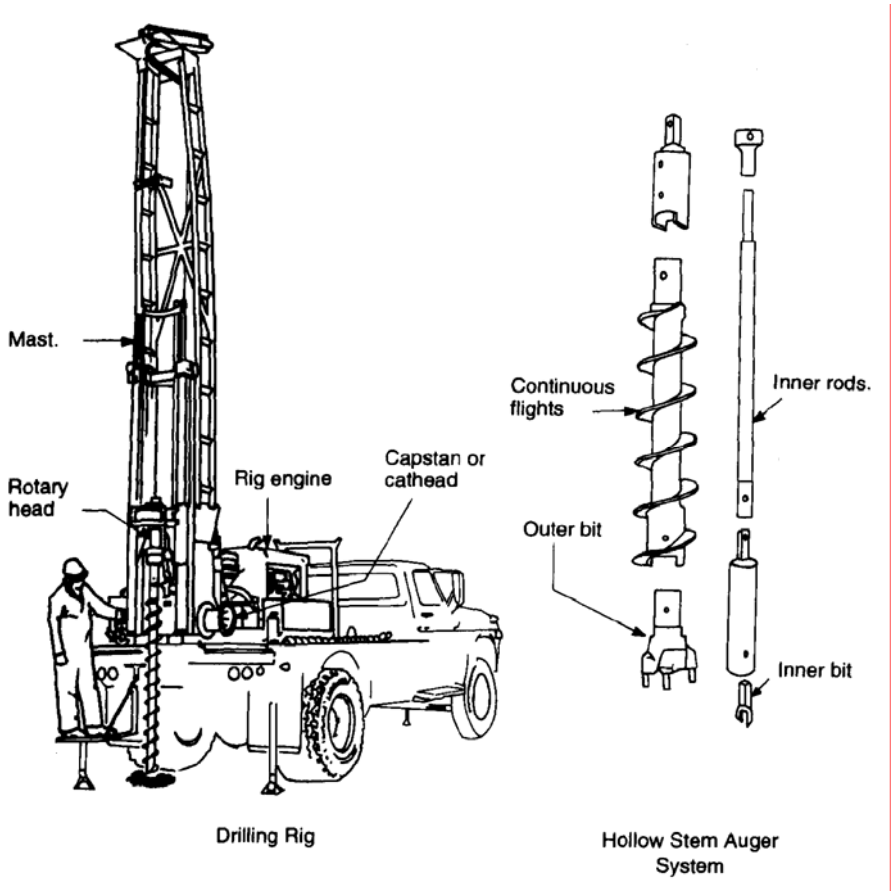


Fig. 5.6 Acker ADII drilling rig and hollow-stem auger system.

First, fissured clays or soils with fabric require relatively large samples for the determination of undrained shear strength and consolidation properties. This means that the hollow stem of the auger must have a large internal diameter (typically 140— 150mm to allow the use of U100 sampling). This

in turn means that a relatively powerful and therefore large drilling rig is required. Even if such a rig is available, access to the site of the borehole may be a problem.

Secondly, there are considerable dangers of disturbing soil ahead of the auger if the driller is overeager in soft or firm soils. Heavy downward thrust may cause the auger to be forced into the soil, displacing material ahead of it instead of boring through it. The hand auger provides a light, portable method of sampling soft to stiff soils near the ground surface.

At least six types of auger are readily available:

- posthole or Iwan auger;
- small helical auger (wood auger);
- dutch auger;
- gravel auger;
- barrel auger; and
- spiral auger.

Figure 5.7 shows a selection of these augers.

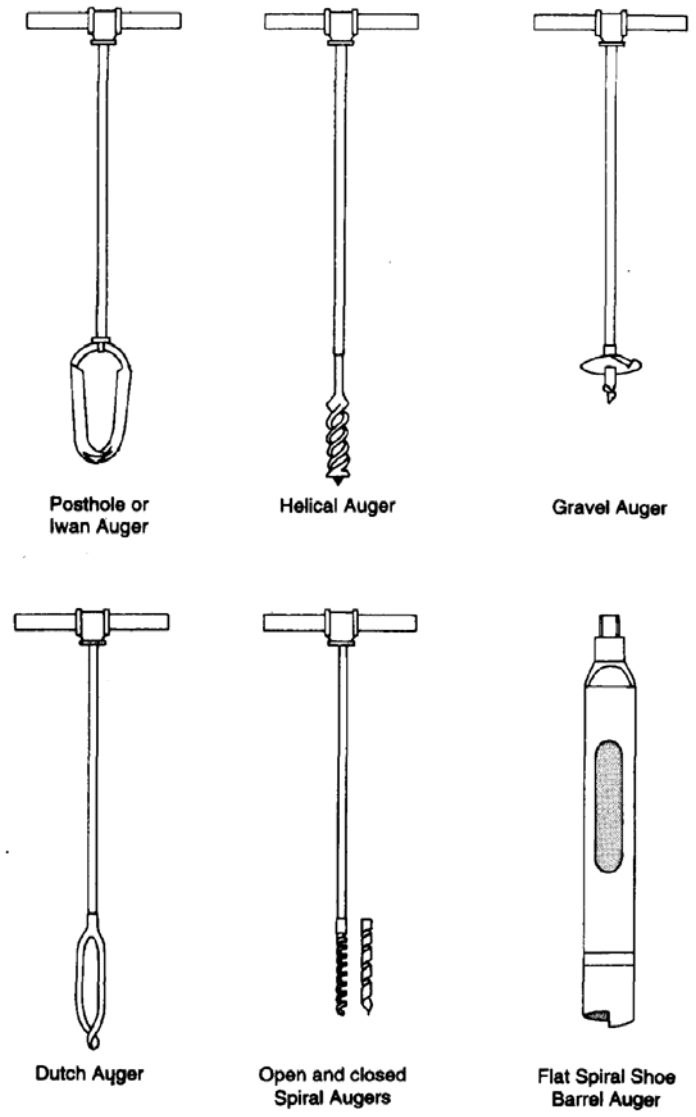


Fig. 5.7 Selection of hand-operated augers.

Hand augers are used by one or two men, who press down on the cross-bar as they rotate it thus advancing the hole. Once the auger is full, or has collected sufficient material, it is brought back to the surface and the soil removed. Although the method is cheap because of the simplicity of the equipment, it does suffer from several disadvantages.

The most commonly used auger for site investigation is the 'Iwan' auger. This is normally used at diameters of between 100 and 200 mm. Small helical augers are quite effective in stiff clays, but become difficult to use once the water table is reached.

Barrel augers are now rarely seen, but were formerly used with the light percussion rig when progress through clays was made using a shell. They allowed the base of the borehole to be very effectively cleaned before sampling took place. Because they are heavy they require a tripod for raising and lowering them in the borehole. When lowered to the bottom of the hole they were turned by hand (see Harding 1949).

In stiff or very stiff clays, hand-auger progress will be very slow, and the depth of boring may have to be limited to about 5 m. When such clays contain gravel, cobbles or boulders it will not normally be possible to advance the hole at all. In uncemented sands or gravels, it will not be possible to advance the hole below the water table, since casing cannot be used and the hole will collapse either on top of the auger (which makes it difficult to recover the auger from the hole) or when the auger is being removed. Only samples of very limited size can be obtained from the hole. In addition, it will not be possible to carry out standard penetration tests without a frame to lift the trip hammer and weight, so that no idea of the relative density of granular deposits can be obtained.

Despite these difficulties, where access for machinery is impossible the hand auger may give valuable information.

Washboring

Washboring is a relatively old method of boring small-diameter exploratory holes in fine-grained cohesive and non-cohesive soils. It was widely used in the USA in the first half of this century, but has been largely replaced by power auger methods. It is still used in areas of the world where labour is relatively cheap, for example southern Brazil.

A very light tripod is erected, and a sheave is hung from it (Fig. 5.8). In its simplest form there are no motorized winches and the drilling water is pumped either by hand, or by a small petrol-driven water pump. Hollow drilling rods are connected to the pump via a flexible hose, and the drilling crew lift the string of rods by hand, or using a 'cathead' (a continuously rotating steel drum, around which a manilla rope is wound).

Progress is made by jetting water out of a bit at the base of the rods. These are continuously turned using a tiller, whilst being surged up and down by the drilling crew. Cuttings of soil are carried up the hole by the drilling water (the 'flush') and emerge from a casing T-piece, being deposited in a sump. Routine identification of the ground conditions at the base of the hole is carried out by the driller placing his hand under the T-piece to collect a sample of cuttings.

Hvorslev (1949) commented that:

> Drillers with adequate experience in washboring can determine changes in and estimate the general character of the soil with satisfactory accuracy, especially when both the drill rod and the pump are operated entirely by hand. On the other hand very serious mistakes may be made by inexperienced or careless drillers, who often fail to recognize changes in the character of the soil, do not clean the borehole properly. and take samples of the coarse segregated material settled at the bottom, instead of the undisturbed material below the bottom. The results of such errors are very misleading soil profiles which often indicate strata of coarse materials at depths

where soft soils of low bearing capacity actually exist. The method should not be used above ground-water level when undisturbed samples are desired of the soil above this level, since the water will enter the soil below the bottom of the hole and change its water content.

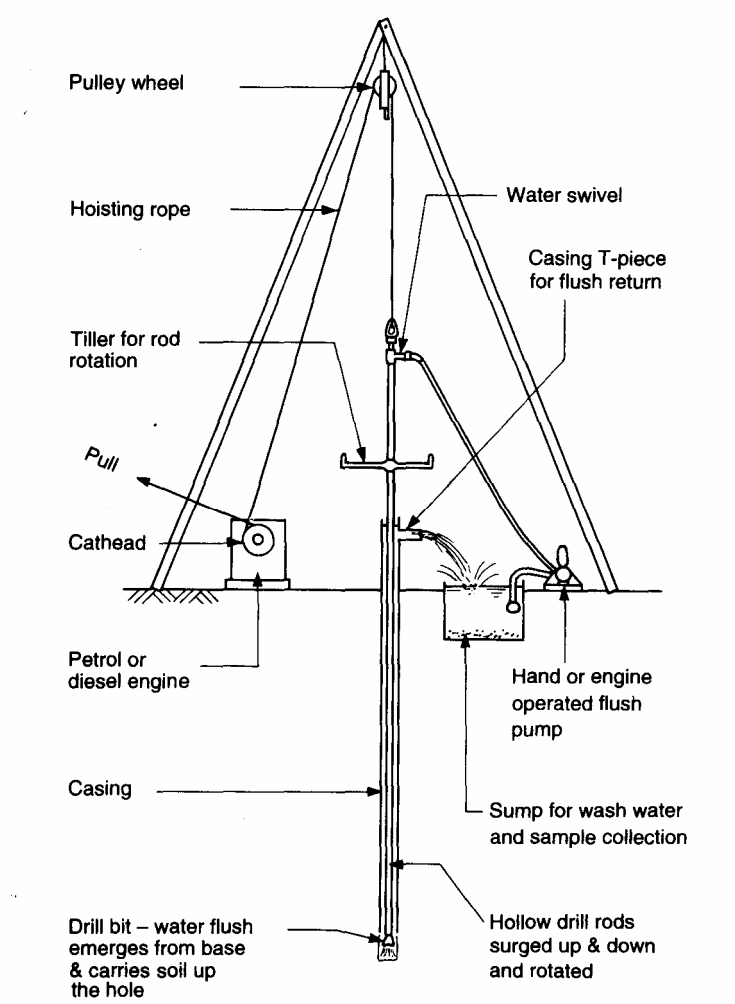


Fig. 5.8 Washboring rig (based on Hvorslev 1949).

DRILLING

Rotary drilling uses a rotary action combined with downward force to grind away the material in which a hole is being made. Rotary methods may be applied to soil or rock, but are generally easier to use in strong intact rock than in the weak weathered rocks and soils that are typically encountered during ground investigations. For a detailed description of equipment and methods the reader is referred to Heinz (1989).

Rotary drilling requires a combination of a number of elements (Fig. 5.9):

1. a drilling machine or ‘rotary rig’, at the ground surface, which delivers torque and thrust;
2. a flush pump, which pumps flush fluid down the hole, in order to cool the mechanical parts and lift the ‘cuttings’ of rock to the ground surface as drilling proceeds;
3. a ‘string’ of hollow drill rods, which transmit the torque and thrust from the rig, and the flush fluid from the flush pump to the bottom of the hole; and
4. a drilling tool, for example a corebarrel, which grinds away the rock, and in addition may be designed to take a sample.

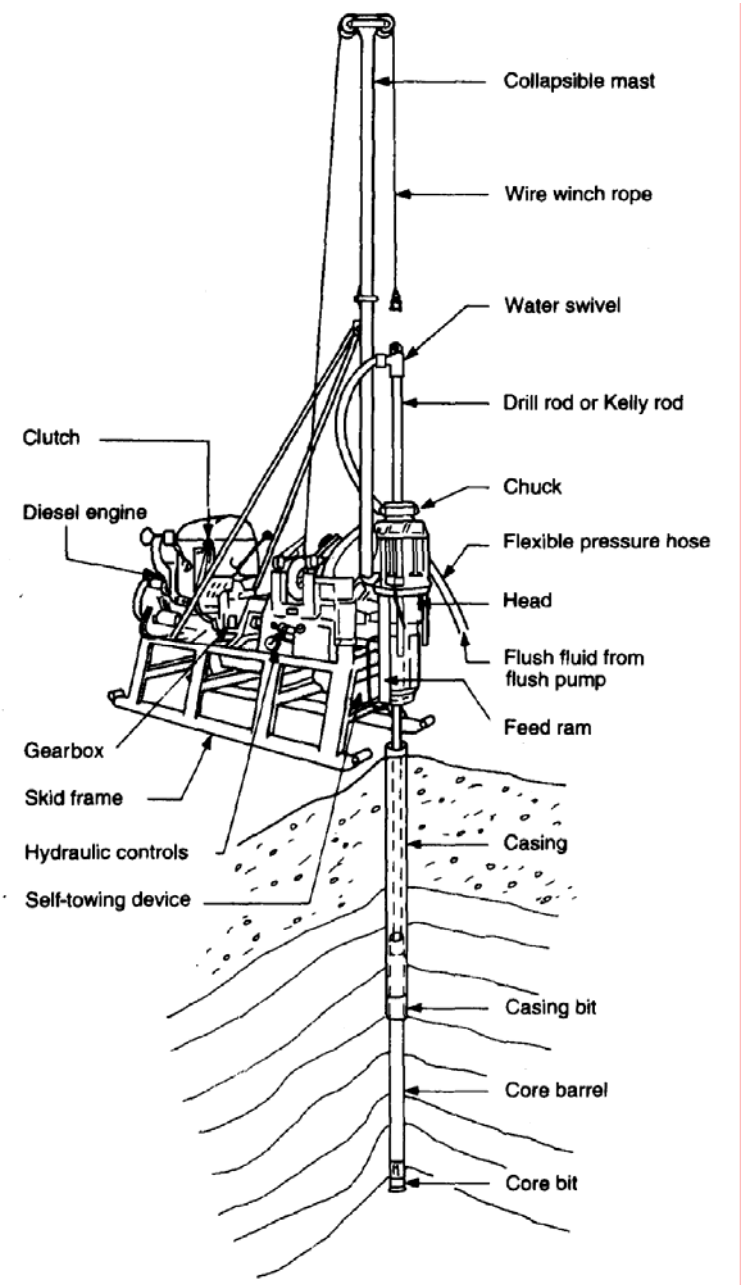


Fig. 5.9 Layout for small-scale rotary core drilling.

Open-holing

Rotary methods may be used to produce a hole in rock, or they may be used to obtain samples of the rock while the hole is being advanced. The formation of a hole in the subsoil without taking intact samples is known as ‘open-holing’. It can be carried out in a number of ways, but in site investigation a commonly used tool is the ‘tricone rock roller bit’ (or roller core bit) (Fig. 5.10). In site investigation such methods are usually used to drill through soft deposits, which have been previously sampled by light percussion or auger rigs. Sampling during open-holing is usually limited to collecting the material abraded away at the bottom of the borehole, termed ‘cuttings’, as it emerges mixed with ‘flush fluid’ at the top of the hole.

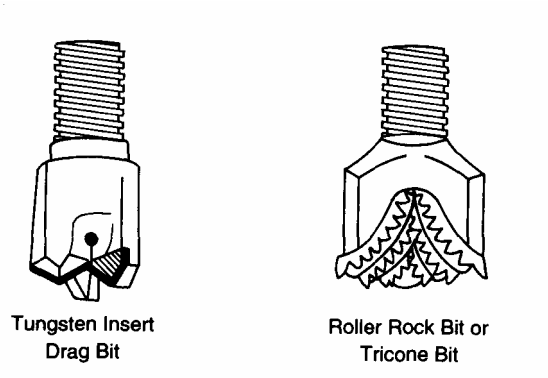


Fig. 5.10 Bits for rotary open holing.

Coring

The most common use of rotary coring in ground investigations is to obtain intact samples of the rock being drilled, at the same time as advancing the borehole. To do this a corebarrel, fitted with a 'corebit' at its lower end, is rotated and grinds away an annulus of rock. The stick of rock, the 'core', in the centre of the annulus passes up into the corebarrel, and is subsequently removed from the borehole when the corebarrel is full. The length of core drilled before it becomes necessary to remove and empty the corebarrel is termed a 'run'.

Coring equipment
The manner in which the rock is abraded away, and by which the cuttings formed by this process are taken to ground level having been discussed, it becomes necessary to discuss the machinery used for the job. At the base of the borehole a bit is rotated against the rock, thus advancing the hole. This bit can be either solid or annular, depending on whether a sample is required. Annular corebits are screwthreaded to the bottom of a 'corebarrel', of which Fig. 5.11 is a typical example in use in site investigation. The corebarrel is screwthreaded to a 'string' (i.e. several lengths screwed together) of 'drill rod', which is generally of smaller diameter than the corebarrel. The function of the rods is to deliver torque and downward force to the bit (via the corebarrel) while at the same time providing the flush fluid to the bit. The drill rods are therefore hollow.

At ground level, the rods emerge from the hole and pass through the 'chuck' of the rig. The chuck grips the drill rods or 'Kelly' and transfers longitudinal and rotational movements to the rods. The Kelly continues upwards and is connected to a 'water swivel' or 'gooseneck', which connects the water or flush hose from the flush pump while allowing the rods to rotate and the hose to remain stationary (Fig. 5.9).

Drill rigs may vary considerably in size and design. Some of the smallest (for example the Acker 1200 PM) mount directly on top of $2\frac{1}{2}$—4 in. drill pipe (casing) installed by other means to rockhead. They consist of a small four-stroke petrol engine, typically of less than 10 h.p., which connects via a gearbox to the top of the rods. The water swivel is built into the machine, and feed is controlled by a mechanical system operated by a handturned wheel. Quite clearly, such a rig has a very limited capability. The load applied to the bit cannot be controlled, and the rig has no inbuilt hoist for lifting the drilling equipment out of the hole.

Most rotary drilling rigs used in site investigation tend to be rather small, when compared with the very large rigs used for oil exploration. They usually incorporate:

1. hydraulic feed control;
2. multispeed forward and reverse rotation;
3. cathead, wire drum hoist, or both;

4. a mast or tripod; and
5. variable mounting options, for both the rig and the drilling head.

Hydraulic feed control is used to vary the pressure between the corebit and the rock being drilled. In soft rocks, the use of excessive pressure will fracture the rock before it can enter the corebarrel, while in hard rocks the use of low feed pressures will result in very slow drilling progress. In very soft deposits the weight of the rods and barrel may be sufficient to fracture the rock and the hydraulic feed may need to be reversed to hold up the rods.

Multispeed forward and reverse rotation is important both from the point of view of good drilling and convenience. Slow speeds of the order of 50 r.p.m., are required for augering, and open-holing with the tricone. Faster speeds, of up to 1000 r.p.m. may be used for rotary coring, depending on the rock type and bit in use. Reverse is useful either for unsnagging or backing out auger tools or ‘breaking’ rods.

For shallow rotary work, or where augering is being carried out, a cathead is used to lift the drilling tools in or out of the hole. A rope attached to the tools or rods requiring lifting is taken up the rig to the top of the mast or tripod, passed over a pulley, and then brought down to the cathead. The cathead consists of a drum which rotates at constant speed. The rope is given two or three turns around the cathead, but because the drum is smooth it does not grip and pull on the rope. The cathead is made to lift the tools by the operator pulling on the free end of the rope. This tightens the rope on the drum, and the friction then acts to pull the rope and lift whatever drilling tools are attached. The cathead normally has limited lifting power, but perhaps more importantly, fine control requires considerable skill.

In situations where greater lifting capacity or finer control of lifting are required a wire drum hoist is normally used. This is particularly necessary when long strings of drill rods or augers are being lifted.

Smaller rigs, such as the Craelius D750 or Boyles BBS 10 provide the rotation of the rods via a bevel gear, which drives an octagonal spindle. Hydraulic feed is then developed by a piston on each side of the spindle, which pulls the spindle down by acting on a crosshead. Rigs with this configuration usually have a limited stroke:

Acker ADII	1.80m
Acker Hillbilly	600—900mm
Acker Teredo	600—900mm
Craelius D750	500mm
Mobile B31	1.73m
Mobile B53	1.98m

Since the corebarrels normally used for rotary work in site investigation are 1.5 m or 3.0 m long these rigs cannot drill the complete length of the corebarrel without having to rechuck; that is to undo the chuck, move it up the rods and reclamp it. To do this, rotation of the corebarrel must be stopped and restarted. This inevitably leads to the exposure of the rock being drilled by the bit to the flush fluid for a longer period than during drilling, and any bad effects of the flush fluid will be emphasized at points on the core where rechucking has taken place.

It can therefore be argued that a long-stroke rig will give much better results when coring soft rocks than the type of rig described above. One type of machine which provides a very long stroke for core drilling is the Acker MPIV hydraulic top drive rig. The rotary action is provided by an hydraulic motor, connected to the engine by flexible hose, which can travel long distances up the mast. The feed is provided by a mechanical system. The Pilcon Traveller 30 and Traveller 50 rotary drilling rigs are examples of lightweight machines capable of drilling a 3 m run without rechucking.

The most common mounting options for site investigation are skid mounting, trailer mounting and lorry mounting. In the UK access is normally poor and many contractors use either trailer or skid mounting. In the Middle East and the USA many more rigs are lorry mounted.

BARRELS

The corebarrel is the normal equipment for recovering samples of rock in site investigation. In its simplest form (as used, for example, to obtain cores of concrete), the corebarrel consists of a single tube with an abrasive lower edge which is loaded and rotated while a flush fluid is passed around the bit under pressure. In this process, first the core inside the barrel is subjected to rotative forces due to the friction of the inside of the barrel against the outside of the core, because the core (being attached to the parent material) does not rotate. Secondly, the flush fluid passes over the surface of the core continuously while it is inside the barrel during drilling.

The effect of the first mechanism is to tend to rotate the core at any points of weakness, such as bedding planes in rock. When rotation of the upper part of the core occurs at such a discontinuity a considerable length of core may be ground away, and a distinctive pattern of circular striations (often called a ‘rotation’) can be seen on the end of each stick of core.

When the flush fluid passes continuously over the core inside the barrel, erosion will occur. This will be particularly serious in soft rocks, where the flush fluid (particularly if water) will tend to soften the outside or along fissures in the stick of core and may well lead to total disintegration of a rock such as soft shale. To counteract these two effects the *double-tube, swivel type corebarrel* is now used as standard in the UK. Figure 5.11 shows a typical example. It consists of the following.

1. An outer barrel, connected to the drill rods and drilling rig above.
2. An inner barrel connected to the outer tube at the top via a swivel which allows the inner barrel to remain stationary while the outer barrel is rotated. Flush passes down the barrel between the inner and outer barrel.
3. A reaming shell (optional and not shown in Fig. 5.11) attached to the base of the outer barrel. This is intended to enlarge the hole produced by the corebit, so that wear on the upper part of the barrel is reduced.
4. A corebit attached to the lower end of the reaming shell. The corebit can be one of many different types, and the illustration shows a face discharge bit.
5. A core lifter or catcher. This device prevents the core from dropping out of the base of the barrel as it is lifted at the end of the run. It consists of:
    i. the catcher box, which is an open-ended cylinder which tapers downwards, and is of slightly greater diameter than the rock core; and
    ii. the catcher spring, which fits inside the catcher box and is fluted or grooved so as to grip the rock core. The catcher spring is cylindrical in shape and has an inside diameter slightly smaller than the diameter of the rock core. The wall of the cylinder is cut through at one point, to allow the spring to expand. When the core tries to drop out of the barrel the spring travels down, and is compressed against the core by the inside taper of the catcher box. Thus the greater the downward force, the more friction is developed between the core and the spring.

The details of the available rotary corebarrels are discussed in Chapter 7.

Triple-tube barrels are identical to double-tube barrels except that a tight-fitting liner tube is used inside the inner barrel. This may be made of stainless steel, or of brass, but in recent UK practice has been composed of clear plastic tube (‘Coreline’). A previously used alternative was Mylar sheet, a thin clear plastic sheet which was held at the lower end of the barrel by a special retainer clip. When using a triple tube, the internal diameter of the catcher and corebit must, of course, be reduced to suit. The advantages of using a third barrel are primarily that the core can easily be withdrawn from the corebarrel, at the end of a run, by pulling the inner liner while holding the barrel horizontally, and that the core can be stored in the liner without disturbing it from its position when drilled. Disadvantages

are that the driller cannot immediately see how much recovery he has achieved, and that the engineer or geologist logging the core must cut the liner (usually with a disc cutter) before he can start work. On balance, the use of Coreline seems to have produced a significant improvement in the quality of core available for logging.

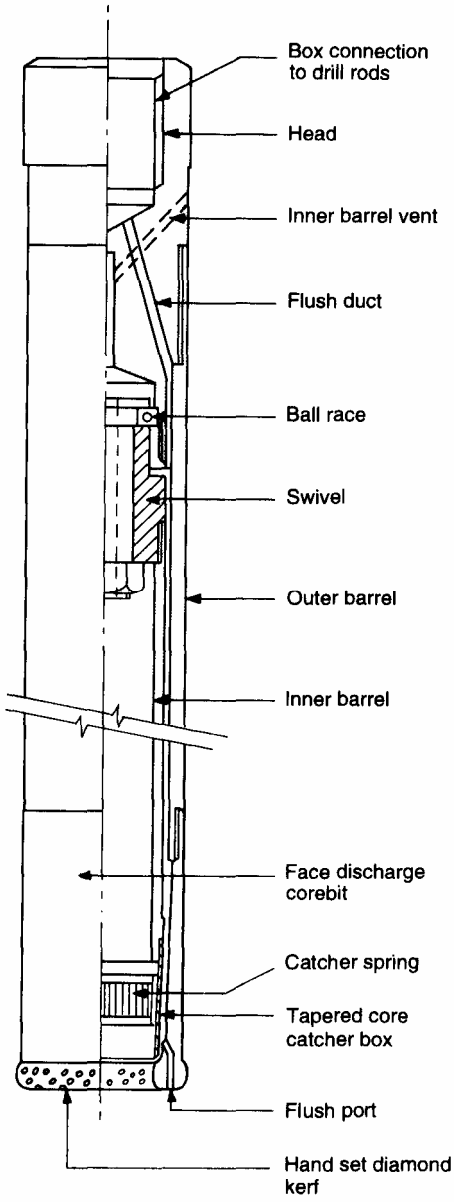


Fig. 5.11 Double-tube swivel type corebarrel with face discharge bit.

Retractor barrels have inner barrels which are spring-mounted, and protrude ahead of the kerf of the bit, in order to provide some protection for the core from the flush. Notable examples are the Mazier and Triefus barrels (see Chapter 7).

Wireline drilling is a technique which has been widely used for deep mineral drilling for many years, principally because it reduces the trip time (i.e. the time necessary to extract the corebarrel from the bottom of the hole, empty the core and replace the barrel). This technique has, in the past ten years, become well established on high quality ground investigations, and has proved particularly effective in the coring of relatively difficult deposits, such as overconsolidated clays, chalks, and interlayered sands, gravels, limestones and clays.

In conventional drilling the outer barrel is connected to a smaller-diameter string of rods, which in turn is passed through the chuck of the drilling rig. Each time the core barrel is withdrawn the entire string of rods must be withdrawn and 'broken' (i.e. unscrew one rod from the other), the core must be extracted from the inner barrel, and then the rod string must be reassembled. Because the corebarrel must be lowered on the rods, trip time increases approximately linearly with depth, and at large depths greatly exceeds drilling time. In unstable ground an outer casing must also be used, further increasing the time necessary to drill a given length.

Wireline drilling does not use any outer casing, but instead uses an outer barrel which extends at full diameter to ground level (Fig. 5.12). The inner barrel is lowered through the full length of the outer barrel, on a wire line. When it reaches the bottom of the hole it latches inside the outer barrel, in the correct vertical position. The outer barrel is then turned by the rig, as flush is pumped down it. The latching mechanism holds the inner barrel down, but does not fix it so that it must rotate with the outer barrel. When the outer and inner barrels have been drilled for the length of the run, the wire line is winched upwards, and the latching mechanism automatically disengages the inner barrel from the outer. The inner barrel and core are hoisted to ground surface, where the core is extracted and a new length of outer barrel is added to the string.

In principle, wireline coring is considerably simpler than conventional double-tube swivel type coring. No casing is used, and there is no swivel to become jammed. In practice, however, the rig used must be considerably heavier than for conventional drilling, because of the torque required to turn the outer barrel, which is in contact with the ground for the entire depth of the hole. Lorry-mounted rigs are the norm. In addition, the bit on the outer barrel can only be changed at the expense of considerable loss of production. It is preferable to use a single bit for the entire length of the hole. Therefore bit wear, and the choice of a type of bit appropriate to the ground conditions are important factors.

Scarrow and Gosling (1986) describe the extensive use of SK6L wireline drilling (producing a core diameter of 102 mm) in the alluvial valley at Baghdad, Iraq. As might be expected, the soils encountered were very variable, consisting mainly of clays, silts and sands, with some gravel being present. Wireline techniques were used in conjunction with polymer drilling mud (see below). Care was taken to restrict the pumping rates, to keep erosion of granular soils in the bit area to a minimum. A constant fluid level was maintained in the borehole at all times, and especially when the corebarrel was being returned to the surface, and the corebarrel was raised and lowered slowly, in order to minimize suction effects and pressure surges, and therefore the chance of piping and base heave (see Chapter 7).

BITS

The selection of the right corebit for the job is a rather difficult task. The variables in a corebit design are:

- face contour;
- cutting material;
- diamond types, grades and sizes;
- mounting matrix;
- waterway size, shape and position; and
- 'kerf' width.

The face of the bit may vary from a 'flat' surface to a 'full-round' surface, where the radius of the surface is equal to half the 'kerf' width (i.e. half the thickness of the diamond inset part of the bit). In practice most bits are semi-round or semi-flat in design.

The cutting material may be tungsten, diamond impregnate, or hand-set diamonds. Tungsten bits usually have large tungsten inserts mounted radially across the kerf. This type of bit can only be used for drilling very soft formations, such as soft shale or coral. However, the coarseness of the inserts

increases the bearing pressure on the rock, and may well lead to a disturbance and fracture ahead of the bit. This type of bit is also used for casing.

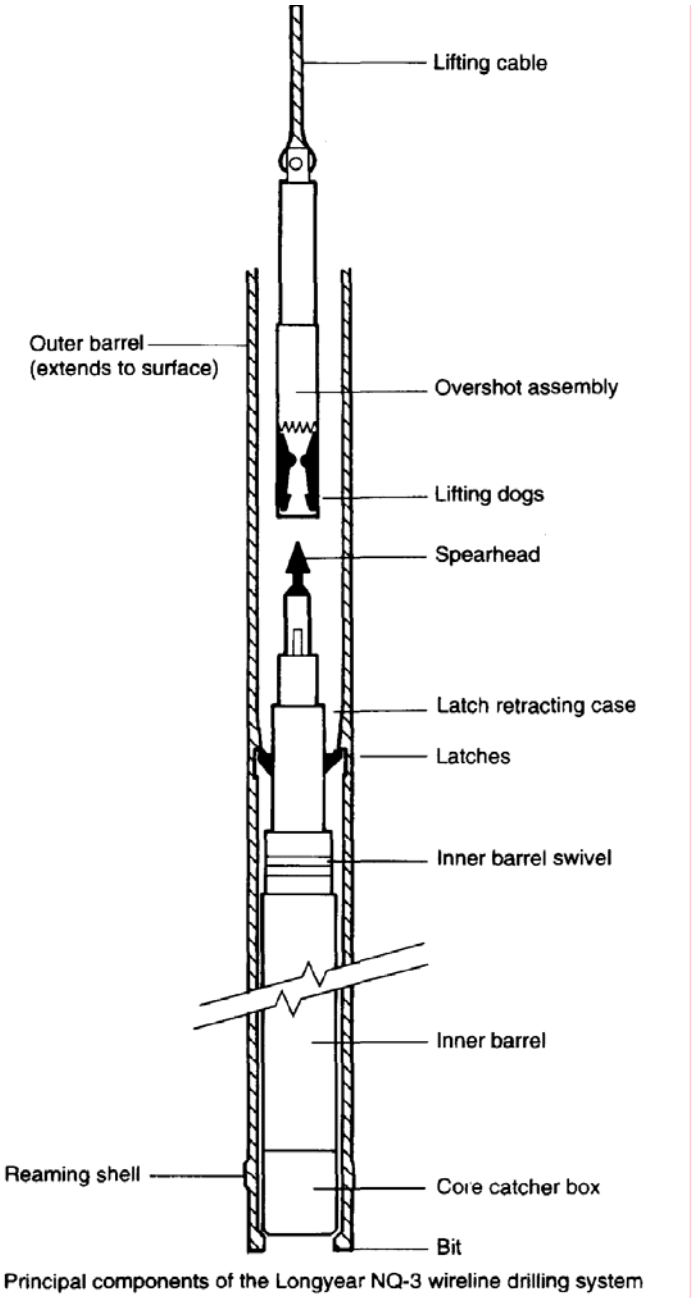


Fig. 5.12 Principal components of the Longyear NQ-3 wireline coring system.

Diamond impregnate bits consist of a sintered powder metal matrix with fragmented or fine ‘Bortz’ (i.e. low grade industrial diamonds) embedded uniformly throughout it. As the matrix wears down, new sharp diamonds are exposed. This type of bit is suitable for hard rocks, and may often be used for casing shoes where casing has to be advanced into the rock.

The best quality diamond bits contain hand-set selected Bortz. The diamonds are of selected size and grade and are placed in the matrix by hand, with the hardest vector of each diamond facing in the direction of the work. This type of bit differs from tungsten or impregnate bits because with the former the bit is used until the ‘crown’ (i.e. the part of the bit formed of the matrix, and set with diamond or tungsten) is consumed. A Bortz-set bit is only used until either the diamonds become polished, or the

matrix is abraded around the Bortz to the point where they are over-exposed. At this stage the diamond bit is returned to the manufacturer, where the diamonds are removed and reset.

The quality of diamonds used in the bit varies. Diamonds are also sometimes classified on a geographical basis, such as 'West Africans', 'Congos', 'Brazilians', 'Angolans', etc. This means only that the diamonds resemble the typical products of these areas. Congos and West Africans are commonly used in drilling bits, with a preference for West Africans. According to Boyles Bros *Diamond Drilling Terms and Equipment Standards*, Congos were previously only considered for use in broken form, but more recent applications seem to use them as large stones.

The size of diamond in use in the bit should be tailored to the soil or rock being drilled. In soft material or fractured and weathered near-surface rock large Congos may be used, because the large size has good clearance and allows good washing without blocking the bit. Large diamonds are also apparently more capable of surviving the shocks administered during drilling fractured rock. As the rock becomes harder, smaller and more numerous diamonds are necessary to provide more cutting edge and therefore keep progress at a reasonable level. In addition, the use of more diamonds provides an even load distribution on the bit. The weight of a stone is measured in terms of its 'carat' where 1 international metric carat = 200 mg. The 'carat weight' is the total weight of diamond set in the bit, which may be between 5 and 50 carat depending on bit size.

The matrix must hold the diamonds in the required position, resist shock, and transfer heat away from the diamonds. The property often used to classify the abrasion resistance of the matrix is hardness, sometimes measured on the Vickers or Rockwell scale. This is not a perfect classification method because hardness is not directly related to either abrasion resistance or the other properties mentioned above.

The design of the waterways also depends on the type of rock to be drilled, and in addition on the flush fluid. Air or mud flush require larger size or more passageways. Soft formations require multi-waterway bits to allow the quick removal of cuttings before blocking occurs. Once the waterways have blocked then not only will the bit overheat, and therefore undergo excessive wear, but the core will also be seriously damaged. In hard rock the cuttings are of finer size and are more granular in nature. Fewer waterways need to be incorporated, and in some cases when drilling very easy materials no waterways are used.

Two types of bit are available: normal or face (bottom) discharge. In normal discharge bits all the flush passes down between the inner and outer barrels, outside of the catcher box, and out of the barrel between the core and the bit. The contact of the flush water with the core, even for this short distance, can have a serious effect and in soft deposits face or bottom discharge bits are commonly used. The drilling bit has ports in the lower end (the face) and the majority of the flush fluid is therefore discharged away from the core, flowing to the outside of the bit.

The face discharge bit represents an improvement on the conventional bit, but suffers from some disadvantages:

1. flush fluid is still allowed to make contact with the core; and
2. over-eager drilling may lead to the ports becoming blocked, especially when drilling in soft rocks or hard clays. Under these conditions it may be necessary to apply no downward pressure to the rods, or in extreme cases even to hold the rods up to reduce the pressure on the face.

One method of overcoming the problems of over-stressing and port blocking may be to use a step-taper bit.

Flush fluid
Flush fluid is passed around the bit while drilling proceeds. The purpose of the fluid is:

1. to remove the cuttings from the borehole;
2. to cool the drilling bit, and drill rods;
3. to reduce mechanical and fluid friction; and
4. to help to retain an open hole wherever possible, without the use of casing.

At the same time, the flush fluid should not encourage the softening or disintegration of the cores, which are the purpose of drilling. A large number of different types of flush fluid are in use, but they are generally classed as:

- water-based (for example water, bentonite/water (drilling mud));
- oil-based;
- air (or mist); and
- stable foam.

The most common flush fluid in use in British site investigation is water, with air being used when water causes serious softening of the formation being drilled. Water is, however, by no means the ideal fluid.

Most drill rigs use normal circulation; that is the flush fluid is pumped down through the drill rods, passes outwards over the bit and travels upwards in the annular space between the drill rods and the outside of the hole carrying the cuttings with it (Fig. 5.9). The requirement of removing cuttings from the base of the hole requires either viscous flush fluid or high flush velocity to maintain the cuttings in suspension.

AIR
Air is readily available for use on site as a flush fluid. It has the attraction that, provided groundwater inflows to the borehole are not great, it does not lead to degradation and softening of the core. From all other points of view, however, it is a relatively undesirable form of flush fluid.

Air has a very low viscosity, which means that satisfactory air flush drilling normally requires uphole velocities of about 1000 m/mm., which can only be obtained in the relatively large-diameter drillholes used for ground investigation by using expensive, high-output air compressors. A 600 cfm (cubic feet/minute) compressor is typically required, and this is noisy and difficult to tow to the site of remote drillholes. The air leaving the borehole produces a dust plume, unless special equipment is used to suppress it.

If air penetrates the ground then it may not reach the ground surface at a sufficiently high velocity to carry the cuttings with it. There will be ‘loss of return’, the cuttings will be dropped, and the hole may become blocked. This will seriously affect bit wear.
In addition to these problems, air flush has negligible lubrication properties, is not particularly effective as a coolant, and frequently leads to excessive erosion of soft rocks around the corebarrel and drill rods, which can make subsequent *in-situ* testing very difficult.

WATER
Water, being cheap to provide in the UK, is most frequently used and overcomes several of these problems. Being more viscous it can lift cuttings at a much lower velocity (c. 24—50m/min) which often means less borehole erosion, and less loss of return. But even water return may be lost in zones of high permeability, and water has the significant disadvantage of causing softening and disintegration of soft rocks, such as shale or chalk, and hard clays (for example, Keuper marl). In arid zones the need for large volumes of water often makes this type of flush impractical.

BENTONITE AND POLYMER MUDS
The use of ‘drilling mud’ (a thin mixture of water and bentonite) has various advantages over water. First, it is more viscous and can therefore lift cuttings adequately at a lower velocity. Secondly it will

cake the edges of the borehole, and the outside of the core, and will largely eliminate the seepage of water out of the borehole, thus reducing problems of loss of return. Because of this, smaller volumes of flush fluid will be required and the fluid may he recirculated via a settling tank (where the cuttings are allowed to drop out of suspension). The cake formed on the outside of the borehole has the effect of considerably improving the stability of the borehole, provided the flush fluid head is maintained higher than that of the groundwater.

Mud is now widely employed in petroleum exploration drilling. Although used as early as 1901 in Texas, even in this field it has only developed into its present state of sophistication in the last 30 years (Cumming and Wicklund 1980). In deep well drilling, mud must have a relatively high density to keep the hole open, and it requires continuous checking and modification with additives as drilling proceeds. Thus a series of standard tests are used (API RP13B 1969) to monitor changes in the mud caused by the drilled formation, and additives are used among others to increase density, remove calcium, control hydration (swelling) and sloughing, and promote wall caking (see API Bul D11 (1965)).

In site investigation, the drilled depths are normally shallow and the demands made on the mud are often quite small. Frequently mud is made up by adding between 10 and 25 lbs of sodium montmorillonite (bentonite) to each barrel (35 imperial gallons) of fresh water. In clays and shales a thin mix is used, and the mud should have the consistency of a thin cream. In coarse-grained soils a thick mix is required to bring the cuttings to the top of the hole. When better wall support is required, barium sulphate (barytes) can be used to increase the density of the mud without giving it an unusable high viscosity. The disintegration of some water sensitive soils (for example, shale) can be reduced by additives such as organic polymers, starch derivatives, gypsum, sodium silicate, chrome lignosulphonates and calcium chloride, but the authors have not seen these used in site investigation drilling.

Whilst bentonite muds have many advantages from the viewpoint of borehole stability and the prevention of softening of weak rock cores, they have two significant disadvantages. First, they are difficult to dispose of, at the end of drilling a borehole. The mud cannot simply be tipped on the site, and it cannot be discharged into nearby sewers. Secondly, bentonite mud must be properly mixed, using appropriate equipment, in order to ensure that it is of the correct consistency and does not contain unmixed dry bentonite lumps, capable of clogging flush ports in the corebarrel. Both of these disadvantages can be overcome by the use of synthetic polymer mud. Many polymer muds are biodegradable, so that disposal is no longer a problem. They are also much easier to mix than bentonite muds. Despite the fact that they tend not to have the caking properties of bentonite, recent practice in the UK has been to use polymer muds in preference to all other forms of flush fluid. The viscous nature of polymer mud means that lower up-hole velocities are required than for water, and that there is less loss of return, and that as a consequence less water needs to be brought to site.

Scarrow and Gosling (1986) describe the use of polymer mud as a flushing medium, for coring alluvial deposits in the Middle East. Water was readily available, but an important consideration was the need to minimize the quantity of material to be transported to the site. In addition, few rig operatives had experience of using mud flush. To meet these considerations GS 550 viscosifier was adopted for general use, with the addition of Lubtub anti-swelling agent and lubricant for use in clays.

GS 550 is a non-toxic biodegradable synthetic co-polymer, with a molecular weight of around 25 million, which rapidly produces a viscous fluid at very low concentrations in water (Table 5.2 and Fig. 5.13). This fluid can however be pumped at relatively low pressures, an especially useful consideration with wire-line drilling equipment with its restricted annular return space. Although biodegradable, the fluid is stable at high ambient temperatures and the use of a bacteriacide is not required.

Lubtub was used as an addition to drilling fluid in the early stages of the investigation to prevent swelling in the clay horizons which, from their mineralogy, were expected to be expansive and could

have caused problems of swelling cores and squeezing boreholes. It was found, however, that although the boreholes did squeeze this was not sufficient to cause high torque build-up with either the wireline drill string or the conventional SX casing even without Lubtub. Consequently the use of the anti-swelling agent was discontinued.

Table 5.2 GS 550 Marsh funnel viscosities

GS 550/water (kg/m3) (%)		Initial viscosity (Marsh funnel seconds)
1.0	(0.1)	90
0.9	(0.09)	75
0.75	(0.075)	50
0.5	(0.05)	35

Marsh funnel viscosity of Water = 26s ± 0.5s.

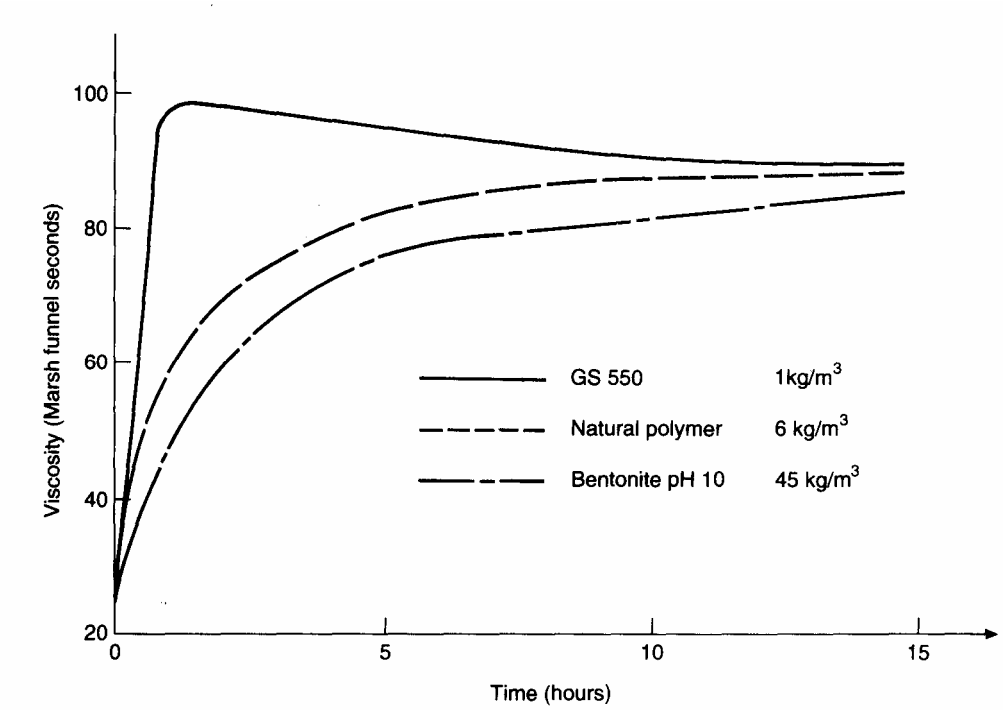


Fig. 5.13 Development of viscosity with time.

MSF

The use of mud has some disadvantages, in that fairly large volumes of water are still required, and some erosion will still occur as a result of the combination of upward flush velocity and viscosity. A seemingly almost ideal flush fluid which has not been greatly used in site investigation is Modified Stable Foam (MSF). MSF consists of a small quantity of water and foaming agent which is injected into the airstream of a low-volume high pressure compressor (Fig. 5.14). The foaming agent may incorporate high molecular weight polymers which increase the foam bubble strength and in this way reduce the required upward flush velocity to as little as 15 m/min. This low up- hole velocity limits hole erosion and reduces the size of air compressor necessary to provide the flush.

In addition to low erosion and good lifting properties, MSF cakes over shales and soft rocks, and rapidly expands into voids and fissures to block them and prevent loss of return. England (1976) gives some examples of the necessary plant outputs required to drill a 375 mm dia. hole with 127 mm dia. drill rods, which is much larger than would normally be required for site investigation purposes:

Outside hole diameter 375 mm ($14^{3/4}$in)
Outside drill rod diameter 127mm (5 in)

Air compressor output 3.5 m3/min (125 cfm)
Water/foaming agent
injector pump output 23 l/min (5 gal/min).

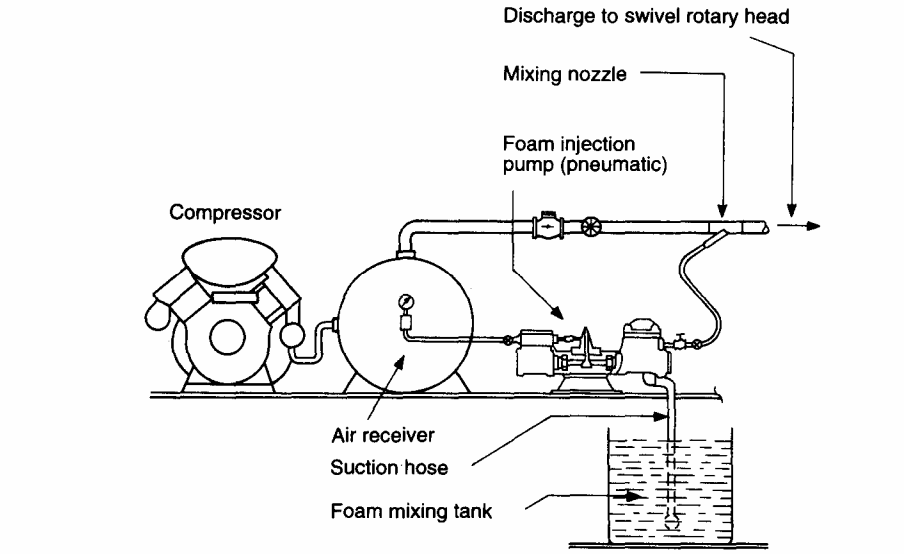


Fig. 5.14 Liagrammatic layout of the pump/compressor circuit for foam flush drilling (England 1976).

Thus the ratio of foam fluid to air is about 1:150, and foaming fluid usually consists of about 1% by volume of foaming agent mixed with water. At the start of drilling the air and foam injection pumps must be adjusted so that the foam emerges from the top of the hole with the appearance of an aerosol shaving cream'. A steady rush of air indicates too much air input, or too little foaming agent/water injection. The foam is channelled to waste after use. The foam and polymer additives are non-toxic and biodegradable and can therefore be used without danger to the environment. One of the disadvantages of both foam and mud flush is that they may make a drillhole unsuitable for permeability testing.

Diamond drill sizes
The use of combinations of letters to identify the size of drilling equipment is at first confusing, but it results from the development of the equipment in the USA, Canada, South Africa and the UK and therefore is widely used.

The earliest sizes E, A, B and N (or about $1^{1/2}$in., 2in., $2^{1/2}$in. and 3m, hole size) resulted from the use of standard steel pipe sizes during the development of equipment by various manufacturers. With the expansion of drilling work before 1930, many problems were found, principally because these sizes were not exact and therefore equipment from different manufacturers could not be interchanged. At a conference held in Chicago in 1929 the sizes of casing and the front end (i.e. bits, catchers and reaming shells) of corebarrels were standardized, and in order to distinguish equipment made to these sizes an X was added after the size letter, for example, EX, AX, BX and NX.

At this stage drill rods were more or less standard, and were not considered. However, the effect of changing the sizes of the drilling equipment caused the Canadian Diamond Drilling Association (CDDA) to introduce larger rod sizes, for which the letter C was added (i.e. AC, NC, etc.). These sizes were experimental, and after some further work in the late 1940s, the US Diamond Core Drill Manufacturers Association (DCDMA) also introduced a larger series of rods to cope with the bigger drilling equipment, which was distinguished by the letter U (for Universal). Finally with the co-operation of both organizations, a common standard was established, which used the suffix, W, for Worldwide (e.g. NW, AW).

In the early stages each size of corebarrel was used with either the E, A, B or N rods (i.e. a BX corebarrel used a B rod). At this stage it therefore became necessary to distinguish barrels which were threaded to match the new W series rods. The corebarrels matching these rods were therefore termed NWX, EWX, etc.

In the immediate post-war period, it became desirable to develop a size which would bridge the gap between rotary drilling and oil well drilling. The size developed and introduced by the CDDA in 1963, HW, is designed for use with both diamond core equipment and with rock roller bits. Later, the British Standards Institution (BS 4019:1966) developed four larger sizes, P, S, U and Z, and based the standards for the smaller sizes on the existing DCDMA and CDDA equipment.

Four standard interchangeable corebarrel designs are available.

1. *WF Series (BS 4019:1974)*. British design, using medium kerf bits, and available in HWF, PWF, SWF, UWF and ZWF sizes. The barrels feature:
    i. face discharge bits;
    ii. double-tube swivel type barrel; and
    iii. knock on catcher box (i.e. friction fit); and are suitable for mud flush.
    This series of barrels is useful for soft formations.
2. *WT Series (CDDA)*. Canadian design, using narrow kerf bits. Sizes are limited to the smaller end of the range, that is: double-tube rigid — EWT, AWT, BWT, NWT, HWT; and double-tube swivel — BWT, NWT, HWT. Because of the narrow kerf design this series of barrels is of particular use in drilling hard dense rock.
3. *WM Series (DCDMA)*. American design, using medium-width kerf bits, with WG core sizes (see below). Screw on catcher box, available with either conventional or face discharge bits. All barrels are double-tube swivel type, available in the following sizes; EWM, AWM, BWM, NWM.
4. *WG Series*. A fully standard version of the WX design, where only the bit, catcher, reaming shell and rod thread were standard. The barrel uses a medium- width kerf bit and is of rigid design, convertible to swivel type in all sizes: double-tube rigid — EWG, AWG, BWG, NWG, HWG.

In addition to the above sizes, metric sizes are available from certain manufacturers such as Craelius (Atlas Copco). This equipment is designated by the hole size it produces (in mm).

In both metric and imperial designs, the size of core obtained varies with corebarrel design.

Figure 5.15 shows corebit sizes, with their respective casings, in diagrammatic form. For further information the reader is referred to BS 4019:1974, Cumming and Wicklund (1980) and information obtainable from diamond core drill manufacturers.

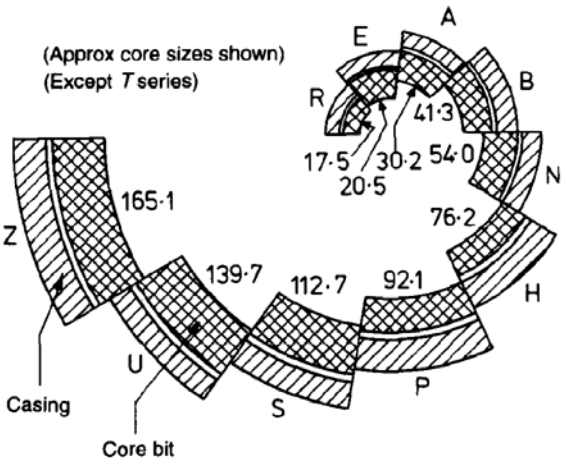
In the UK the conventional view is that soft rocks which may often be susceptible to softening by the flush fluid are better cored by large diameter double-tube swivel type corebarrels with face discharge bits. NWX, HWF, PWF and SWF barrels are most frequently used, in conjunction with NW rods. Thin-walled barrels are most easily damaged by careless handling, and are therefore more appropriate in hard rock conditions or long holes when faster drilling will give significant financial gain.

Drillhole testing

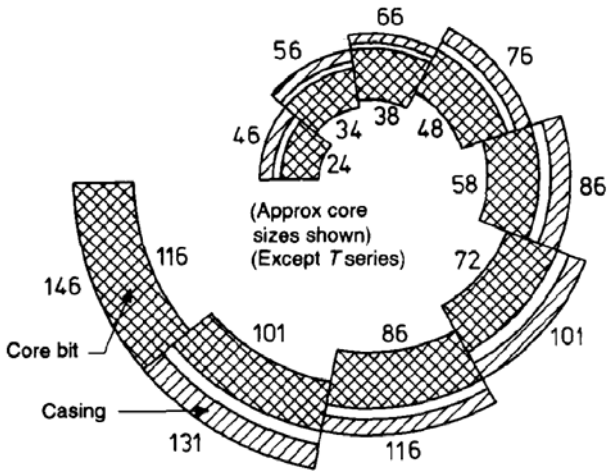
In the UK, the majority of site investigation for civil engineering purposes can be carried out without significant use of rotary drilling techniques. Where rotary drilling is used (for example for reservoirs, tunnels and dams) instrumentation is often very basic.

Where rotary core of any length is required four major types of information will be necessary in order to supplement that available from the core itself. These are:

1. the inclination of the drillhole at various points along its depth;
2. the plan direction of the drillhole at various points along its depth;
3. the orientation of the various runs of core and the discontinuities (fractures, bedding planes, etc.) within each run; and
4. the position of any cavities or highly fractured zones where core recovery has been missed.



General relation of casing to core bit
(From BS4019: Part 1: 1974)



General relation of metric casing to core bits.
(Craelius K_3 face discharge bits with CMS casing)

Fig. 5.15 Casing and core bit sizes (mm).

In general (1), (2) and (3) above are important only in major investigations in rock for projects where both the *in-situ* fracture pattern and orientation are of importance, such as tunnels, deep mines, and major underground excavations. The positions of cavities and missing core are of the utmost importance in all site investigation drillholes, particularly since it is not normally possible to detect whether low percentages of core recovery are caused by poor drilling techniques, low quality rock, or voids.

There are many different types of down-hole instrumentation, a considerable number of which have been discussed by Barr (1977) and Cumming and Wicklund (1980). This section discusses core and hole orientation, and the impression packer.

Orientation
When drillholes are relatively shallow and vertical, deviations from their intended line will not normally be significant unless worn or damaged equipment is used, or the drill string is deflected by harder ground or cavities. Long holes, and particularly angle holes, will suffer from deviations due to layering of hard and soft rock materials and due to the natural tendency of an angled drill string to flatten out as it progresses, particularly when smaller diameter drill rods are used above the corebarrel. In most site investigations it is not worth attempting to control the line of the hole, but it is often important that the hole is surveyed.

The inclination of a drillhole can be determined very simply using the etching tube. A glass culture tube is part filled with a 4% solution of hydrofluoric acid, sealed, placed in a watertight cylinder screwed to the base of the drill rods, and lowered to the desired position in the drillhole. The etching tube should be left in a stationary position for a time equal to that required to lower it down the hole, or a minimum of 30mm. When brought to the top of the hole, the tube is washed and the inclination of the line etched by the surface contact of the hydrofluoric acid to the long axis of the tube can be measured to obtain the angle of drillhole. More conveniently, a *Thompson— Cumming True-dip Etch Chart* (available from J. K. Smit of Toronto, Canada) can be inserted in the tube to overcome capillarity problems (see, for example, Cumming and Wicklund (1980)) (Fig. 5.16).

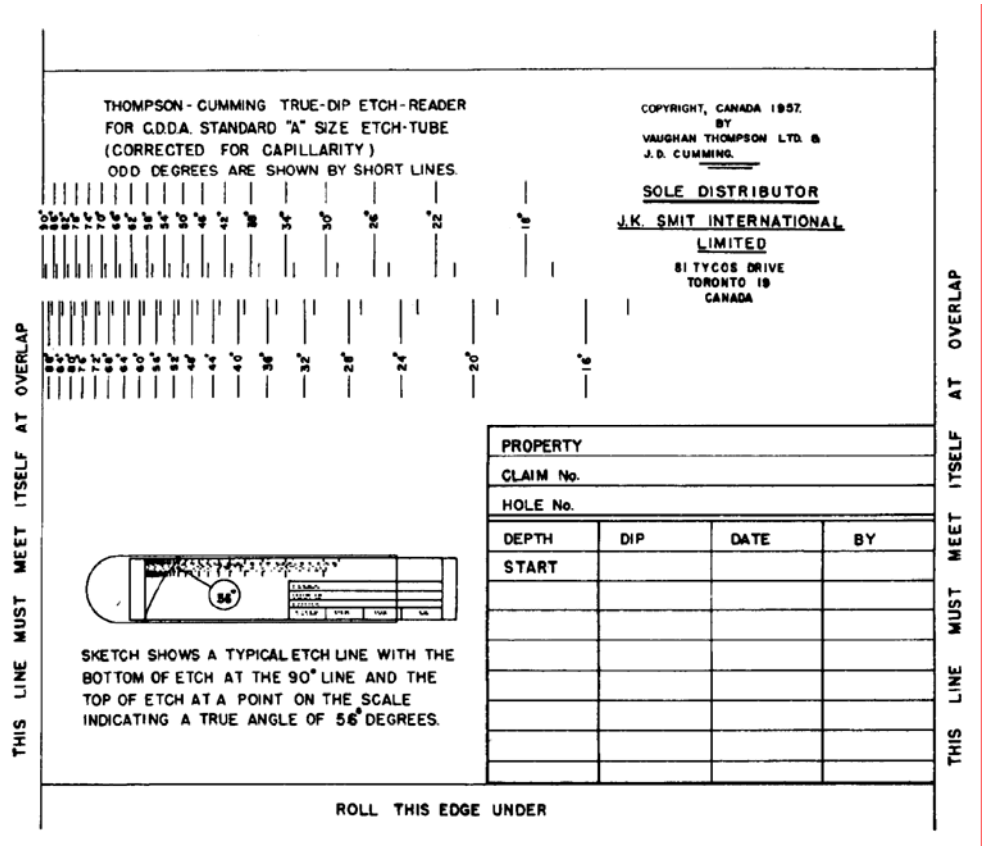


Fig. 5.16 Thompson-Cumming true dip etch reader.

More accurate mechanical inclination measuring systems are available, for example the Eastman International Company (Hannover, Germany) borehole drift indicator, but where more precision is needed, it is normal to require measurements of both dip and direction of borehole discontinuities. Photographic survey instruments such as the Eastman International types A and DT instruments are reliable, but relatively expensive (Barr 1977). The Tro-Pari Surveying instrument (Trotter—Pajari Instruments, Ontario, Canada) is a mechanical device which incorporates a compass mounted on gimbals in a clockwork timing and clamping mechanism. When lowered down the hole the compass needle is free and the compass itself hangs plumb in the gimbals. When the preset time elapses, the

mechanical system locks in position and fixes the compass and dip scales for examination when brought to the surface. Direct readings of inclination and orientation to about 1° can be made.

The use of the survey instruments described above gives an idea of the position of a drillhole, but not of the orientation of the core within it. In good rock the Atlas Copco Craelius core orientator, made in Daventry, England (for example, see Hoek and Bray (1974)) can be lowered to the bottom of a drillhole in a corebarrel to measure the fracture plane on the end of the core stub. The core orientator is a mechanical device which incorporates six self-locking prongs which take up the profile of the rock stub at the base of the hole, and a steel ball bearing in a cylinder which is impressed on an aluminium plate to mark the bottom of the device. The core orientator moves up the corebarrel as drilling proceeds, and allows adequate alignment of the top of the run, provided discontinuities are not approximately normal or parallel to the direction of drilling, and the hole is not within 10^{o} of vertical (Barr 1977).

Borehole impression packer

A more satisfactory device, in that it combines mechanical simplicity and ruggedness with a complete record of the inclination and direction of the discontinuities traced on the walls of the drillhole, is the borehole impression packer (Hinds 1974; Barr and Hocking 1976). The device consists of an inflatable rubber packer overlain by two stainless steel shells, suitably curved to conform to the perimeter of hole, covered with a PVC foam over which is laid sheets of thermoplastic film (Parafilm ‘M’). The packer can best be orientated by incorporating a device such as the Tro-Pari compass at its base. When lowered to the desired section of the drillhole the packer is inflated and the thermoplastic film forced against the sides of the hole. The resilient foam forces the thermoplastic film into any voids or fissures, and, in soft rocks material on the sides of the drillhole, may adhere to the film. After about 1 mm at the required pressure the packer can be deflated and returned to the surface, providing a permanent record of the hole. This device appears ideally suited for site investigation purposes and is currently in regular use providing discontinuity data for slope stability analysis in Hong Kong (Fig. 5.17).

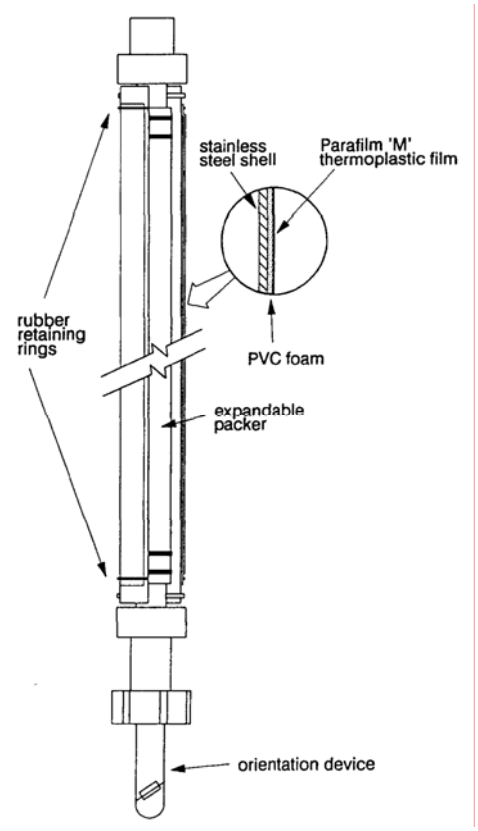


Fig. 5.17 Borehole impression packer.

PROBING

A wide range of dynamic and static penetrometers are available, with different types being used in different countries. However, the objective of all probing is the same, namely to provide a profile of penetration resistance with depth, in order to give an assessment of the variability of a site. Probing is carried out rapidly, with simple equipment. It produces simple results, in terms of blows per unit depth of penetration, which are generally plotted as blowcount/depth graphs.

The Mackintosh probe

The Mackintosh prospecting tool consists of rods which can be threaded together with barrel connectors and which are normally fitted with a driving point at their base, and a light hand-operated driving hammer at their top (Fig. 5.18). The tool provides a very economical method of determining the thickness of soft deposits such as peat.

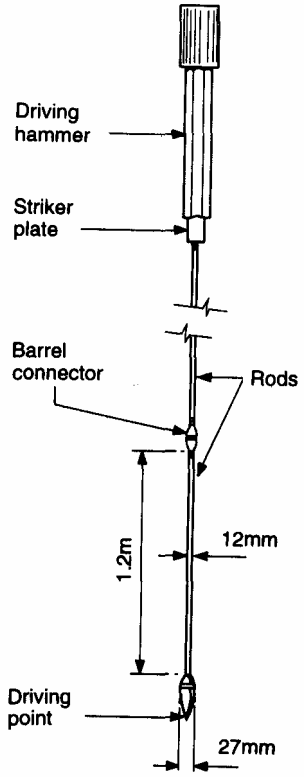


Fig. 5.18 Mackintosh probe.

The driving point is streamlined in longitudinal section with a maximum diameter of 27mm. The drive hammer has a total weight of about 4kg. The rods are 1.2 m long and 12mm dia. In the UK the device is often used to provide a depth profile by driving the point and rods into the ground with equal blows of the full drop height available from the hammer: the number of blows for each 150 mm of penetration is recorded. When small pockets of stiff clay are to be penetrated, an auger or a core tube can be substituted for the driving point. The rods can be rotated clockwise at ground level by using a box spanner and tommy bar. Tools can be pushed into or pulled out of the soil using a lifting/driving tool.

Because of the light hammer weight the Mackintosh probe is limited in the depths and materials it can penetrate.

Dynamic probing

Many dynamic probing tests appear in the literature, but in principal all are used for the same purpose, and in a similar way. Only the details of the apparatus differ. Dynamic probing tests were the subject of standardization in the Report of the SubCommittee on the Penetration Test for use in Europe (ISSMFE 1977), and are currently standardized in the UK (BS 1377: part 9, clause 3.2), and Germany (DIN 4094, parts 1 and 2).

Dynamic probing involves driving a solid cone into the ground, using repeated blows of a hammer with a fixed mass falling through a fixed distance. The hammer strikes an anvil which is rigidly fixed to rods which are of a smaller diameter than the cone, and transmit the hammer energy to it. Typically the rate of driving is between 15 and 30 blows per minute. This is achieved using a small, purpose built rig (Fig. 5.19). As the cone is driven into the ground the number of blows required to drive it each increment (typically 100 mm) is recorded. The blow count is plotted against depth to provide a more-or-less continuous profile of penetration resistance with depth. Rods are generally quite short, and as each new rod is added there is typically a requirement that it should be turned through one or more revolutions, in order to reduce friction. For some tests the torque necessary to turn the rod string is recorded, and these records can be used to judge the build-up of rod friction.

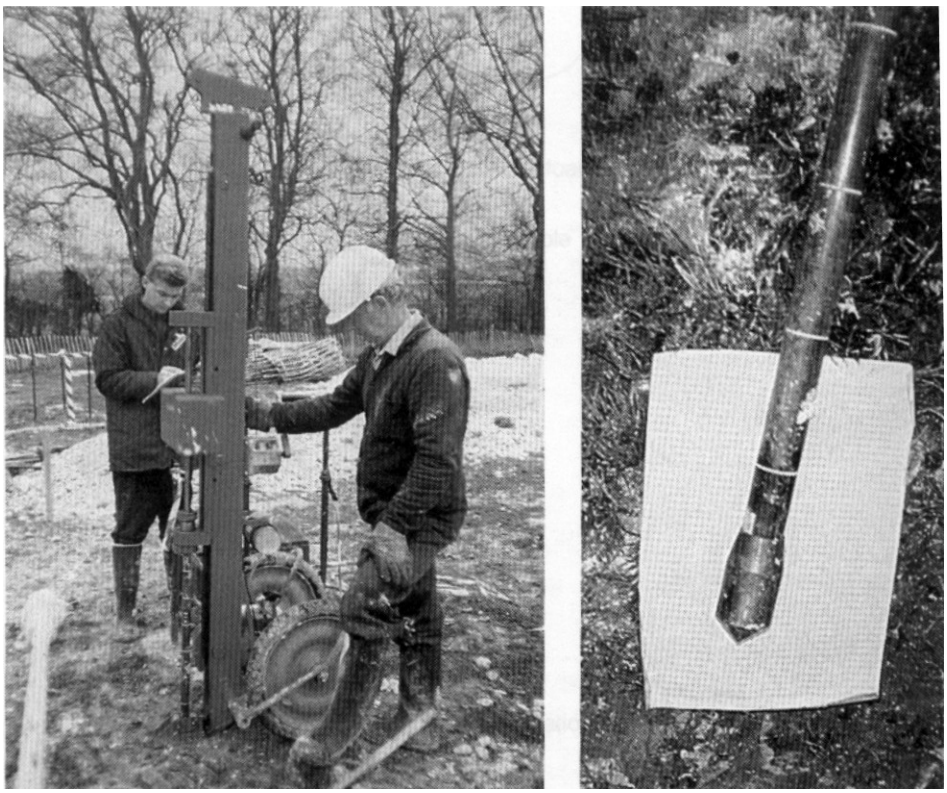


Fig. 5.19 Dynamic probing to DIN 4094.

Any build-up of friction between the rods and the surrounding soil will clearly influence the measured penetration resistance, and so several tests make provision for this to be reduced, either by pouring mud or water down the outside of the rods, or by pumping mud down the rods to a flush port just above the tip, or by the use of casing. In the authors' experience these measures are seldom used, since they complicate an otherwise attractively simple and rapid method of exploration.

For fine-grained soils (i.e. sands and finer) the results of probing to different standards can be compared by considering the work done per unit swept volume:

$$H_e = \frac{Mgh}{la} \tag{5.1}$$

where H_e = hammer energy/blow/unit volume (J/mm^3), M = mass of hammer (kg), g = gravitational acceleration, h = drop height (m), l = distance of standard cone drive (e.g. 100 mm), and a = cross-sectional area of cone (mm^2).

The hammer energy/blow/unit volume (see Table 5.3) can be multiplied by the blow count to give an average hammer energy/unit volume (referred to as 'hammer energy' in DIN 4094), for example:

$$A = H_e n_{100} \tag{5.2}$$

The International Society for Soil Mechanics and Foundation Engineering's *Report of the Sub-Committee on the Penetration Test for use in Europe* suggests that the results from different types of dynamic probing may be presented as resistance values q or *rd* in units of stress (e.g. kPa) such as:

$$r_d = \frac{Mgh}{ae} \tag{5.3}$$

or

$$q_d = \left(\frac{M}{M + M'}\right)\left(\frac{Mgh}{ae}\right) \tag{5.4}$$

where M = mass of hammer, M' = total mass of drive rods, anvil and guide rods, and e = average penetration per blow.

Figure 5.20 shows the results of dynamic probe tests carried out to assess the variability of a high porosity chalk site.

Table 5.3 Weights, dimensions and hammer energies for various dynamic probes

Reference	Hammer mass, M (kg)	Drop height, h (mm)	Cone diameter, d (mm)	Cone area, a (cm^2)	Drive length, 1 (mm)	Rate (blows/ min.)	Hammer energy, H_e (J/mm^3 x 10^{-4})	Designation
BS 1377:1990	50±0.5	500±10	43.7 ±0.3	15	100	15—30	16.4	DPH DPSH
	63.5 ± 0.5	750 ± 20	50.5 ± 0.5	20	100	15—30	23.4	
DIN 4094	10	500	25.2	5	100	15—30	9.8	LRS5
	10	500	35.6	10	100	15—30	4.9	LRS1O
	30	200	35.6	10	100	15—30	5.9	MRSAIO
	30	500	35.6	10	100	15—30	14.7	MRSB1O
	50	500	35.6	10	100	15—30	24.5	SRS1O
	50	500	43.7	15	100	15—30	16.4	SRSI5
ISSMFE (1977)	63.5	750	62	30	200	20—60	7.8	DPA
	63.5	750	51	20	200	20—60	11.7	DPB

Swedish weight sounding

The Swedish weight-penetrometer consists of a screw-shaped point (Fig. 5.21), rods, weights, and a mechanism or machine to rotate the rods. The device has rarely, if ever, been used in the UK. It is used as a static penetrometer in soft cohesive soils. When the static penetration resistance exceeds 1 kN the penetrometer is rotated in addition to being loaded vertically. Under these conditions it can then penetrate stiff clays and dense sands.

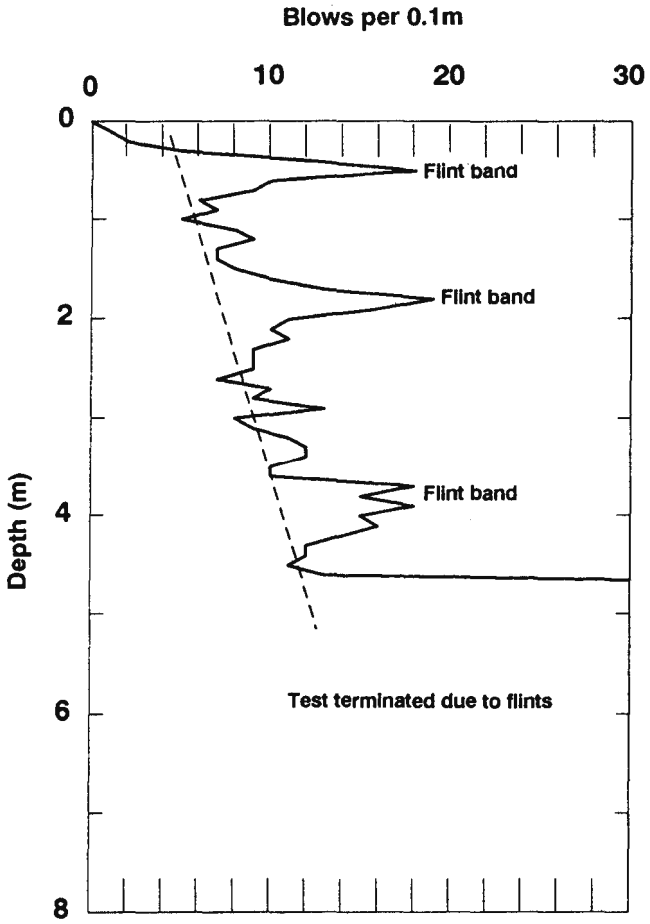


Fig. 5.20 Dynamic probe test results for a chalk site.

The point is attached to rods, and is then loaded in steps using the standard loads in Table 5.4.

Table 5.4 Standard loads for Swedish weight-penetrometer

	Mass (kg)
Rod weight alone	0
Clamp	5
Single 10kg weight	15
Two 10kg weights	25
Two 10kg weights +25 kg weight	50
Two 10kg weights + two 25 kg weights	75
Two 10kg weights + three 25 kg weights	100

The penetrometer is used as a static penetrometer in soft soils, with the loads being added or removed to give a penetration of about 50mm/s. Although loads are normally added, it is sometimes necessary to remove them, for example after penetrating a desiccated crust. Once the static penetration resistance exceeds 1 kN, or the penetration rate under 1 kN load is less than 20mm/s, the rod is rotated. The load of l kN is maintained, and the number of half turns required to give 0.2 m of penetration is measured. The rate of rotation should be between about 30 r.p.m., when carrying out mechanized weight sounding. Figure 5.22 shows an example of the results obtained.

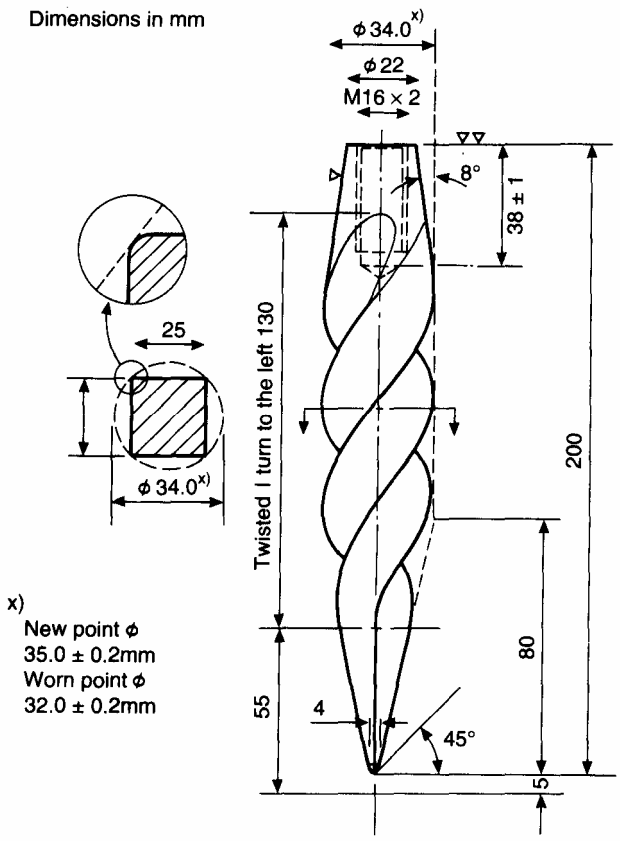


Fig. 5.21 Swedish weight sounding point.

EXAMINATION *IN SITU*

Trial pitting

Trial pits provide the best method of obtaining very detailed information on strength, stratification, pre-existing shear surfaces, and discontinuities in soil. Very high quality block samples can be taken only from trial pits.

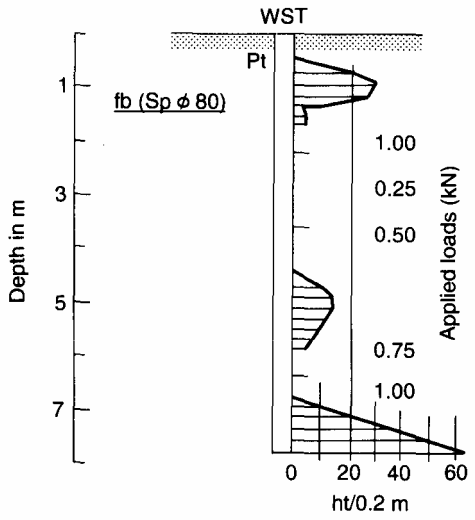
It is as well to note that every year many people are killed during the collapse of unsupported trenches. Remember to be careful — do not enter trenches or pits more than 1.2m deep without either supporting the sides or battering back the sides. Even so, if a pit is dug and remains stable without support, a quick means of exit such as a ladder should be provided.

Trial pits may be excavated by either hand digging or machine excavation. In general, machine excavation is used for shallow pits, whereas hand excavation is used for deep pits which must be supported. In the limited space of a trial pit, which is often 1.5m x 3m in plan area at ground level, it is usually impossible to place supports as machine excavation proceeds. Shallow trial pits provide a cheap method of examining near-surface deposits *in situ*, but the cost increases dramatically with depth, because of the need to support.

Shallow trial pits can be excavated by wheeled offset back-actor excavators such as the JCB 3c, MF50, etc., which have a digging depth of only about 3.5—4.0m, and may not be able to move easily across wet steeply sloping sites. Deeper pits, or pits where access is difficult can be excavated by 360° slew-tracked hydraulic excavators. Machine types commonly used in trial pitting are the JCB 6c,

Hymac 580c, and Poclain L60. These machines have a digging depth of about 6 m, and an available digging force about 50—100% greater than the back-actor type excavator.

Very deep pits can only be economically excavated by machine if their sides are very stable, when they can be dug to the required depth without requiring support. In some soil conditions (for example, chalk) it has been found possible to excavate pits to a 12 m depth using a 22RB tracked rope-operated excavator with a heavy rope grab, but under these conditions an elaborate safety cage must be provided to protect engineers and geologists engaged upon logging the face of the excavation. An example of the resulting engineer's trial pit record is shown in Fig. 5.23.



WST	Weight sounding test
ht/0.2m	Number of halfturns per 0.2m of penetration
Pt	Dry crust of clay
fb (Sp ϕ 80)	Preboring to this level with 80mm diam auger

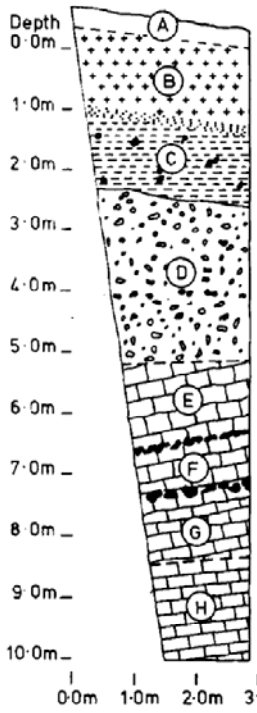
Fig. 5.22 Example of the results of Swedish weight sounding (ISSMFE 1977).

Large bored shafts

Shafts bored by piling rigs may be used for *in-situ* examination of soil. The shaft is usually auger bored, abut 1.00m dia., and inspected from a cage lowered by crane. The cage is just big enough to hold a man, and is normally equipped with air, light and telephone.

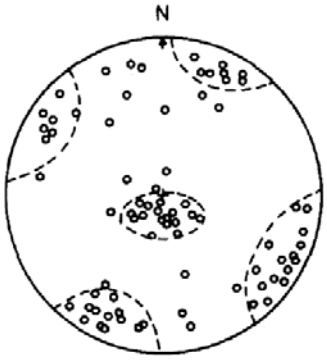
There are several problems with this method. First, the equipment is very expensive to hire, particularly as it may have to stand idle while inspection of the hole is carried out. Secondly, the action of auguring creates a smear zone around the edge of the hole, which leads to difficulties when trying to assess the soil description. In addition, the taking of block samples is very difficult because of the extremely confined working space, and discontinuities in rock are less easily recognized and recorded than in a square pit.

In the two instances where this method is known to have been used, the material was chalk. In one a 46 m deep road cutting was proposed, and in the other several 22 m wide silo foundations were being investigated. The method is, however, relatively common in South Africa.



Horizon No	Depth from/to	Chalk grade	Engineering description
A	0·0/0·3	-	Thin organic top soil.
B	0·3/1·65	-	FILL: Very stiff to hard, light, white, brown, structureless made ground consisting of very well compacted chalk and angular f.m.c. flints in a silty chalk matrix.
C	1·65/2·90	-	FILL: Stiff dark brown structureless CLAY with f.m.c. blue grey flint gravel and some chalk fragments.
D	2·90/4·95 3·1 3·9 4·0	V	Very stiff to hard off white iron stained coarse CHALK GRAVEL with some f.m.c flints in a matrix of chalky silt and clay m.c. FLINT bands.
E	4·95/5·70 5·70	IV	Very stiff to hard extremely friable white highly fractured CHALK with some f.m.c. flints. Material insitu. ie. not transported Continuous blue/grey FLINT band
F	5·70/6·70 6·70	IV	Very stiff to hard extremely friable white with fe staining highly fractured CHALK with m c blue grey flints Some fractures infilled with clay. Continuous coarse blue grey FLINT band.
G	6·70/8·05 8·05	III	Hard non friable off white iron stained blocky CHALK well jointed and fractured some infilled with clay m c flints occurring in bands. Almost continuous coarse blue/grey FLINT band.
H	8·05/10·0	III to II	Hard blocky to massive white CHALK jointed and fractured.

(a) One of the detailed logs produced by geologists to describe soil types, fracture spacing, bedding, weathering and so on



(b) Dip and strike of exposed joints, bedding planes, fractures plotted on a polar net

Fig. 5.23 Results obtained from a l0m deep pit in Upper Chalk in Hampshire (Anon 1974).

Tunnels and drifts

This method is extremely expensive, and most projects do not merit its application. The most common use of tunnels and drifts is during preliminary exploration for underground power stations, where it is necessary to determine the in-situ stress regime in order to design the roof of the main hail.

TV and borehole cameras

TV and borehole cameras can be placed inside a relatively small hole (75—150mm) and can, therefore, be used with conventional drilling methods to examine deep features.

TV cameras are usually used to examine the sides of a borehole for jointing and other features, and to investigate the extent of old mine workings. Simple borehole cameras can be constructed by placing fairly conventional photographic equipment within a drill barrel (for example see Anon. 1970), but this type of equipment is only of use in examining subsurface cavities, because of the focal length of

the apparatus. Trantina and Cluff (1964) have described a borehole camera which overcomes the focal length problem by using a vertically aligned camera viewing a conical mirror. This apparatus can be used to examine the joint patterns on the walls of an NX drillhole (Fig. 5.24).

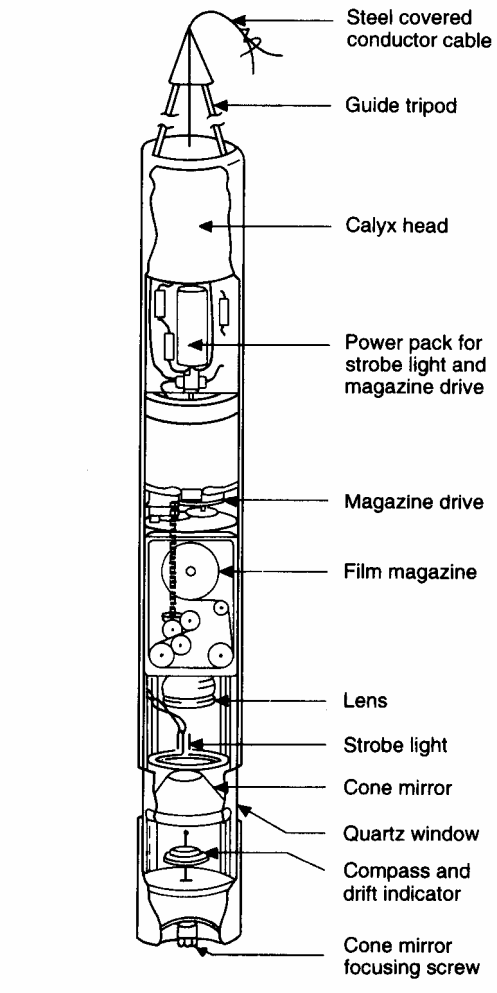


Fig. 5.24 NX borehole camera (after Trantina and Cluff, 1964).

Optical methods of examining the sides of boreholes or drillholes can only be relied on under the most favourable conditions, when the hole is dry. These devices cannot give results in the muddy conditions which normally exist in water-filled holes. They are often expensive to use, and it is therefore doubtful if their use is worthwhile until it is known that the groundwater lies below the level of interest.

Chapter 6

Sampling and sample disturbance

INTRODUCTION

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

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

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

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

SAMPLE SIZES

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

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

Representative particle sizes

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

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

Table 6.1 Sample size necessary for particle size distribution tests

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

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

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

Rate of consolidation

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

Rowe (1968b) made the following conclusions.

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

$$c_v = \frac{k}{m_v \gamma_w} \tag{6.1}$$

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

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

Undrained shear strength

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

Table 6.3 Sample size effects: fissured London clay

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

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

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

Required volume of material for testing programme

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

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

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

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

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

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

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

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

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

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

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

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

* Sufficient material to give the stated mass of fines.

†Fine-grained 90% passing 2mm: medium-grained 90% passing 20mm: coarse-grained 90% passing 37.mm.

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

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

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

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

SOIL DISTURBANCE

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

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

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

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

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

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

Table 6.6 Principal causes of soil disturbance

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

Stress relief

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

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

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

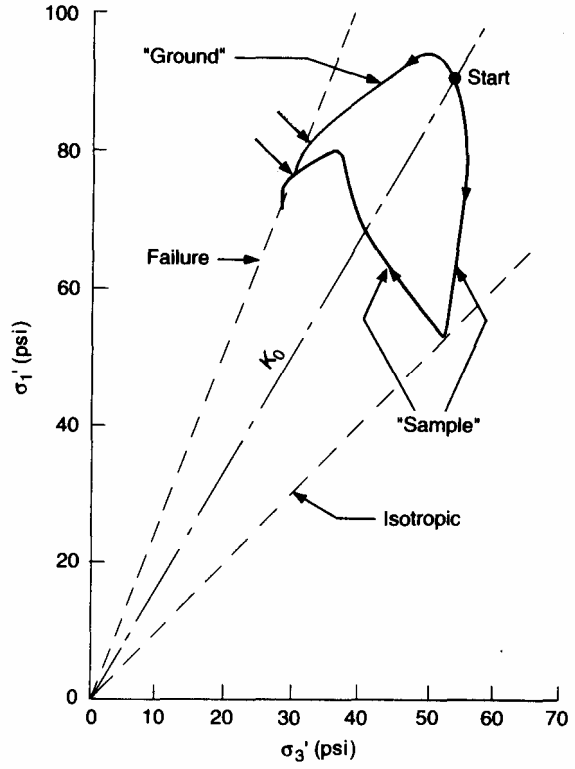


Fig. 6.1 Stress paths for 'ground' and 'sampling' after Skempton and Sowa (1963).

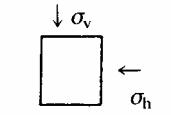
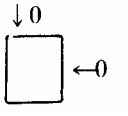
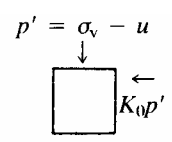
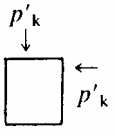
Swelling

Swelling occurs as a consequence of stress relief. In response to the reduction of applied total stresses, the pore water pressures in a soil will reduce and may normally be expected to become negative. If the

soil is coarse-grained, it will have a high coefficient of permeability and a large average pore size, and water or air will rapidly penetrate it and dissipate the negative pore pressures. Thus, with total and effective stresses reduced to zero, a granular soil has little strength and is very difficult to sample or prepare for laboratory testing.

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

Table 6.7 Stress changes occurring in a saturated clay

Stresses	Soil in ground	After sampling
Total stresses		
Pore pressure	$+ u_0$	$+ u_k$
Effective stresses		 $= -u_k$

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

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

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

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

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

For a saturated clay B=1, therefore:

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

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

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

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

therefore:

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

Now:

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

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

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

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

Table 6.8 Differences between predicted and observed results for Weald clay

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

Compaction

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

Soil disturbance during drilling

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

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

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

Table 6.9 Recommendations for the use of fluid filled boreholes

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

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

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

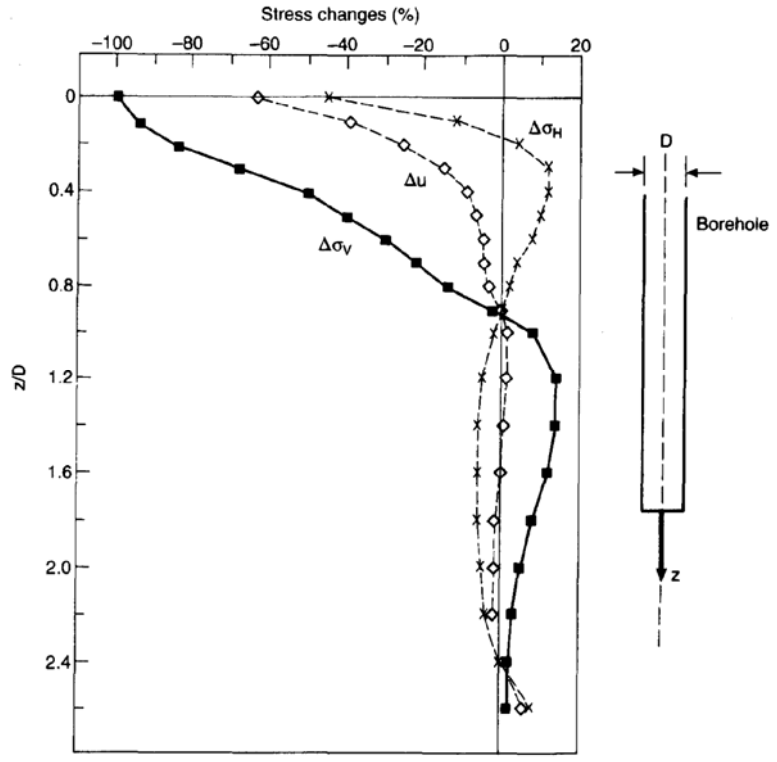


Fig. 6.2 Stress changes below the bottom of a borehole (modified from Galle and Wilhoit (1962) by Hopper (1992)).

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

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

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

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

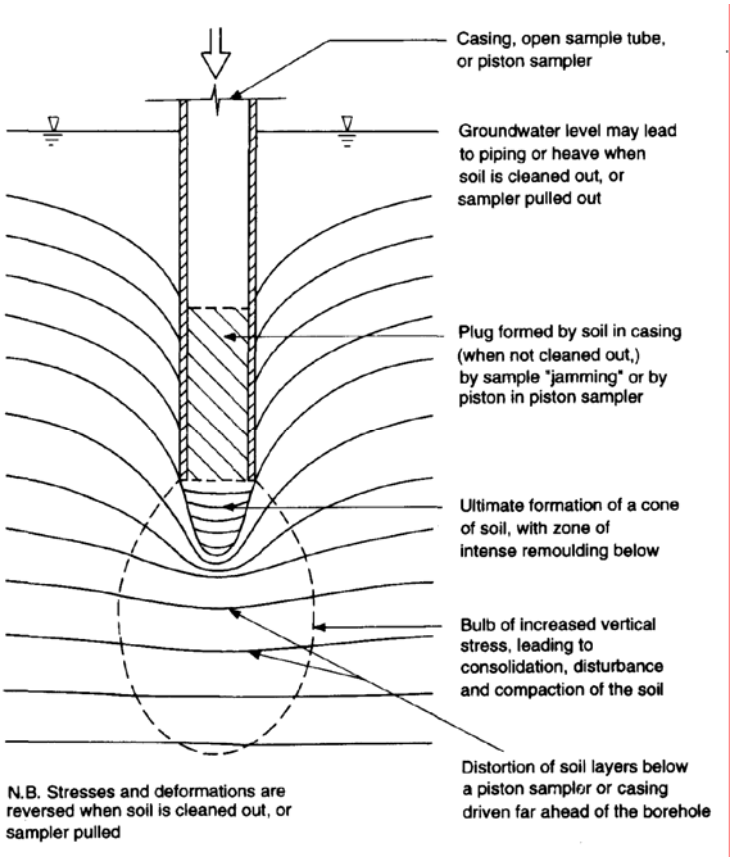


Fig. 6.3 Displacement of soil beneath casing or a sampler tube (largely after Hvorslev 1949).

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

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

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

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

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

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

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

The most serious effects of piping occur because it is normal to use *in situ* tests to determine the design parameters, such as allowable bearing pressure, for a granular soil. In a borehole, the most common test would be the 'Standard Penetration Test' (see Chapter 9), where a 50mm dia. tube is driven into the soil at the bottom of the borehole by repeated blows of a standard weight falling through a fixed distance. The number of blows necessary to drive the tube approximately 300mm is known as the SPT N value, and is used empirically to obtain various soil properties. Piping reduces the density of the soil at the base of the hole, and can therefore give completely false N values; for example, N values have been observed to decrease from 25 blows/300mm to 8 blows/300 mm in sand,

which might lead to an unnecessary reduction in allowable bearing pressure for footings from about 250 kN/m^2 to 80kN/m^2. Sutherland (1963) observed the results shown in Fig. 6.4, where piping appears to have reduced N values by a factor of three or four.

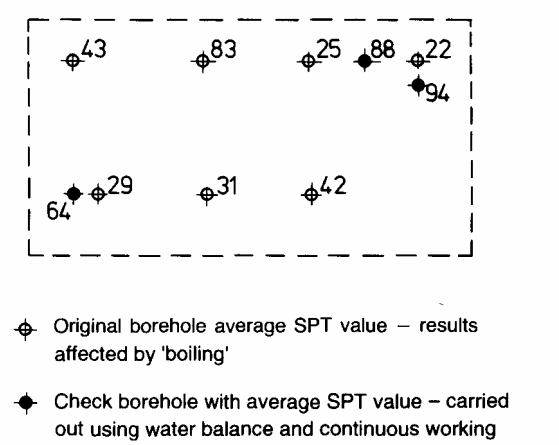


Fig. 6.4 Effects of boiling on SPT 'N' values in fine to medium sand (Sutherland 1963).

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

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

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

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

Stabilization of the sides of boreholes is essential in soils which may collapse or slough. It may be carried out by a variety of methods, the most common of which use water, mud, or casing. Water stabilization is the least effective method, and works by reducing the stress level decreases on the sides of the hole. Further benefits come from the elimination of groundwater flow into the sides of the borehole. Water stabilization may work well in soft cohesive alluvial deposits, but it is not successful

in a wide range of ground conditions. In partially saturated soils the loss of strength may encourage collapse, and in stiff fissured cohesive soils above the water-table the rate of swelling will be increased.

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

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

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

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

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

Soil disturbance during sampling

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

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

This section considers only block sampling and tube sampling.

Block sampling

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

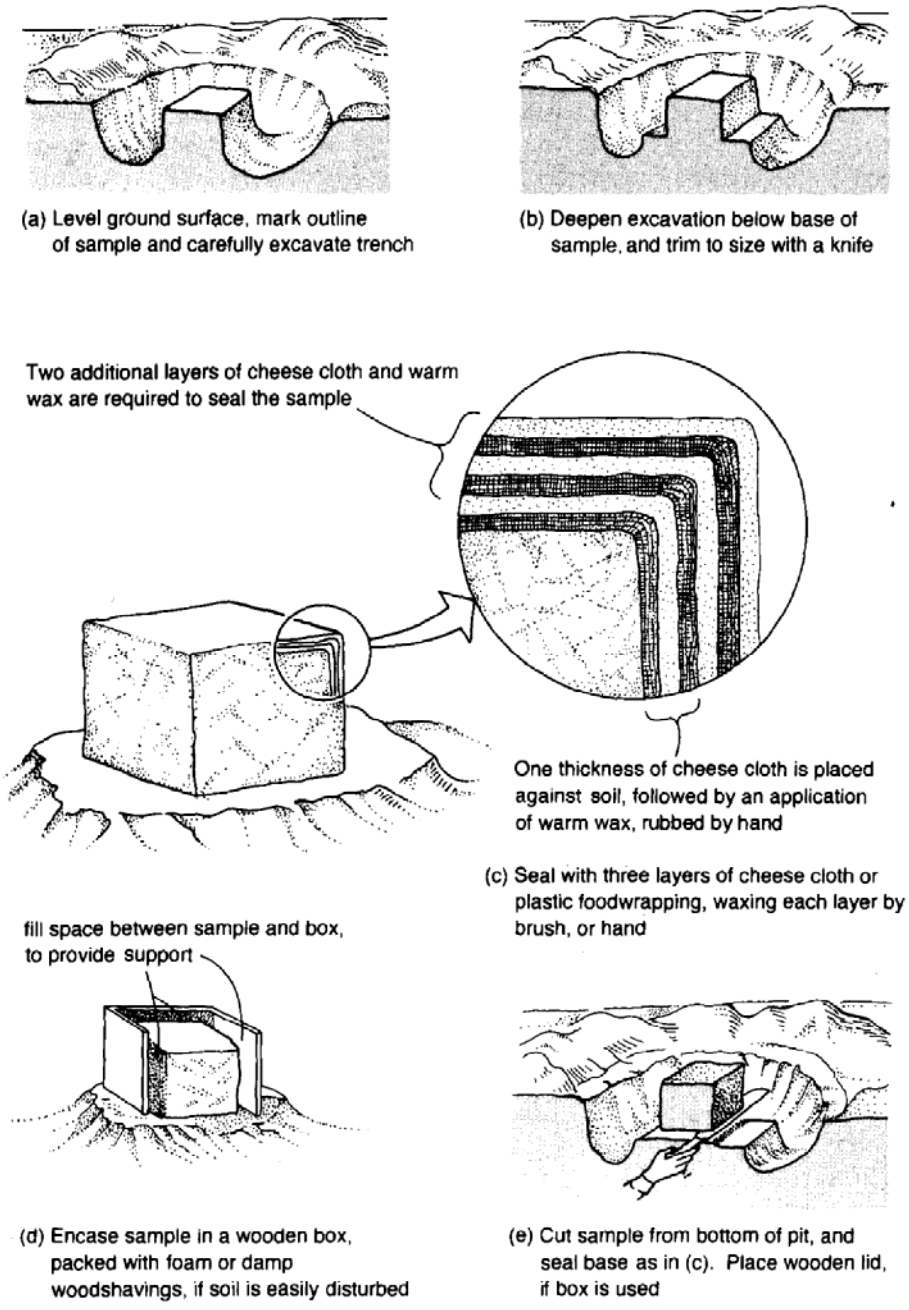


Fig. 6.5 Block sampling in a trial pit.

Trial pits are normally only dug to shallow depths, and shafts and large-diameter auger holes tend to be expensive. Therefore block samples have not traditionally been available for testing from deep deposits of clay. In the past decade, however, there has been an increasing use of rotary coring methods to obtain such samples. When carried out carefully, without displacing the soil, rotary coring

is capable of producing very good quality samples. When the blocks are cut by hand then obviously the pit will be air-filled, but when carried out in a borehole it will typically be full of drilling mud.

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

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

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

Tube sampling

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

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

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

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

Baligh *et al.* (1987) and Siddique (1990) applied the undrained strain paths deduced from Baligh’s strain path method to reconstituted normally consolidated unaged laboratory specimens, and observed the resulting stress paths. Hajj (1990) carried out tests on normally consolidated and overconsolidated reconstituted kaolin. Hopper (1992) carried out similar work on reconstituted overconsolidated unaged clay (OCR = 3.7), and in addition tested high-quality (Sherbrooke and Laval) samples of intact lightly overconsolidated estuarine clay. These tests have shown that for normally and lightly overconsolidated soils the stress paths during tube sampling are of the form shown in Fig. 6.7. Only very small strains are necessary to cause severe disturbance to unaged reconstituted normally consolidated clays. Lightly overconsolidated (OCR = 1.5) structured natural clays appear to be able to withstand axial strain

excursions of up to ±0.5% without significant loss of structure. Both normally and lightly overconsolidated soils suffer very large decreases in mean effective stress during tube sampling. More heavily overconsolidated clays appear to suffer little in the way either of destructuring or of effective stress change.

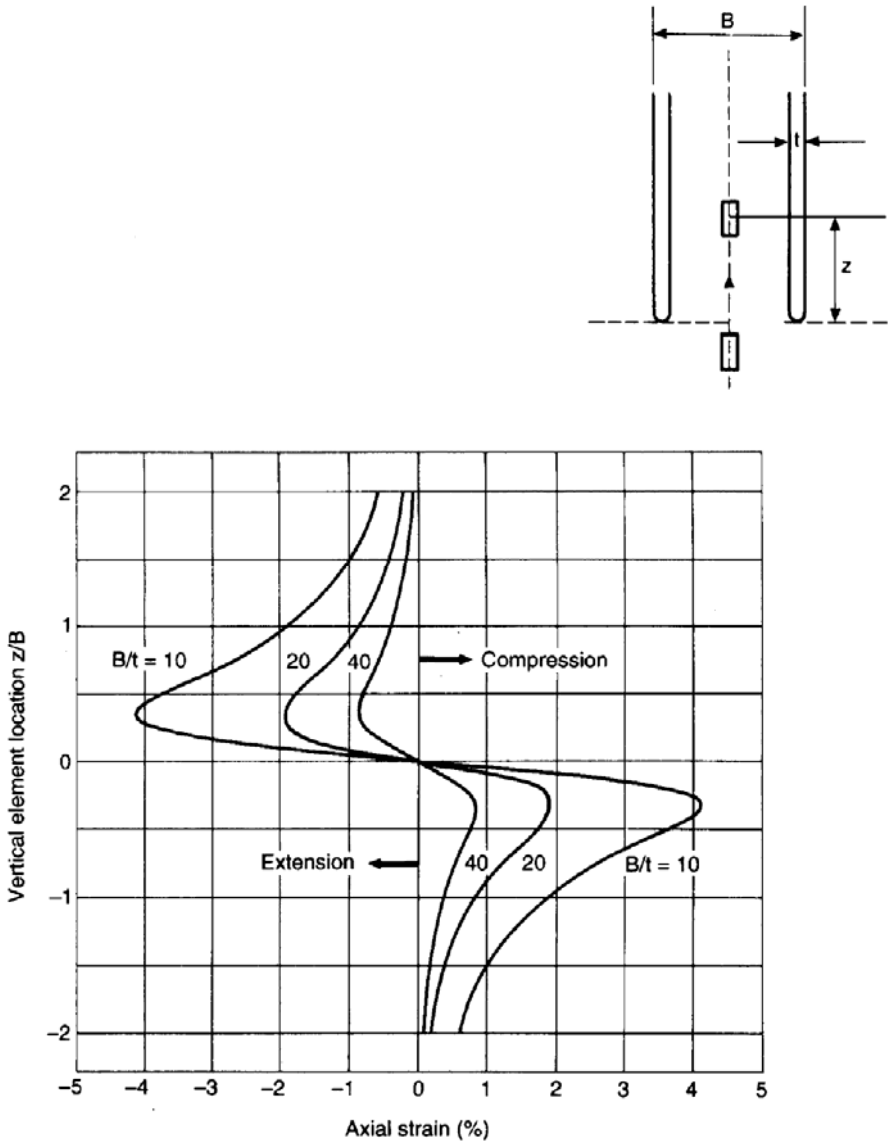


Fig. 6.6 Axial strain history at the centre-line of a simple sampler (after Baligh 1985).

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

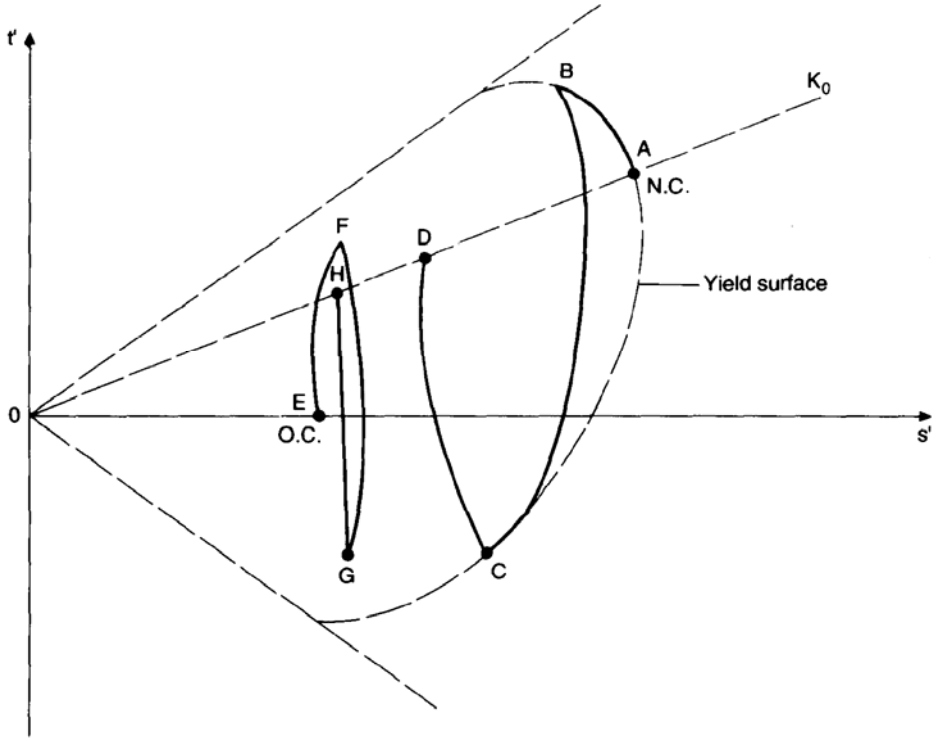


Fig. 6.7 Stress paths induced by tube sampling on normally consolidated, and lightly overconsolidated clays

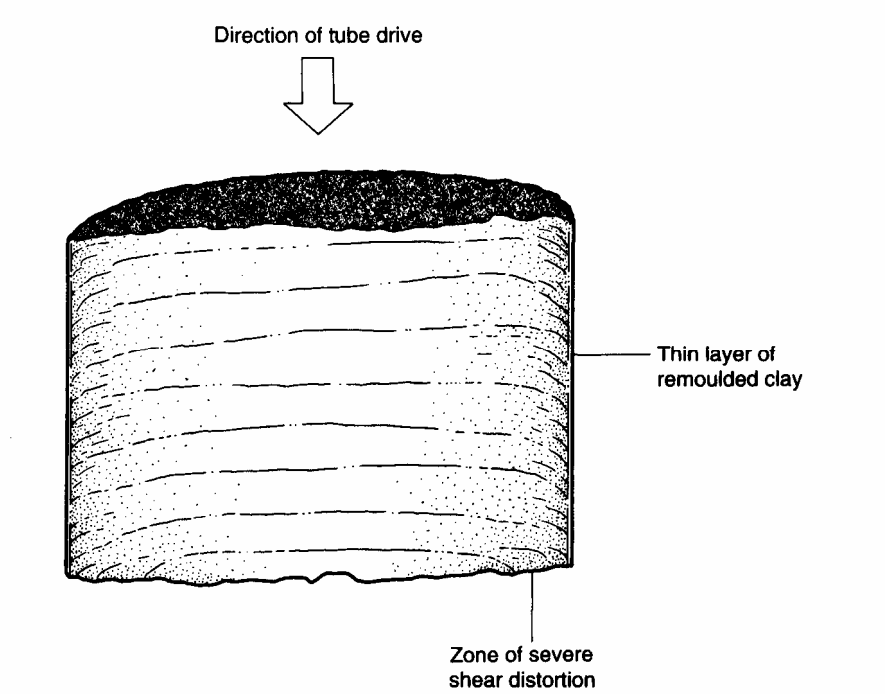


Fig. 6.8 Sketch of shear distortions induced in laminated heavily overconsolidated London clay by tube sampling.

Figure 6.9a shows estimates of the effects of tube sampling on the mean effective stress of London clay, shown by comparing the effective stresses in U100 tube samples (Fig. 6.11) with those in block samples (Fig. 6.5) from a similar depth in the London clay (about 22m below ground level) (Chandler

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

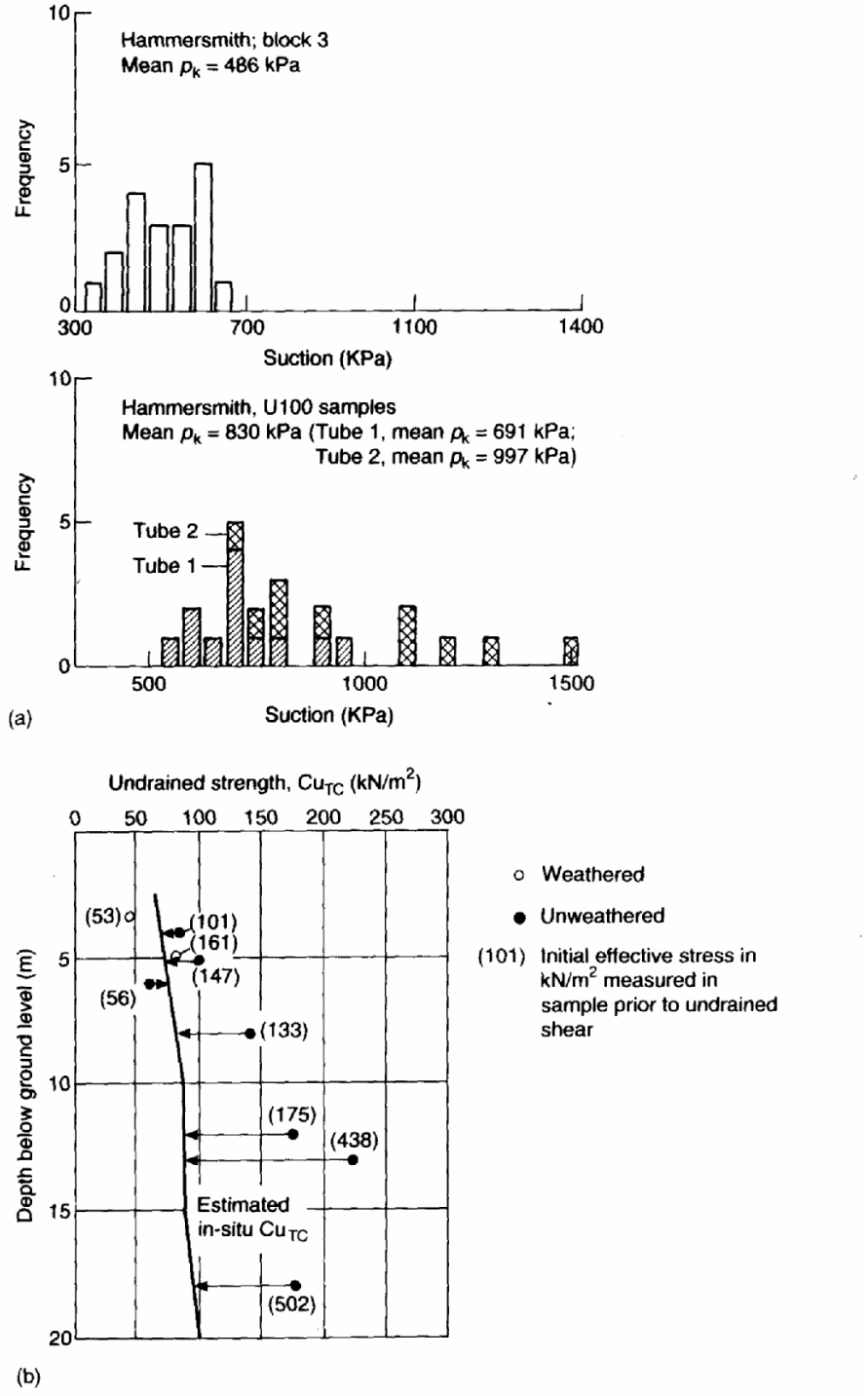


Fig. 6.9 Increases in effective stress in London Clay induced by tube sampling (Chandler, Harwood and Skinner 1992), and their effect on undrained shear strength (Hight 1986).

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

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

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

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

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

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

AREA RATIO

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

$$area\ ratio = \frac{D_e^{\,2} - D_i^{\,2}}{D_e^{\,2}} \tag{6.9}$$

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

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

CUTTING EDGE TAPER ANGLE

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

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

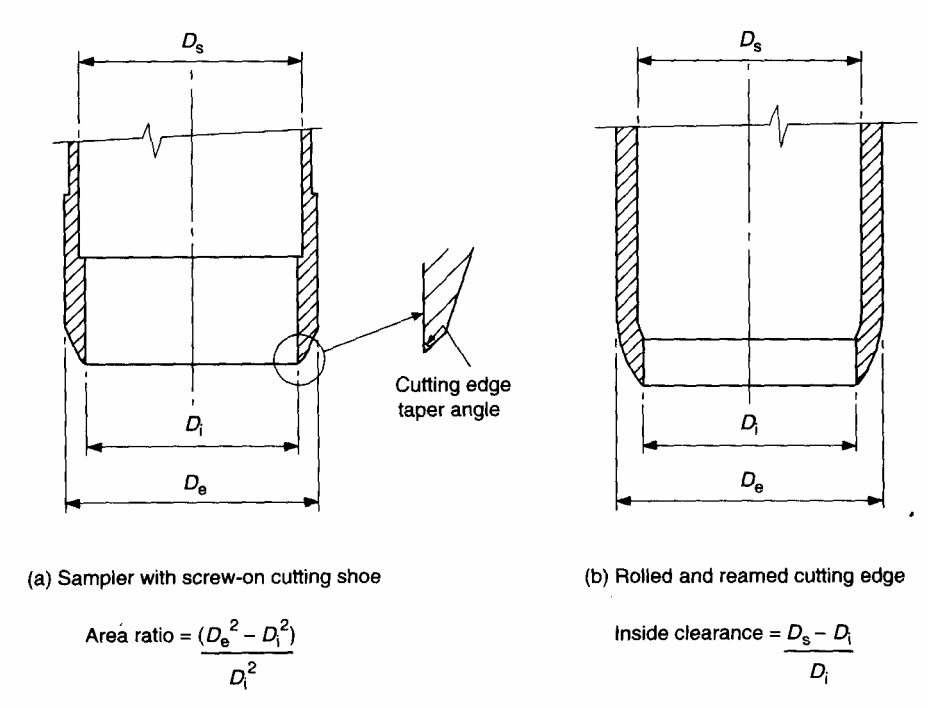


Fig. 6.10 Definition of area ratio and inside clearance.

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

Table 6.10 Combinations of area ratios and cutting edge taper

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

INSIDE CLEARANCE AND L/D RATIO

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

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

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

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

$$A = \pi DL\alpha c_u \tag{6.10}$$

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

The bearing capacity of the soil beneath the tube is:

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

SAMPLE DRIVING METHODS

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

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

Table 6.12 Driving methods (Hvorslev 1949)

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

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

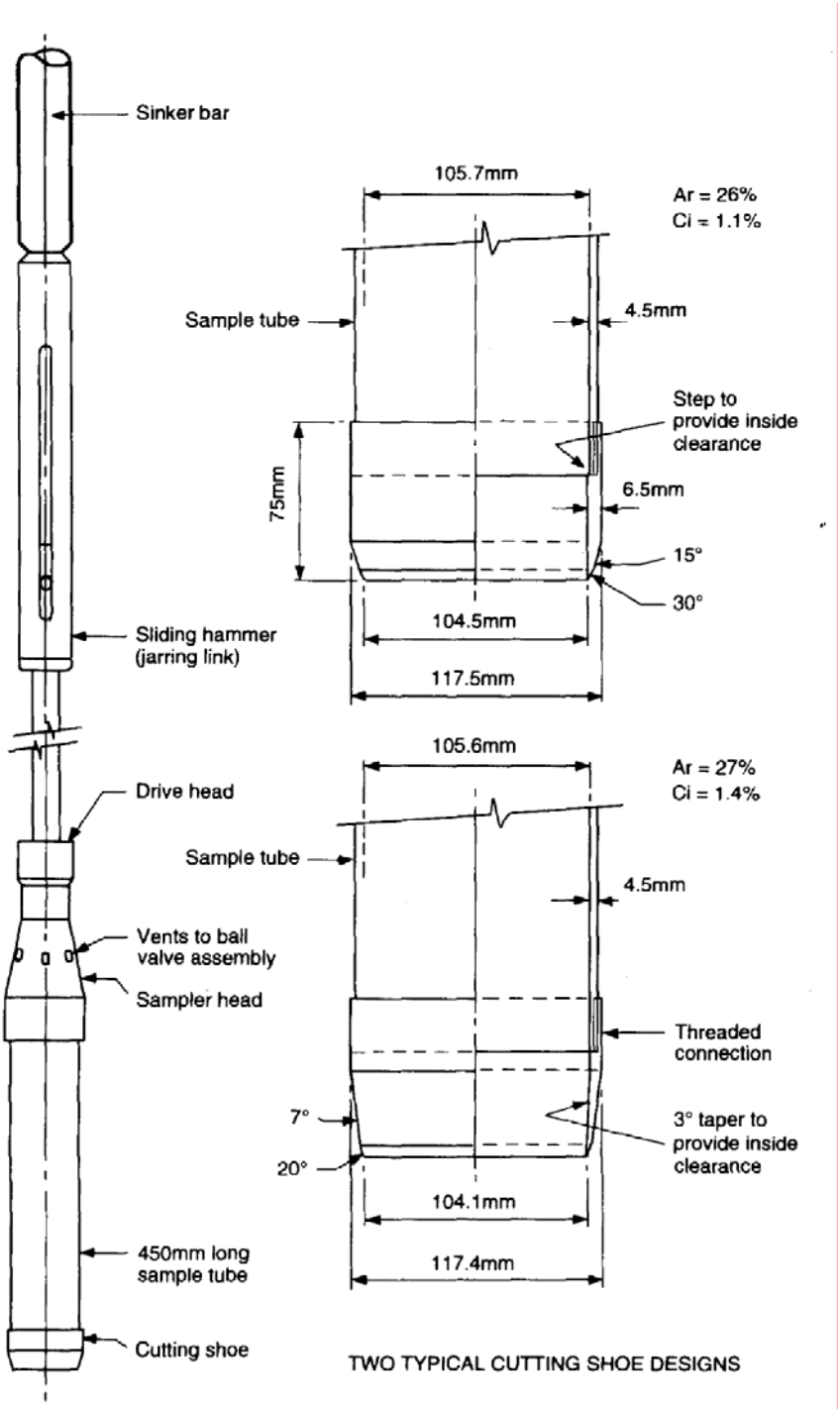


Fig. 6.11 U100 sampler assembly and details of cutting shoes.

If samples are to be hammered into the soil, then it is essential that the sampler should be rigidly

connected to rods extending to ground level. If the borehole is deep and large compared with the rod size, spacers may be required to reduce rod buckling as the hammer energy travels to the base of the hole.

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

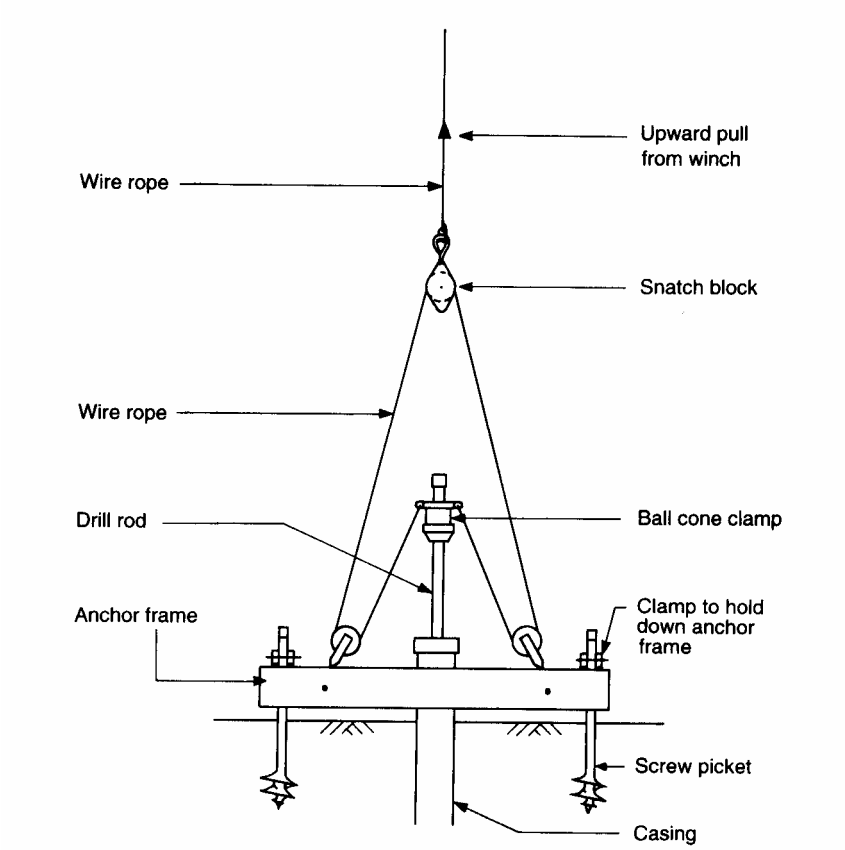


Fig. 6.12 Continuous push driving by means of winch and block and tackle.

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

Disturbance after sampling

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

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

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

Moisture loss

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

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

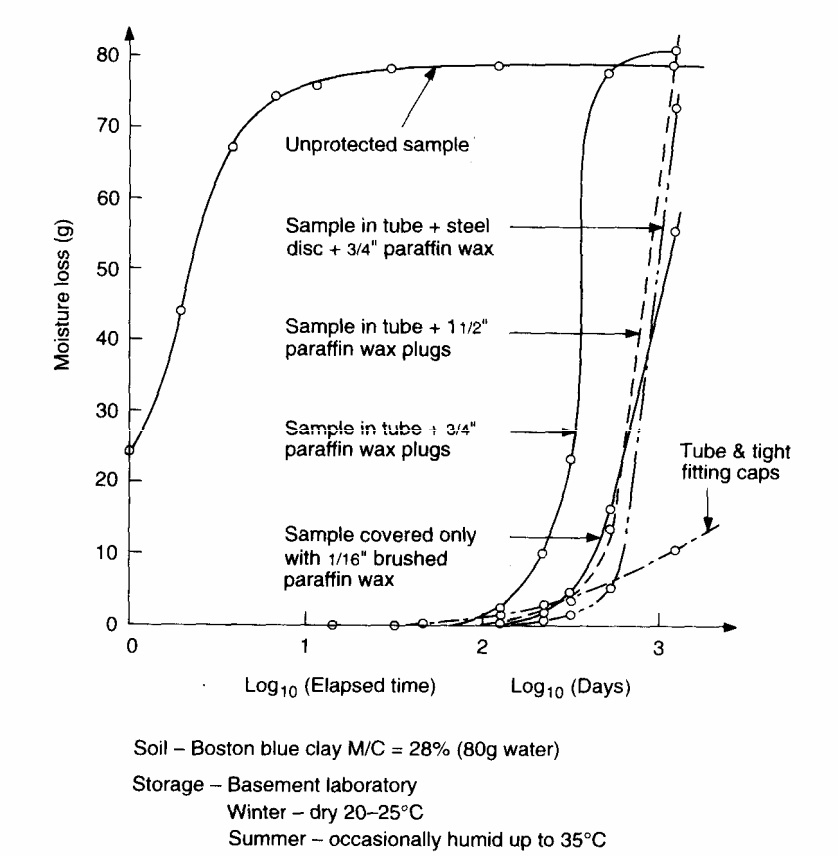


Fig. 6.13 Moisture loss with various sample sealing methods (data from Hvorslev 1949).

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

Paraffin wax shrinks upon cooling, and small cracks will lead to rapid moisture loss. All paraffin wax should be applied as close to melting point as possible to reduce shrinkage. At this temperature it

should be possible to dip a finger in the wax without being burnt. (Note that the melting point of paraffin wax is about 50°C, and therefore in very hot climates it will not be a suitable sealant.)

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

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

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

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

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

Migration of moisture within samples

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

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

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

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

Freezing

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

Vibration, shock, and mechanical disturbance

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

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

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

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

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

methods, the hydraulic system is the most convenient, provided that it can provide a stroke of sufficient length.

Reactions between soil and tube during storage

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

Disturbance in the soil-testing laboratory

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

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

Effects of sample disturbance

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

Failure to recover

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

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

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

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

Strength

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

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

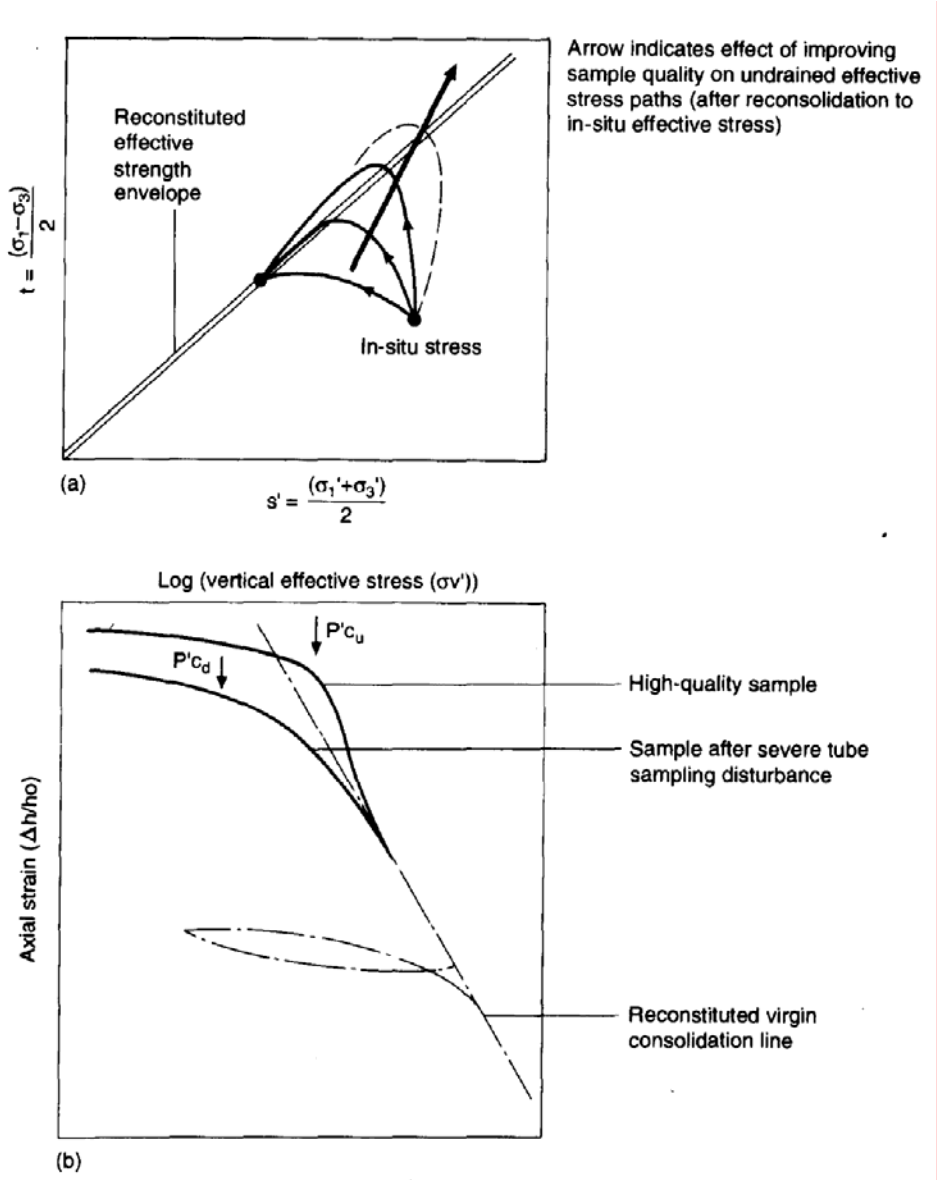


Fig. 6.14 Effects of tube sampling disturbance of lightly overconsolidated natural ('structured') clay on: (a) stress path and strength during undrained triaxial compression (b) one-dimensional compressibility during oedometer testing.

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

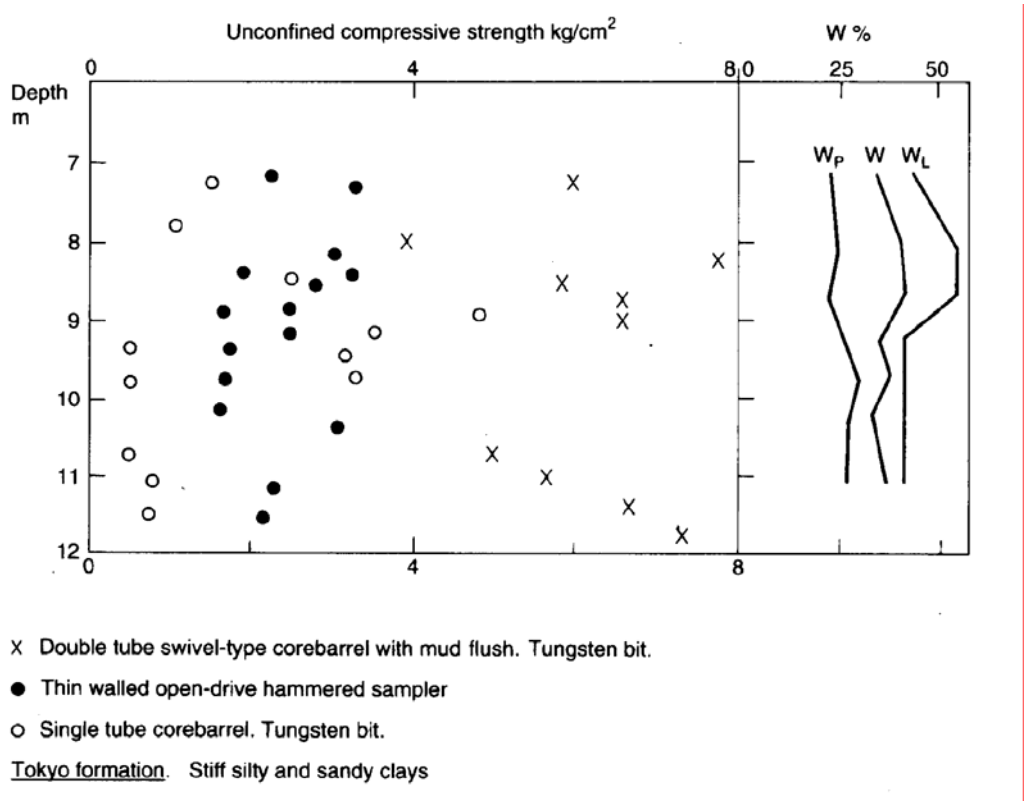


Fig. 6.15 Variation in unconfined compressive strength of Tokyo clay caused by different sampling methods (Seko and Tobe 1977).

Compressibility and stiffness

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

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

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

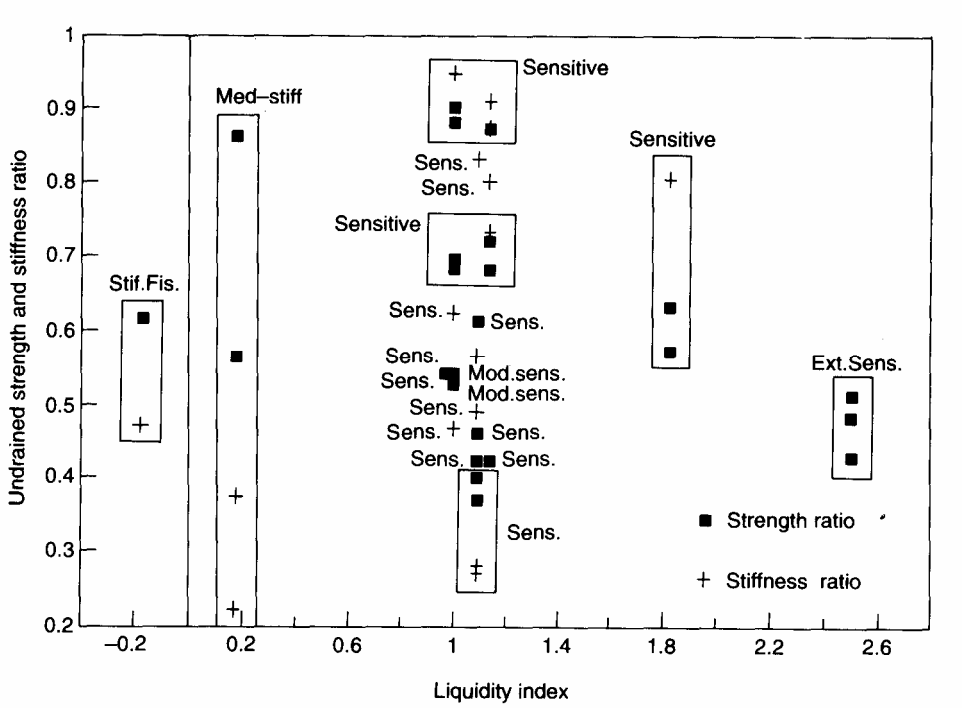


Fig. 6.16 Influence of tube sampling disturbance on undrained strength and stiffness (from a survey by Hopper 1992).

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

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

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

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

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

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

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

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

CLASSIFICATION OF SOIL SAMPLES

Hvorslev 's classification

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

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

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

Rowe's classification

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

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

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

*Items changed from German classification.

BRITISH PRACTICE, AND THE BS 5930 CLASSIFICATION

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

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

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

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

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

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

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

Chapter 7

Undisturbed sampling techniques

INTRODUCTION

Of the very large number of sampling techniques devised worldwide since the turn of the century, few are now in current use, and even fewer are in current use in the UK. Here, the most widely used tools are the 100 mm dia. thick-walled open-drive sampler, the 'Standard Penetration Test' 35 mm thick-walled open-drive split barrel sampler, 54mm or 102mm thin-walled fixed-piston samplers, and double-tube swivel type core barrels. The types of sampler adopted in each part of the world depend on the state of development of the area, its sampling tradition, economics, and its principal soil types. In the heavily developed South East and Midlands of England, soil types are typically stiff or very stiff clays and weak rocks. In the valleys, alluvium often consists of coarse gravels. Sampling is therefore based on the use of rugged tools in a large diameter borehole.

When carrying out site investigation abroad, the available drilling equipment is often very different from that used at home, and the familiar sampling tools may be either unobtainable or inappropriate. When drilling at home the solution of new problems may require a reappraisal of the value of commonly used techniques. These factors require an engineer to be aware of as many types of sampler as possible and this chapter therefore sets out to review the main types of equipment now available. In Chapter 6 (Sampling and sample disturbance) the way in which a number of common types of sampler are constructed, and the manner in which they work was described. Samples are obtained in a number of ways:

1. by using a number of techniques in shallow pits, shafts and exposures; and
2. in boreholes, using either drive or rotary techniques.

Drive samplers are pushed into the soil without rotation, displacing the soil as they penetrate. They generally have a sharp cutting edge at their base. In contrast, rotary samplers (often termed 'corebarrels') have a relatively thick and blunt cutting surface, which has hard inclusions of tungsten or diamond set into it. The sampler is rotated and pushed (relatively) gently downwards, cutting and grinding the soil away beneath it. A general classification of samplers is shown in Fig. 7.1.

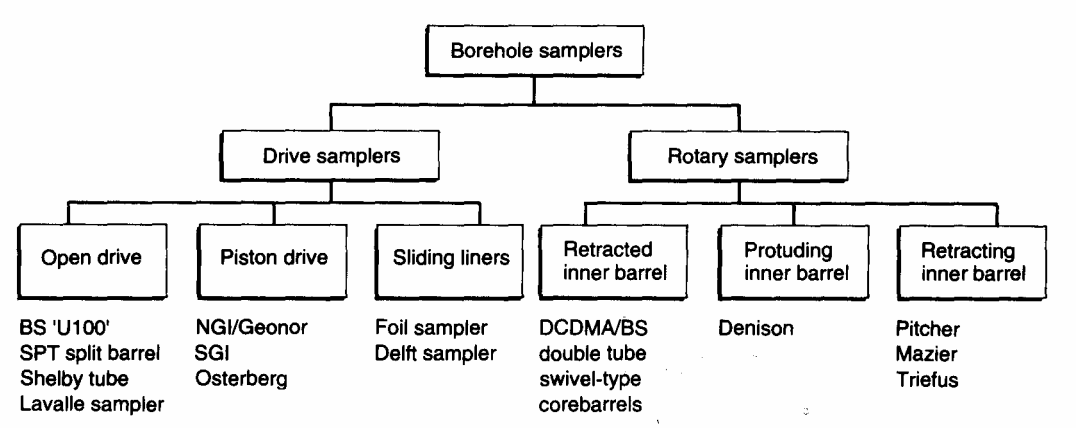


Fig. 7.1 General classification of borehole sampling devices.

It is generally believed that undisturbed sampling is not possible in granular soils. Nonetheless, special techniques for sand sampling have been developed over the years, and these are described in a later section of this chapter. Finally, we consider the selection of appropriate samplers for different purposes, and to suit different ground conditions.

SAMPLES FROM PITS AND EXPOSURES

Trial pits, trenches and shallow excavations are often used in site investigations, particularly during investigations for low- and medium-rise construction, because they provide an economical means of acquiring a very detailed record of the complex soil conditions which often exist near to the ground surface. It is worth remembering, however, that trial pits and other exposures can also be used for *in situ* testing and to obtain high-quality samples.

The types of samples taken will vary according to the needs of the investigation. Disturbed samples of granular soil are likely to be more representative than those that can be taken from boreholes. Disturbed samples are often taken for moisture content or plasticity determination in the laboratory, and in association with determinations of *in situ* density. *In situ* density testing is described in Chapter 9.

Undisturbed samples can be obtained either by drive sampling (see below) or block sampling (as described in Chapter 6). In either case it is important to recognize the disturbance created by excavating the trial excavation, and ensure that disturbed material is carefully removed before or after sampling. To this end, the faces and bottom of the pit should be hand trimmed in the areas to be sampled (or described), particularly when the pit has been machine excavated. In exposures, an attempt should be made to remove the weathered surface of the soil.

In the UK 38mm open-drive tubes are often hammered into the sides and base of trial pits. These tubes normally have no check valve, a high area ratio, and no inside clearance. It is often necessary to dig the sample tube out of the soil in order to avoid losses. When U100 tubes (see under ‘Drive samplers’) are used, they require considerable force and are commonly pushed into the soil with a ‘back-actor’ bucket. Care must be taken to prevent rocking of sample tubes during driving since this causes serious disturbance to the soil at the shoe level. The use of a frame to align the sampler during driving is advisable.

Better quality samples of firm to stiff clay soils can be obtained by trimming the soil in advance of a large diameter (100—200 mm) sampler. This eliminates the disturbance caused by soil displacement ahead of the cutting shoe, but may allow slight lateral expansion of the soil. When the material to be sampled is either hard, stoney or coarse and granular, it is essential that advanced trimming of large diameter samples is used.

When the soil is sufficiently stiff or cemented to stand up under its own weight, a block sample may be taken. The normal technique is to cut a column of soil about 300mm cube, so that it will fit inside a box with a clearance of 10—20 mm on all sides. A box with a detachable lid and bottom is used for storage. With the lid and bottom removed, the sides of the box are slid over the prepared soil block, which is as yet attached to the bottom of the pit. After filling the space between the sides of the box and block with paraffin wax, and similarly sealing the top of the block, the lid is placed on the box. The block is then cut from the soil using a spade, and the base of the sample trimmed and sealed. Block samples allow complete stress relief, and may therefore lead to expansion of the soil, but in very stiff clays this technique is widely regarded as providing the best available samples (see Fig. 6.5).

The Sherbrooke sampler

Block samples can only be taken from depth in heavily overconsolidated soils, such as the London clay. In normally and lightly overconsolidated clays, excavation of a pit or shaft to more than a few metres depth is often impossible because base heave will occur. Lefebvre and Poulin (1979) calculate that, for example, in a clay with an undrained shear strength the depth of a trench or pit will be limited to about 4 m, if a factor of safety of two is to be maintained.

To overcome this problem, Levebvre and Poulin (1979) designed the apparatus shown in Fig. 7.2, which is essentially a down-borehole block sampler. The equipment needs a borehole of about 400 mm diameter, which is best cleaned using a flat- bottomed auger, in order to reduce disturbance and minimize the amount of disturbed material left in the base of the hole before sampling. The hole is kept full of bentonite mud. The sampler is lowered to the base of the hole, and rotated, either by hand or using a small electric motor, at about 5r.p.m. A cylinder of soil, about 250 mm in diameter, is carved out by three circumferential blades, spaced at 120°. They make a slot about 50mm wide, and are fed by bentonite or water to help clear the cuttings. The time taken to obtain a sample obviously depends upon ground conditions, but may be about 30—40min. After carving out a cylinder about 350 mm high, the operator pulls a pin, and the blades (which are spring-mounted) gradually rotate under the base of the sample, as rotation is continued. Closure of the blades separates the sample from the underlying soil, and the sample is then lifted to the surface with a block and tackle. Lifting takes place very slowly for the first 0.5 m, in order to avoid suction at the base of the sample. The sample is coated with layers of paraffin wax, and may be placed in a container packed with damp sawdust or other suitable material. The complete process takes about 3 h, including preparation for shipment.

Tests by Lefebvre and Poulin have shown that this sampler is capable of obtaining soil of comparable quality to that produced by block sampling in the sensitive clays of eastern Canada. The sampler was used in the Bothkennar clay in Scotland, where it provided the highest quality of samples obtainable (Clayton *et al.* 1992). It was found, however, that the apparatus is quite time-consuming and difficult to use. Lefebvre and Poulin (1979) note that it is not intended that this technique should replace tube sampling for routine investigations. But where the highest quality samples are required for testing of soft or sensitive clays, at the time of writing this apparatus provides the best method of obtaining undisturbed samples from depth.

DRIVE SAMPLERS

Drive samplers are samplers which are either pushed or driven into the soil without rotation. The volume of soil corresponding to the thickness of the sampler wall is displaced into the surrounding soil, which is either compacted or compressed.

Drive samplers can be divided into two broad groups: open-drive samplers and piston drive samplers. Open-drive samplers consist of a tube which is open at its lower end, while piston drive samplers have a movable piston located within the sampler tube. Piston samplers can be pushed through a soft soil to the desired sampling level, but open-drive samplers will admit soil as soon as they are brought into contact with, for example, the bottom of a borehole.

Open-drive samplers

Open-drive samplers suffer from several disadvantages, as Hvorslev (1949) pointed out. Poor cleaning of the borehole before sampling, or collapse of sides of the borehole after cleaning may mean that much of the recovered soil is not only highly disturbed, but also non-representative. The use of a large area ratio can induce soil displaced by the sampler drive and causing large-scale remoulding of the

sample. The problems of pressure above the sample during the drive, and of sample retention during withdrawal have been noted in the previous chapter.

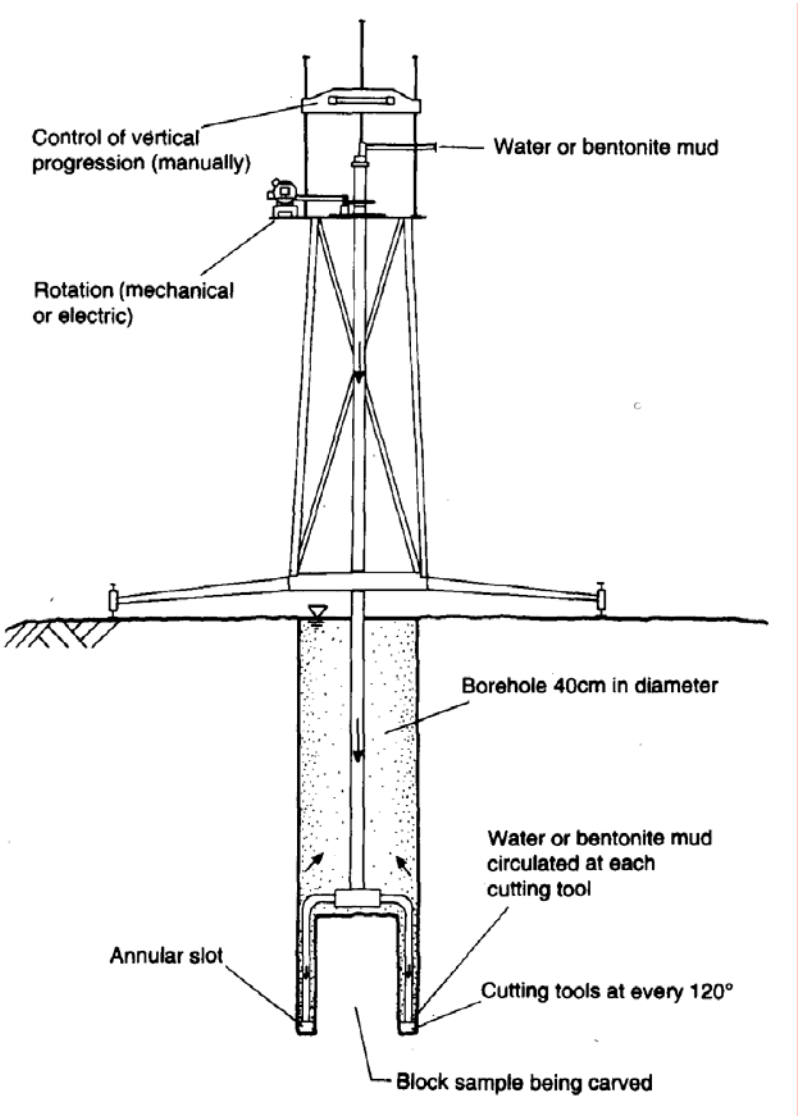


Fig. 7.2 Schematic diagram of the Sherbrooke down-hole block sampler (Lefebvre and Poulin 1979).

The advantages of open-drive sampling are principally those of cheapness, ruggedness and simplicity of operation. Open-drive samplers can be arbitrarily divided into two groups. Thin-wall open-drive samplers have been defined as those with a wall thickness of sampling tube of less than 2.5% of the diameter, corresponding approximately to an area ratio of 10% (Hvorslev 1949). This classification is not a good guide to the amount of sampling disturbance because of the influence of cutting shoe taper and *in situ* stress level in the soil. In the following discussion thin-wall sampling devices are taken to be those with an area ratio of less than 20%, and a suitable cutting shoe taper, while thick-wall samplers are taken to have an area ratio greater than 20%.

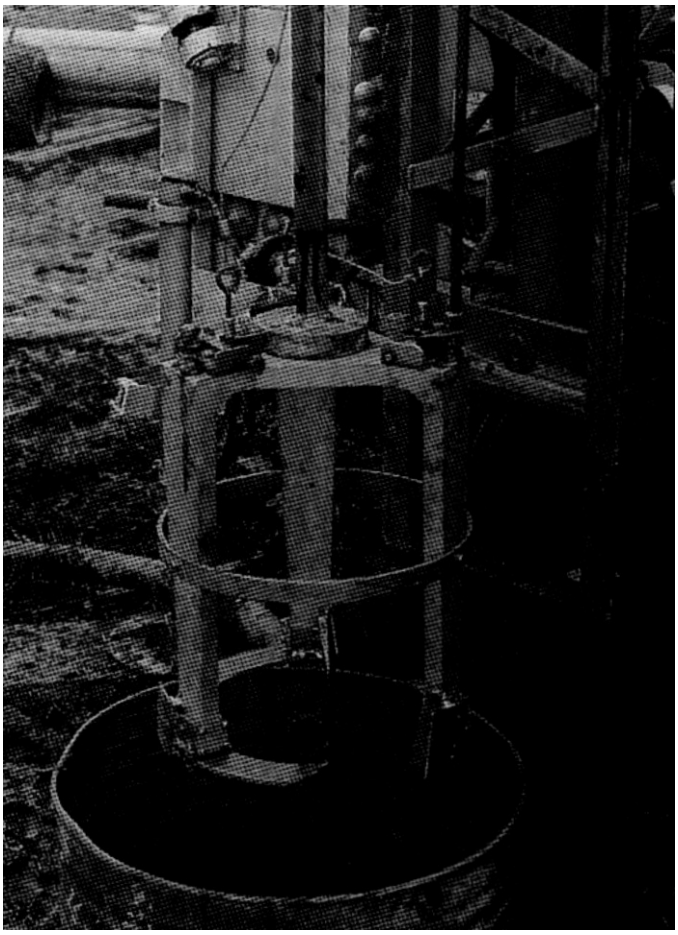


Fig. 7.3 The Sherbrooke sampler in operation (photograph courtesy Dr D.W. Hight).

Thick-walled open-drive samplers

Thick-walled open-drive samplers are widely used throughout the world. In their most common forms they consist of a solid or split sampler barrel, threaded at both ends to take a cutting shoe (typically with inside clearance) and a sampler head provided with either a check valve or vents.

BS general purpose sampler

The British Standard General Purpose 100 mm Sampler (BS ‘5930:1981), commonly termed the U100 sampler, evolved during the 1930s and 1940s (Le Grand *et al.* 1934; Cooling and Smith 1936; Cooling 1942; Longsdon 1945; Rodin 1949). The sampler is rugged, cheap and will provide a core sample in most British clays, which are typically heavily overconsolidated. Its size and form were adopted because of the common borehole size at the time of its development, and because of its ability to sample (however ‘inadequately) in stoney very stiff glacial clays.

The British Standard U100 sampler will fit inside a 150mm dia. borehole. Harding (1949) noted that:

> some believe that the smaller the [casing] tube the cheaper will be the hole. This is a fallacy. In British gravel-laden deposits, nothing less than 6-inches [152 mm] diameter is worthwhile. This permits the average type of stone to be brought up by shell without pounding with a chisel and also allows of 4-inch diameter sampling.

Light percussion drillers will often use 200mm tools in preference to the 150mm size because of their greater weight, and thus their improved ability to make fast progress.

Figure 6.11 shows a typical British 100mm dia. open-drive sampler. It has a 104 mm inside dia. at the base of the cutting shoe, a 27% area ratio, and an inside clearance of 1.4% provided by tapering the inside diameter of the cutting shoe at an angle of 30 to meet the 106mm internal diameter of the sample tube, or by providing a uniform internal diameter to the cutting shoe and thus stepping out, abruptly at the junction of the shoe and sampler tube. The outside cutting edge taper may be 20° up to a thickness of 2.3 mm, and 7° thereafter, or alternatively may initially be 30°, and then 15° up to a 6.5mm thickness. Designs vary according to the manufacturer, but shoes and tubes are always interchangeable. The normal sample tube length is 457 mm, but two tubes are often coupled together in order to allow debris at the bottom of the borehole to pass into the upper tube during driving. Thus the normal length to diameter ratio is about 4.5, with a possible maximum of 9.

The U100 sampler head incorporates vents and a ball valve assembly to allow air or water to leave the top of the tube as soil enters at its base. The ball valve is also intended to improve the sample retention by preventing air or water re-entering the top of the tube if the sample starts to slide out. Because of the conditions under which the vents and ball valve are expected to work, it is important to ensure that the sampler head is cleaned before each sampling operation. Even though the build-up of pressure above the sample can be reduced to an acceptable level, it is doubtful if the ball-valve assembly can be effective in reducing sample losses.

The so-called British Standard sampler is not as completely standardized as, for example the Swedish piston sampler. In CP 2001:1957 the maximum area ratio was specified as 25%, with an inside clearance of between 1% and 3%, and a drawing of a suitable design was given. BS 5930:1981 allows an area ratio of 30%, but other recommendations remain the same. The vent area should be not less than 600mm2 cutting shoe taper is not specified. The sampler may or may not use liners, to allow the specimen to be transported and stored in a lightweight cylinder which does not have to be able to resist driving forces.

In the 1982 edition of this book we wrote that ‘most users of this type of sampler do not fit liners’. It is unfortunate that in the past decade it appears that many UK companies, apparently driven by the need to reduce costs, have taken to using plastic liners inside steel outer tubes, in order to reduce the number of metal tubes they require to hold in stock. We have repeatedly observed the severe distortions induced when plastic liners are used, in comparison with the relatively low level of distortions when they are not. Figure 7.4 shows an example of this, in a laminated clay. The inclusion of a plastic liner, typically about 3mm thck (means that the cutting shoe thickness must be increased. Taking the example of the lower cutting shoe in Fig. 6.11, it can be calculated that the area ratio will increase from 27% to 41%. Examination of cutting shoes used with this type of sampler suggests that a value of area ratio of 45—50% is usual, equivalent to a B/t ratio of about 11. As we have shown, the minimum axial strain (at the centreline) will be of the order of 4—5%, and peripheral strains and shear distortions can be expected to have a very significant effect on soil properties (Georgiannou and Right, 1994).

The U100 sampler is rugged, and easy to use. The sample tubes are screw-threaded to the head and cutting shoe before sampling. The sampler head is screwed to a sliding hammer (also termed a ‘jarring link’) and lowered down to the base of the hole on square rods. The sampler is driven into the soil by repeatedly lifting the rods through about 500mm and allowing them to fall. The number of blows and the distance moved by the sampler head during the drive are recorded by the driller. The sampler may be pulled immediately from the soil, or in stiff cohesive soils it may be left in the soil for a few minutes before it is brought to the top of the hole. After sampling, the tube and soil are carefully separated from the cutting shoe and sampler head. A small quantity of soil is removed from either end of the tube if necessary, and the ends of the sample are waxed, packed and then sealed with either plastic, or screw-threaded metal caps. If damaged or blunt, the cutting shoe is replaced before the sampler is next used.



Fig. 7.4 Heavy shear distortion in a clay sample taken using a driven BS U100 sampler with a plastic liner (courtesy Surrey Geotechnical Consultants Ltd).

In many countries the use of light-weight drilling rigs, percussion equipment and large borehole diameters is not advantageous. American manufacturers of samplers provide a very wide range of sampler diameters, with a variety of lengths. The Acker Solid Tube Sampler is available in a great many sizes. Its inside diameter may be 1in. (38mm), 2in. (52mm), 2km. (64mm), 3m. (76mm), 4in. (102mm) or 5in. (127mm). Its length may be either l8in. (457mm) or 60in. (1524mm). The difference between inside and outside diameters is quoted as ½ in. (12.7 mm) and the area ratios and length to diameter ratios of the various samplers are therefore as listed in Table 7.1.

Table 7.1 Open-drive sampler characteristics for a tube thickness of 0.5 in.

Internal tube diameter (in.)	Area ratio (%)	Length/diameter ratio	
		L=l8in.	L=60in.
1 ½	78	12	40
2	56	9	30
2 ½	44	7.2	24
3	36	6	20
4	27	4.5	15
5	21	3.6	12

Clearly, most of these tubes have excessive area and length/diameter ratios, and will not provide undisturbed soil for laboratory testing to give soil properties relevant to *in situ* conditions. However, in soil conditions where small diameter boreholes can be very much more economical than holes of 150mm dia. and larger, they are particularly useful. Terzaghi (1939) indicated the importance of what he described as the ‘variation survey’, a completely sampled profile of soil along several vertical lines of a site:

> In 1925, under the illusion that soil strata really are fairly homogeneous. I had the habit of requesting

> one ‘dry [i.e., tube] sample’ for every 5 or 10 feet of test borings through homogeneous strata. Since that time the painstaking investigations of Mr A. Casagrande have destroyed my cherished illusion. No longer is there any doubt that homogeneous beds of clay are very rare Due to the universal absence of homogeneity the essential prerequisite for selecting representative samples consists in securing complete data on the variation of at least one property of the soil along several vertical lines. The samples for the more elaborate soil tests are then selected in such a manner that the properties of the most frequent soil types are determined. The weighted average of the test results is obtained by statistical methods.

The variation survey was originally accomplished by obtaining continuous 2 in. (50 mm) open-drive samples with a thin-wall sampler 30in. (450mm) long. The samples were cut into 6 in. sections before being tested for moisture content or compressive strength.

In the UK, despite the advice from Terzaghi and from others (for example, Rowe (1968, 1972)), few site investigations involve taking continuous samples of any sort.

Presumably this is because of a desire to reduce the cost of site investigation. Such an attitude must be considered very short-sighted.

Both of the sampler types discussed above have ‘solid’ tubes; their sampler barrels are continuous around the circumference.

Thick-walled split barrel samplers

A further common sampler is the thick-walled open-drive split barrel sampler. Here the sampler barrel is split longitudinally into two halves. During driving these are held together by the shoe and head which are screwed on to each end. The split barrel allows easy examination and extraction of the sample, but makes the sampler considerably weaker. To compensate for this, such samplers are usually short, and have a high area ratio.

One of the most common thick-walled open-drive split barrel samplers is used during the Standard Penetration Test (see Chapter 9). During this test the sampler is driven into the soil by repeated blows of a 65 kg hammer falling freely through 760 mm, and the number of blows required to drive the sampler a distance of 300mm is recorded as the SPT N value. The N value is assumed to be dependent on relative density in granular soils, and undrained shear strength in cohesive soils.

Figure 7.5 shows the apparatus used in the UK for the SPT test. The dimensions of the sampler are defined in BS 1377:1975. Any sample obtained from this sampler will be highly disturbed, because the SPT split barrel sampler has an area ratio of about 100% and a length to diameter ratio of 13. No inside clearance is used. Samples of fine soils obtained from the apparatus should be considered as remoulded.

Samples f coarse granular soils must be considered unrepresentative, because the coarser particles will not be able to enter the barrel during driving. For this reason the Report of the Subcommittee on the Penetration Test for use in Europe allows the sampler cutting shoe to be replaced by a solid steel cone with a J0 apex angle, when drilling in gravelly soils. This avoids cutting shoe damage, and prevents high penetration resistances due to the lodgement of large particles in the end of the shoe.

Because of its design, the SPT thick-wall open-drive split barrel sampler will give low recoveries in most soils. It is therefore unsuitable for obtaining continuous representative samples for a variation or reconnaissance survey. In the UK the resulting sample is normally broken into lengths of about 75 mm and placed in a small disturbed sample container such as a glass jar.

Samplers using liners inside the sample tube were termed ‘composite samplers’ by Hvorslev (1949). Liners allow considerable savings to be made because the structural outer sampler barrel, which transmits the driving force to the cutting shoe, can be used repeatedly. Only the liner is removed,

complete with sample, and sealed and transported to the laboratory. Where cohesionless or very soft soils are to be sampled it is, of course, necessary that they are not removed from the sampler tube before they are to be examined. We have already noted (above) the increase in the use of plastic liners in the BS U100 sampler in the UK during the last decade. In other countries better-designed liners are typically made of metal.

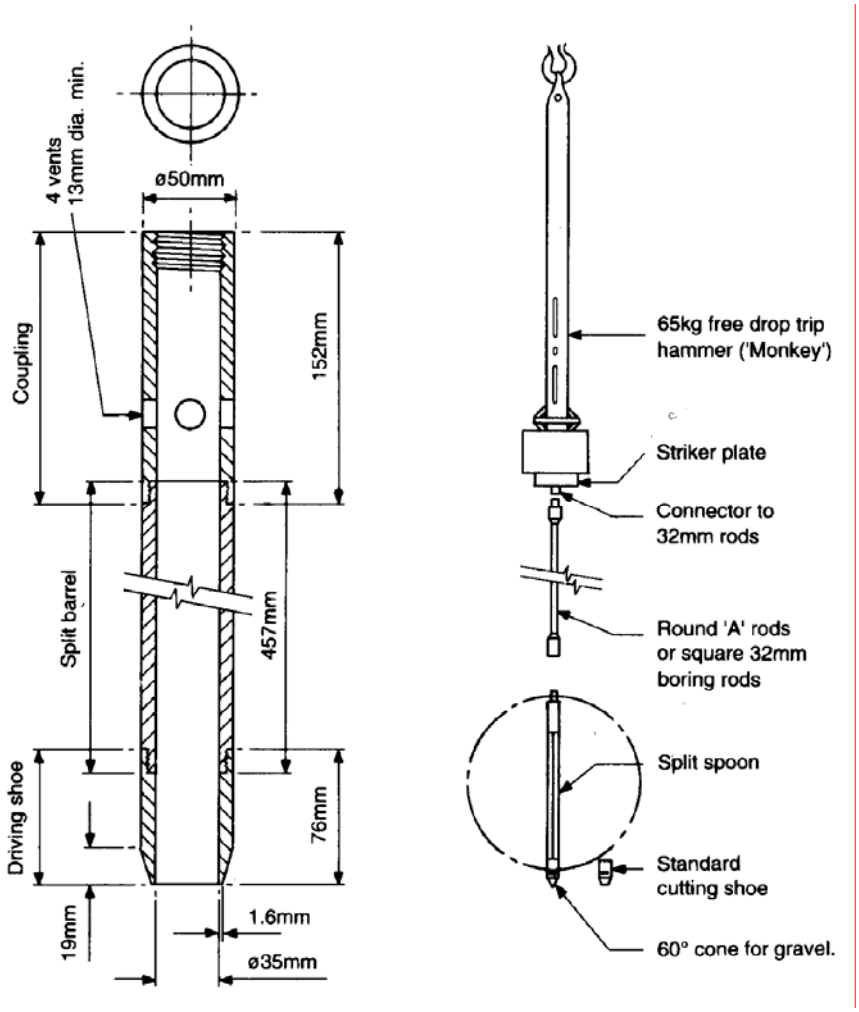


Fig. 7.5 Standard Penetration Test' equipment.

The Acker-split tube sampler is available with either solid or sectional liners. Sectional liners can be very useful in reducing the need for a laboratory extruder to remove soil from the tube, and they also allow the soil to be examined in the field, if necessary. If an extruder is not required for sample extraction, the liners can be used successfully in a wider range of soils. The inside diameter of the cutting shoe is 2 $^{15/32}$ in. (62.7mm), while its outside diameter is 3 ¼ in. (82.6mm), giving an area ratio of 74%. The inside diameter of the liner is 2 ½ in. (63.5mm), giving an inside clearance of 1.6%.

Thin-walled open-drive samplers

The thin-walled open-drive sampler, or 'Shelby Tubing' sampler was introduced in the USA in the late 1930s (Terzaghi 1939; Hvorslev 1940, 1949). 'Shelby Tubing' is a trade name for hard-drawn, seamless steel tube manufactured by the National Tube Company of the USA. Early devices took two forms. The US Engineer Office, Boston District, sampler (Fig. 7.6) attached the thin-wall sampler tube to the head by spot welding it to a short length of heavy tube, which in turn threaded into the head. Another method of fixing the tube to the head, suggested by H. A. Mohr. uses tubing which is a close fit over the lower section of the sampler head, and which is fixed to the head by two Allen set screws which, when engaged, lie flush with the outer surface of the sampler tube. This design has been

incorporated into a large number of samplers, and is now in use worldwide (see, for example ASTM D 1587—74).

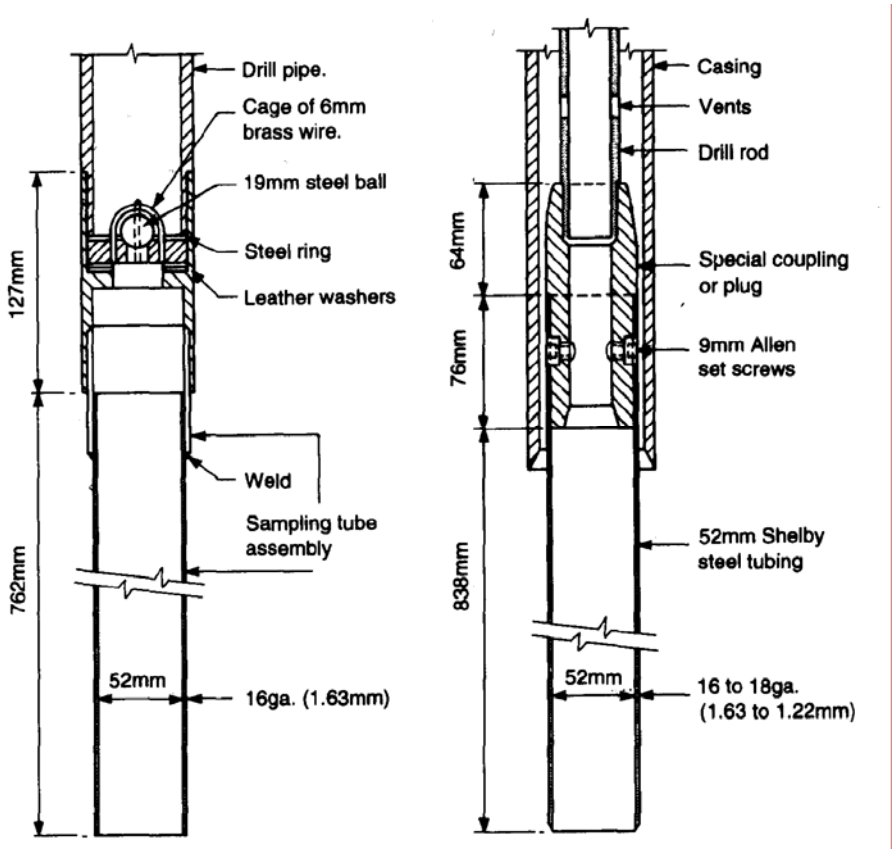


Fig. 7.6 Two early thin-walled open-drive samplers (from Hvorslev 1940).

Early thin-walled samplers, such as those in Fig. 7.6 had relatively small area ratios (approximately 10—14%), but had length to diameter ratios of 15—20 and did not use inside clearance. The cutting edge was either cut square, or bevelled. Most modern thin-walled tubes are drawn in, in order to provide a suitable inside clearance (Fig. 7.7) and are sharpened.

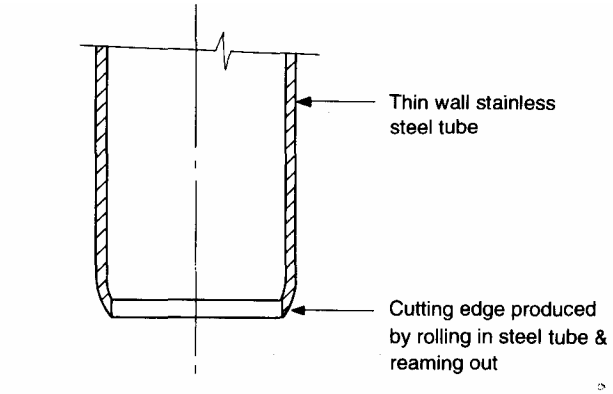


Fig. 7.7 Typical detail of thin-walled open-drive sampler, showing drawn in, sharpened cutting edge.

Thin-walled open-drive sample tubes are readily damaged, either by buckling or blunting or tearing the cutting edge, when they are driven into very stiff, hard, or stoney soils. Pushing, rather than hammering, tends to reduce the chances of damaging the tube. When the cutting edge is damaged, the tube must be sent to the metal workshop for reforming.

In the UK, thin-walled open-drive samplers have, during the past decade, become used for the high

quality tube sampling of very stiff and hard clays, such as the London clay. Such a sampler, which is hydraulically jacked (from a frame at ground surface) into the bottom of the hole has been described by Harrison (1991). They are in wide use elsewhere, and can, with a certain amount of care, be used to obtain undisturbed samples from very soft soils with undrained shear strengths of the order of 5 kN/rn2. In very soft sensitive soils sampling will normally need to be carried out with a piston sampler.

Thin-walled open-drive sample tubes are typically 24—3Oin. (i.e. 610—762mm) long, and give a maximum sample length 2—3 in. (52—70 mm) less than this. They may be expected to have an inside clearance of up to 1— 1%. Available in a wide variety of diameters, they typically have area ratios similar to those in Table 7.2.

Table 7.2 Thin-walled open-drive sample tubes

Internal tube diameter (mm)	Area ratio (%)	Length/diameter ratio
48	15	11.5
60	13	9.1
73	12	7.5
86	10	6.4
121	8	4.9

It can be seen that for this type of sampler, the 121 mm internal diameter tube provides an excellent combination of low area ratio and low length to diameter ratio, which would give acceptable results with a minimum of inside clearance. At the other end of the scale, the small diameter sampler is very similar to that used by Terzaghi for his variation surveys, with the improvements of a sharpened cutting edge and inside clearance.

Laval sampler

Probably the most effective tube sampler available for sampling soft and sensitive clays is the Laval sampler (La Rochelle *et al.* 1981). The use of such an expensive, time-consuming and delicate sampling process for routine sampling probably cannot be justified, but it has been shown (not only by the originators, but also as a result of trials at Bothkennar in Scotland, reported by Clayton *et al.* (1992)) that this sampler recovers soft and sensitive soil almost of the quality that can be achieved using block sampling techniques.

The Laval sampler is shown in Figs 7.8 and 7.9. The device consists of a thin-walled sampling tube mounted on a sampler head, and housed within an external corebarrel. The sampling tube contains a screw-type head valve which ensures that an effective vacuum can be achieved above the sample during withdrawal from the ground. The external corebarrel is used to remove soil around the sampling tube, after tube penetration, to ensure that no vacuum exists at the bottom of the cutting edge during sample withdrawal. No inside clearance is required because, in the soil types in which it is intended to be used, the shearing action between the tube and the soil leads to positive excess pore pressures, a reduction locally in effective stress, and a consequent lubricating effect. The inclusion of inside clearance was thought by La Rochelle *et al.* to introduce unnecessary ‘squeezing in’ of additional soil, and consequent disturbance. The sampling tube is precision machined from ZW-1035 carbon steel tubing with an i.d. of 200mm and an o.d. of 218 mm, to give a uniform circular internal cross-section along its length, and an internal diameter of 208 ± 0.03 mm. With a wall thickness of 5mm, the area ratio is 10% and the B/t ratio is 42. The cutting edge angle is 5°.



Fig. 7.8 The Laval sampler (photograph courtesy Dr D.W. Hight).

The operation of the Laval sampler is shown in Fig. 7.10. A borehole is made to the required depth, either open-hole using a fishtail bit, or as a result of previous sampling. No casing need normally be used, since bentonite mud flush provides wall support. The sampling assembly is lowered to the bottom of the hole with the sampler hooked on to the collar inside the top of the corebarrel (Fig. 7.10). With the head valve open, the sampling tube is gently unhooked by lifting and turning the inner rod at the surface. The sampler is pushed slowly into the soil, stopping some 50mm before contact is made between the bottom of the sampler head and the upper surface of the soil (Fig. 7.l0b). The head valve is then closed, and the corebarrel is used in conjunction with mud flush to clear soil from around the outside of the sampler tube, and to a depth of approximately 20mm below the bottom of its cutting edge (Fig. 7.l0c). The sample tube is rotated through about 90° in order to shear the soil at its base, and is then pulled gently out of the soil and hooked back on to the internal collar (Fig. 7.9). The assembly is finally removed from the borehole. The sample is extruded immediately, and is cut into slices 130mm or 200mm high which are placed on waxed plywood board and sealed in several layers of Saran paper sandwiched between brushed paraffin wax/vaseline mixture. The cost of the steel sampling tubes is such that they cannot economically be used for sample storage.

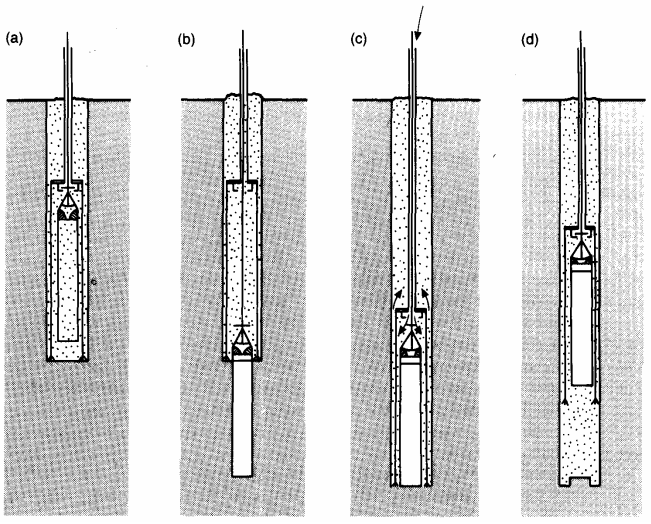


Fig. 7.10 General operation of the Laval sampler (La Rochelle *et al.* 1981).

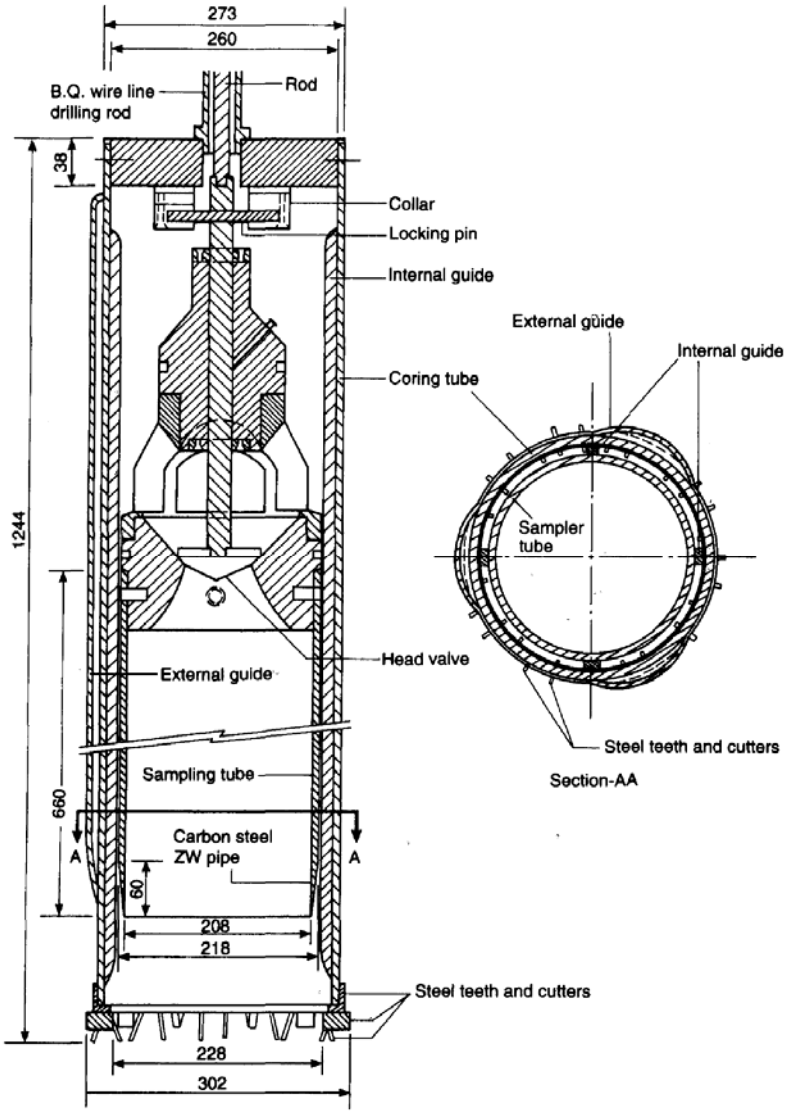


Fig. 7.9 The Laval 200mm diameter tube sampler (La Rochelle, Sarrailh, Tavenas, Roy and Leroueil 1981).

Piston drive samplers

Piston samplers were developed in both Europe and the USA in the period between 1900 and 1940. All piston samplers have a piston contained within the sample tube, which is moved upwards relative to the sample tube at some stage of the sampling process. Because there may be various reasons for including this piston in the design, however, the mechanisms associated with the piston movement are numerous.

Pistons have been included in sampler designs in order:

1. to prevent soil entering the sampler tube before the sampling position is reached. Many piston samplers have been specifically designed to be pushed, without a pre-drilled borehole, through the soil to the desired sampling depth (Bastin and Davis 1909; Olsson 1925, 1936; Petterson 1933; Porter 1936, 1937, 1939; Stokstad 1939), although it should be remembered that when this is done the upper part of the sample will normally be highly remoulded.

2. to reduce losses of samples, by providing an efficient airtight seal to the top of the soil in the tube during withdrawal. Any tendency of the sample to slide out of the tube is counteracted by pressure decrease above the sample, (for example, see Ehrenberg (1933)).
3. to reduce the entry of excess soil into the tube during the early stages of sampling, as a result of using a relatively high area ratio, and to prevent too little soil entering the sampler at the end of the drive, as a result of the build-up of internal friction; and
4. to increase the acceptable length to diameter ratio. Adhesion between the tube and the soil entering it will tend to reduce recovery once large length/diameter ratios are reached, but the movement of the top of the sample away from the underside of the piston will form a vacuum which will tend to increase the recovery.

Early American piston samplers, such as the Davis peat sampler (Bastin and Davis 1909), differed from early Swedish piston samplers (for example, Olsson (1925)) because the piston in the former was retracted to the top of the sample tube before it was driven, while in the latter case the piston remained fixed at the same level relative to ground surface throughout the drive (Hvorslev 1940). Hvorslev (1949) defined three main groups of piston sampler: (i) free piston samplers; (ii) retracted piston samplers; and (iii) fixed piston samplers.

Free piston samplers

Free piston samplers have an internal piston which may be clamped during withdrawal of the sampler, and during driving of the sampler to the required sampling depth. However, when the sample tube is being pushed into the soil during sampling the piston is free to move both with respect to the sample tube and to ground level. Figure 7.11 shows the Ehrenberg piston sampler and the Meijn piston sampler. In the former, the piston is free to move upwards at all times, but it cannot move downwards relative to the sample tube. It is not suitable for pushing through soft soil in order to reach the desired sampling level. In contrast, the Meijn sampler holds the piston at the bottom of the sample tube with two lugs on the inner rod which locate below a grooved collar in the sampler head. When the sampler reaches the correct level the sampler head and tube are rotated through 90° and the lugs clear the collar. After pushing the sample tube into the soil, loss of material is prevented during withdrawal by a cone clamp in the sampler head which prevents the piston rod sliding downwards through the head.

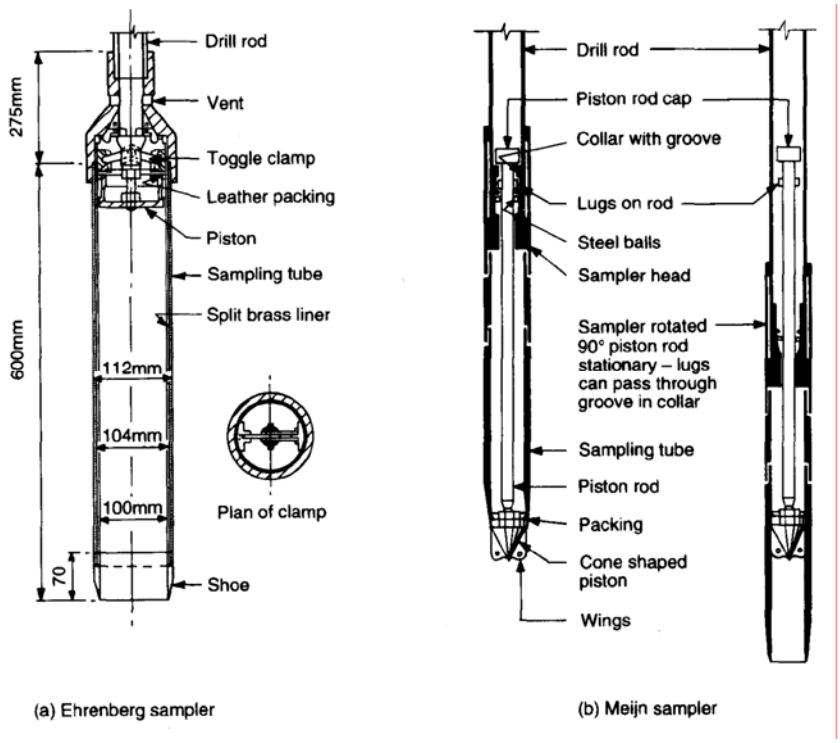


Fig. 7.11 Two types of free piston sampler (Ehrenberg 1933; Huizinga 1944).

Free piston samplers overcome most of the disadvantages of the open-drive type of sampler, but they remain easy to use. Their main advantages are that they can be designed so that they can be pushed through debris at the base of a borehole, and that sample losses are greatly decreased by the provision of an efficient seal at the top of the sample. Despite this they are not used in the UK, perhaps because of fears that friction between the piston packing and the inside of the sample tube may cause sample compression or significantly reduce recovery.

Retracted piston samplers

Retracted piston samplers use the piston primarily to prevent the entrance of unwanted soil during the process of pushing the sampler to the required sampling depth (Bastin and Davis 1909; Porter 1936, 1937, 1939; Stokstad 1939). Once this depth is reached the piston is retracted to the top of the tube, and the sampler is then driven into the soil. The retraction of the piston may cause soft soil to flow upwards into the tube, and during driving a large area ratio may lead to the entry of excess soil into the tube. This type of sampler is not in use in the UK. It retains several of the disadvantages of the open-drive sampler, and is more difficult to use.

Fixed piston samplers

Fixed piston samplers can be used with or without a borehole. The sampler is pushed to the level at which sampling is to start with the piston rod fixed relative to the sampler head and tube, and located at the base of the tube to prevent the entry of soil. At this point the piston is freed at the sampler head, but refixed at the ground surface to the drilling rig or to a suitable frame in order to prevent it moving vertically during sampling. The sample tube is then pushed ahead of the piston into the soil. After sample driving, the inner rods extending to the ground surface from above the sampler head can be removed, since the piston is prevented from moving downwards relative to the sample tube by a clamp located in the sampler head.

Fixed piston samplers have all of the advantages discussed above: they prevent the entry of debris before sampling, they reduce the entry of excess soil during sampling and they largely eliminate sample losses. Hvorslev (1949) commented that ‘the drive sampler with a stationary piston has more advantages and comes closer to fulfilling the requirements for an all-purpose sampler than any other type’. Its disadvantages lie principally with its cost and complexity in use.

The original fixed piston sampler developed by Swedish engineer, John Olsson (Olsson 1925, 1936) controlled the movement of the piston before and during sampling by extending the piston rod to the ground surface inside the outer rods, and clamping it either to the outer rods or a frame at ground level. Modifications and improvements were tried over a period of many years (Petterson 1933; Bretting 1936; Kjellman 1938; Fahlquist 1941; Hvorslev 1949; Osterberg 1952; Hong 1961). In 1961, the Swedish Committee on Piston Sampling produced their findings (*Swedish Geotechnical institute Report No. 19*) and Kallstenius gave precise details of the apparatus in use. Since that time, work has been carried out to assess the effectiveness of piston samplers in obtaining good quality undisturbed samples (Berre *et al.* 1969; Schjetne 1971; Holm and Holtz 1977) and modifications to the apparatus have continued (for example Osterberg (1973) and Tornaghi and Cestari (1977)). The Japanese Society for Soil Mechanics and Foundation Engineering have subsequently published a Draft Standard for stationary piston sampling (Mon 1977).

Piston samplers with fixed pistons are available with a variety of sampler barrels. These may be thin-walled (made of either seamless steel tubing or of aluminium tube) or of the composite type.

Figure 7.12 shows a thin-walled seamless steel tube type fixed piston sampler similar to those described by Hvorslev (1949). The sampling tube has a rolled and reamed cutting edge. The sampler may be pushed through soft soils to the desired sampling level and, during this process, the conical piston is held at the base of the sampler tube. This is achieved by attaching the piston rod to the upper

part of the head via a few turns of the left-hand thread of the piston rod screw clamp. When the sampling level is reached, the piston rods are turned clockwise at ground surface, tightening the rods above the sampler but disengaging them at the screw clamp. The piston rod is then fixed to the rig or a frame at ground level, and the hollow outer rods are pushed smoothly downwards to drive the sampling tube ahead of the piston into the soil. After sampling, the piston rods can be unclamped and the sampler pulled to the surface using the outer rods. The piston is held up by the ball cone clamp in the sampler head. Once the sampler is at ground surface, the tube is released from the head by screwing the Allen set screws inwards. The ball cone clamp must be released by turning a screw on the side of the head through 900 before the sample tube can be pulled from the head. The vacuum release screw must be slackened before the piston can be pulled out of the sample tube. The various screws must be reset before the next sample is taken.

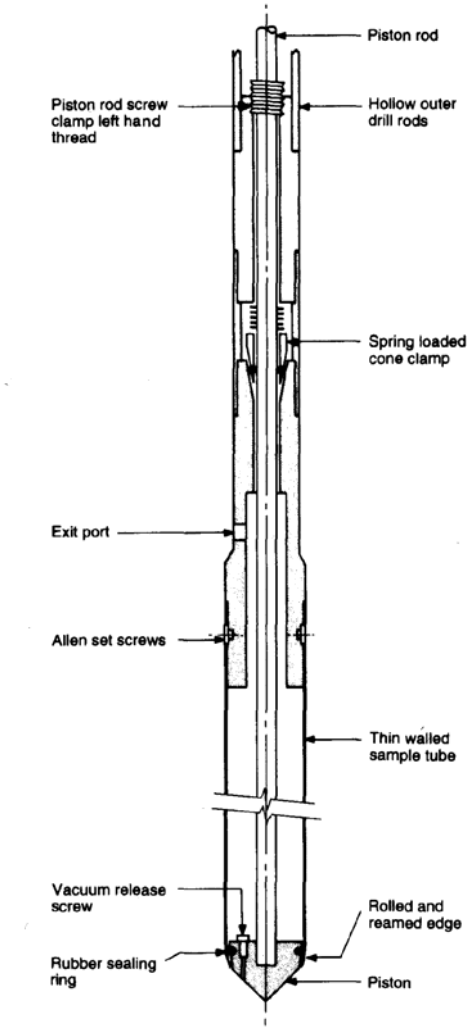


Fig. 7.12 Thin-walled seamless steel tube fixed piston sampler.

A device such as that described above has been widely and successfully used in very soft and sensitive clays in Norway. The Norwegian Geotechnical Institute 54mm dia. sampler has the following characteristics (Void 1956; Berre *et al.* 1969):

maximum tube length	880 mm
maximum sample length	725 mm
outside diameter	57mm
area ratio	11—12%
inside clearance	1.0—1.3%
maximum length/diameter ratio	13.4

outside cutting edge angle	120

The NGI sampler has also been built in a 95mm version, giving 1000mm long samples (Berre *et al.* 1969). Evidence from oedometer tests suggests that 50cm2 specimens from 95 mm dia. piston samples give much less scattered test results than specimens from 54mm piston samples, in soft to firm clay. This may be a consequence of small-scale heterogeneity, or of reduced sampling or extrusion disturbance.

An adaptor kit is available from the UK company, Engineering Laboratory Equipment Ltd, to allow the 54mm NGI/Geonor sampler to take 101mm dia. sample tubes with lengths of either 457mm or 1000mm. These tubes are aluminium, with an outside bevel and no inside clearance. They have the following characteristics:

maximum sample length	330 or 875mm
outside diameter	105mm
area ratio	8%
length/diameter ratio	3.3 or 8.8.

These types of piston sampler have been used widely and reasonably successfully in the UK, where typically they are pushed into the soil at the base of a borehole. Their major disadvantage lies in the slow speed with which they can be used. Inner and outer rods are screwed on in I m lengths, and the complex holding mechanisms require careful use. To speed the sampling process several researchers have proposed mechanisms which allow a fixed piston sampler to be used with only one set of rods. Bretting (1936) developed a sampler with an integral hydraulically actuated piston, intended for use in a cased borehole. The piston was fixed to the outer barrel of the sampler and held at the required level by drill rods extending to the ground surface. The sample tube extended over the fixed piston to a second (upper) piston, which could be forced downwards by the application of water pressure to the inside of the drill rods at ground level. This principle was subsequently also used by Osterberg (1952, 1973) for his hydraulic piston sampler (see Fig. 7.13).

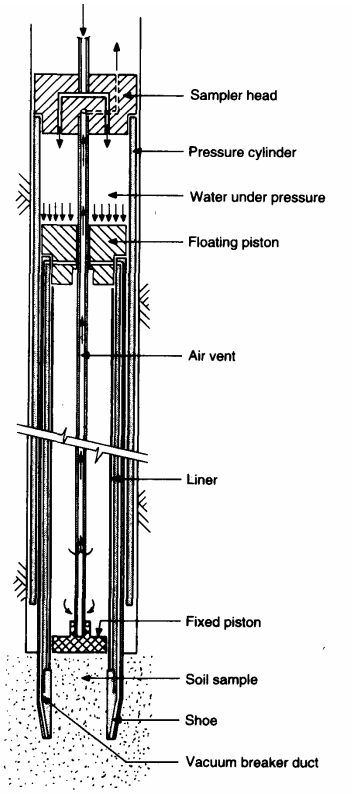


Fig. 7.13 The Osterberg composite hydraulic fixed piston sampler (Osterberg 1973).

The original Osterberg thin-walled hydraulic piston sampler was available in 127mm dia. (6% area ratio) and 72mm dia. (6.3% area ratio) forms. Raymond *et al.* (1971) and Raymond (1977) used this sampler with a restricted sample length of 600mm in the sensitive Leda clay and found it to give results second only to block sampling. The improved Osterberg composite hydraulic fixed piston sampler (Osterberg 1973) gives a maximum sample length of 1625mm with a diameter of 127 mm. It has an area ratio of 18%, inside clearance of 0.4%, a length to diameter ratio of 12.8 and a cutting edge angle of 7°. It has vents in the sample tube to allow vacuum relief below the tube when pulled from the soil, and the sampler can be rotated before pulling, in order to shear the soil at the base of the tube.

Fixed piston samplers are also widely used in Europe and the USA with liners. These composite fixed piston samplers have the advantages that they can be made more rugged and therefore more suitable for displacement boring, but the tube used to retain and store the soil after sampling can still remain lightweight and cheap. Evidence from sampling the soft, sensitive Leda clay (Raymond *et al.* 1971) suggests that the use of liners may be essential in small diameter samplers in some soil types, since both extrusion forces and the vibrations caused by sawing seamless steel tubing cause severe disturbance.

The Swedish composite fixed piston sampler (Kallstenius 1961) has a 700 mm stroke and a 50mm dia. at the cutting edge. Its outside diameter is 60mm, with plastic sectional liners of 170mm length and 50.2 mm inside diameter used to retain the sample. The base of the piston is 50° cone ended. Its characteristics are therefore:

maximum sample length	700mm
internal diameter at cutting shoe	50mm
area ratio	44%
inside clearance	0.4%
length to diameter ratio	14

Swedish experience has shown that very high quality samples can be obtained even with such a high area ratio and length to diameter ratio. Kallstenius (1958) showed the importance of cutting shoe design: for the Swedish standard piston sampler the taper is specified as 45° up to a thickness of 0.3mm and 5° thereafter. Similarly, in the USA, the Lowe—Acker composite fixed piston sampler (Lowe 1960) eliminates the effects of the high area ratio of a composite barrel by coupling a 150mm length of thin-walled tube to a 75mm long tapered coupling at its base. Despite a 70mm sample diameter and an area ratio of about 60%, Lowe (1960) has claimed that the effect of the thicker walled barrel section of the sampler is negligible.

The Swedish standard piston sampler is of rather small diameter compared with many modern devices, and the importance of a large diameter specimen in reducing sample disturbance and obtaining representative samples has already been noted. Holm and Holtz (1977) have presented the results of a study in which the Swedish Standard 50mm dia. device was compared with the NGI 95mm sampler, the Osterberg 127mm hydraulic piston sampler, and a 124mm dia. research sampler developed at the Swedish Geotechnical Institute. The results of this study indicate no significant differences between either the ratio (preconsolidation pressure/*in situ* vertical stress) or undrained shear strength derived from laboratory tests on specimens obtained by the various devices, but there are indications that:

1. results of oedometer tests on 50mm samples are more scattered, supporting the findings of Berre *et al.* (1969); and
2. the undrained modulus obtained from 50mm samples may be significantly lower.

Foil and stockinette samplers

Several devices have been developed to allow very long samples to be taken. Long samples are

particularly desirable when soil is highly variable, containing for example interbedded clays and sands. Begemann (1974) has argued that in these conditions the calculation of settlement or predictions of the effectiveness of vertical sand drains cannot be made without a full, undisturbed, detailed picture of the soil, and this view is certainly supported by the work of Rowe (1968a, b, 1972).

Inside friction can be reduced by the cautious use of inside clearance; it cannot be eliminated without causing serious disturbance to the soil inside the sampler as a consequence of the reduced support. The provision of sliding liners within the sampler barrel means that lateral restraint can be maintained while frictional forces between the soil and its container are eliminated. Two types of device are available: that developed by the Swedish Geotechnical Institute and described by Kjellman *et al.* (1950) inserts aluminium foils between soil and sampler, whilst a sampler developed at the Delft Soil Mechanics Laboratory by Begemann (1961, 1971) surrounds the soil with a nylon stocking reinforced plastic skin and supports it with bentonite fluid.

The principles of operation of the Swedish foil sampler are shown in Fig. 7.14. According to Broms and Hallen (1971), two types of sampler exist, giving sample diameters of either 68mm or 40mm. There are sixteen rolls of very thin high strength steel foil in the sampler head of the 68mm sampler. Each foil is 12.5mm wide, and about 0.1mm thick. The thickness may be varied depending on the required maximum sample length and the anticipated frictional forces to be resisted. The sampler is pushed into the soil without a borehole. As the sampler is pushed downwards the foils, which are attached to a stationary piston, unwind from their rolls and completely surround the sample.

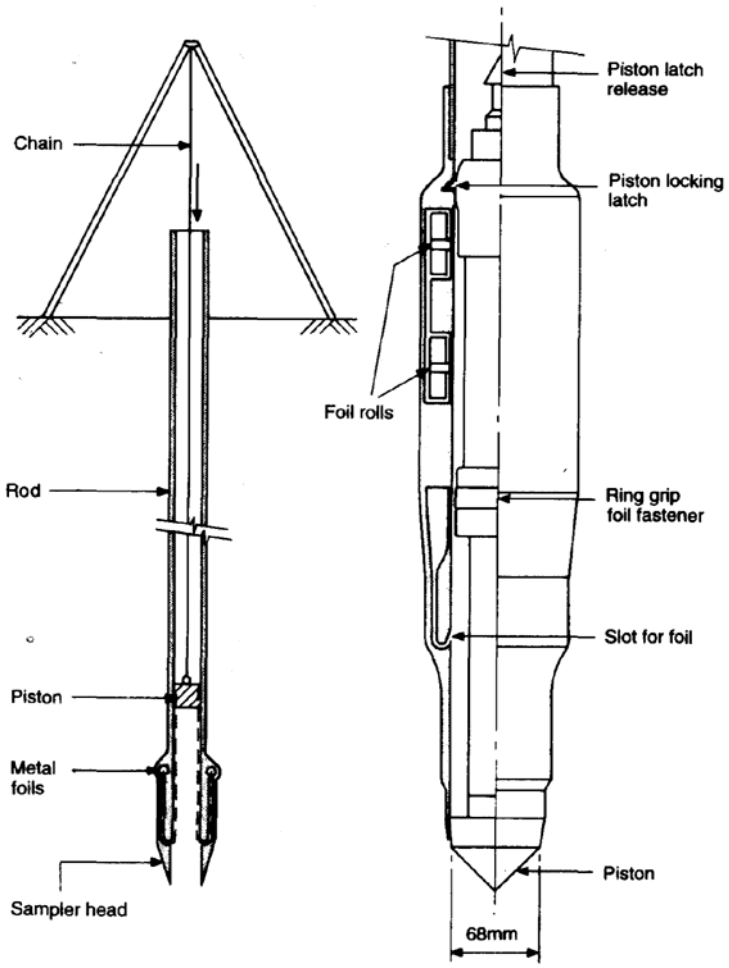


Fig. 7.14 Principle of operation of the Swedish foil sampler, and detail of the Mark V sampler head (Broms and Hallen 1971; Kjellman, Kallstenius and Wager 1950).

The maximum sample length that can be obtained depends on the strength of the foils and the size of the foils and their magazines. The 40mm foil sampler can hold a maximum length of 12m of foil, while the 68mm sampler can store 30m.

The friction between the foils and the sampler can be reduced by lubrication when sampling clays, and it has then been found possible to obtain continuous cores more than 20m long in very soft to soft soil. In sands, lubricants may penetrate the soil and cannot therefore be used; the length of sample is reduced.

The sampler is generally pushed or driven into soft cohesive soils. When silty or sandy soils are met, jetting may be needed to reduce the driving resistance. Ramming and jetting reduce the quality of the sample. Broms and Hallen (1971) describe a drilling rig for use with the foil sampler to obtain continuous samples of hard materials where rotary drilling is required. Using this equipment it has been possible to obtain cores of sand or hard boulder clay (till) up to l0m long.

The 66mm dia. *Delft continuous soil sampler* is shown in Fig. 7.15. In the early 30mm dia. version of this sampler (Begemann 1961, 1971) the soil entering the tube was primarily prevented from collapsing by bentonite fluid pressure. A stocking stored on the ‘stocking tube’ was surrounded by red vulcanizing fluid held in the chamber between the stocking tube and the outer sampler tube. The end of the stocking was secured to a cone-ended piston fixed at ground level. The inside of the sampler was filled with bentonite—water slurry. As the sampler was pushed into the ground the stocking unrolled from the stocking tube and rolled on to the outside of the soil. Contact between the slurry and the red vulcanizing fluid caused it to solidify, thus making a water-tight container, and preventing lateral strain of the soil.

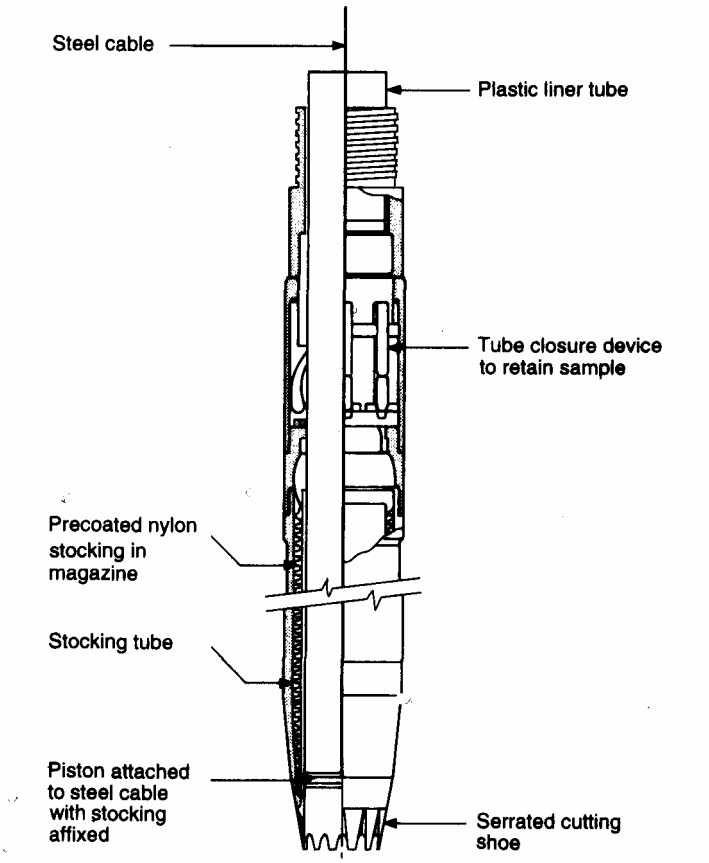


Fig. 7.15 66mm dia. Delft stocking sampler (Begemann 1974).

The sampler was originally pushed into the ground with a 10 tonne penetrometer, such as is used for the cone penetration test. The maximum sample length was determined by the maximum penetration

force available, and the length of stocking that could be stored. This varied between l0m and 20m. The sampler was advanced in 1 m drives, and after each section of tube was added it was filled with bentonite slurry. When the desired depth was reached, the bottom of the sample was closed by a diabolo valve operated by rotating the tubes at ground surface, to prevent loss of bentonite and soil during withdrawal.

The 66mm diameter continuous sampler (known in The Netherlands as a Begemann boring) provides samples large enough for laboratory consolidation and triaxial testing (Fig. 7.16). A bentonite flush, weighted with barytes to increase its density, is used in order to provide support for the sample as it passes the opening through which the stocking emerges. In soft soils the density apparently may need to be modified so that the imposed horizontal stress does not exceed that in the soil. The larger sampler includes a plastic liner which virtually eliminates the space between the soil and the inside of the extension tubes. These liners also provide support as the samples are transported to the laboratory. The maximum sample length for the 66mm sampler is 19 m according to Begemann (1974), and a 17 tonne penetrometer has been used.

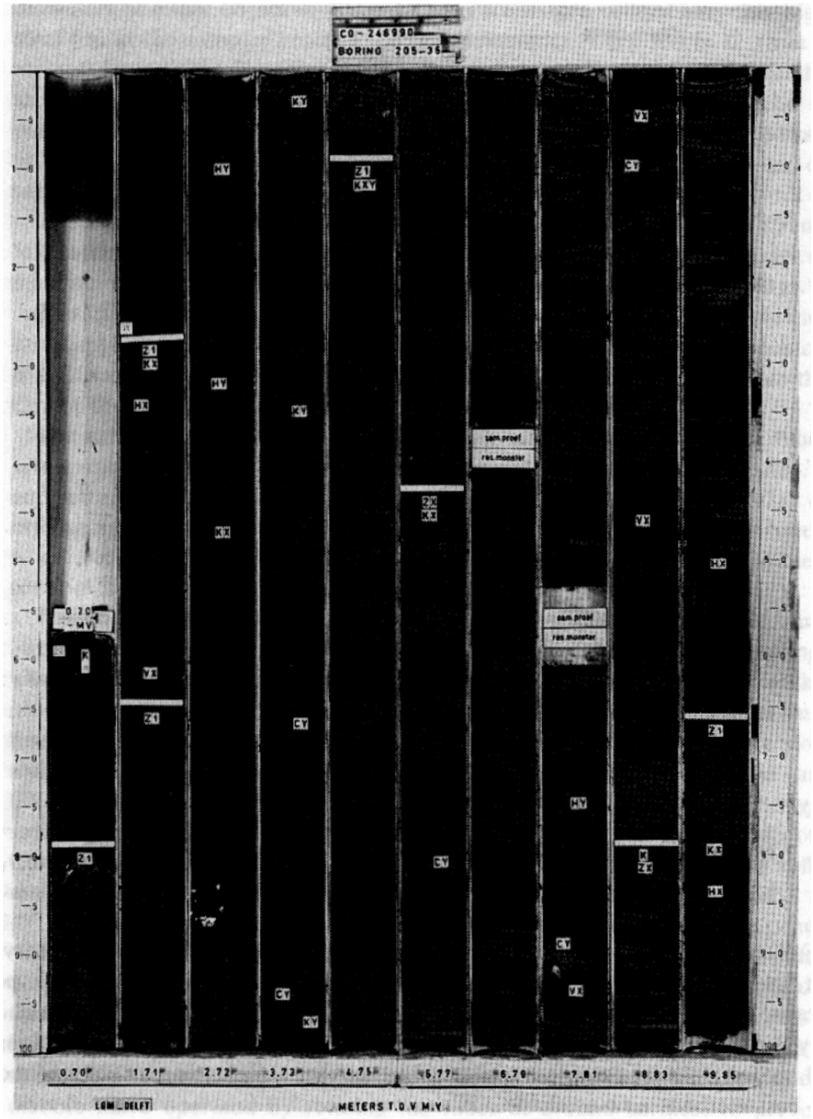


Fig. 7.16 Continuous l0m long sample in soft soil obtained using the Delft sampler (photograph courtesy Delft Geotechnics Laboratory).

Foil and stockinette samplers are relatively expensive, and are best reserved for soft, primarily cohesive soil types. They are no widely used in the UK, perhaps because large thicknesses of soft, fine sediment are not particularly common. Whilst they are very useful in providing a continuous record of

complex soil conditions, they will not normally be expected to give as high quality samples as may be obtained using the best methods now available.

ROTARY SAMPLERS

Chapter 5 described the general principles of rotary coring, as carried out routinely during site investigation. The main tool used for rotary sampling of hard rocks in the UK and most of the world is the rotary corebarrel (Fig. 5.11) but rotary samplers are increasingly being adapted to sample virtually all types of soil and rock. In this section we describe the three principal types of rotary corebarrel used on stiff and hard clays, and weak, fractured and weathered rocks. Also described is a simple rotary sampler designed to give very high quality samples in soft sensitive clays.

Rotary corebarrels designed to sample the harder materials encountered during site investigations can be classified into three broad groups:

1. corebarrels with retracted inner barrels, such as the conventional double-tube swivel type corebarrel (Fig. 5.11), here termed ‘retracted corebarrels’;
2. corebarrels where the inner barrel protrudes ahead of the outer barrel, in an attempt to protect the ground being sampled from the deleterious effects of flush fluid, here termed ‘protruding corebarrels’; and
3. corebarrels where the inner barrel is spring mounted, so that it protrudes in relatively soft ground, but retracts when harder layers are encountered, here termed ‘retractor barrels’.

Refracted corebarrels

Double-tube swivel type corebarrels

British site investigation practice uses large diameter double-tube swivel type core- barrels, normally with face discharge diamond bits and a built-in core catcher. Common barrel sizes in use in the UK are NW, HWF, PWF and SWF, giving core diameters of 54.0, 76.2, 92.1, and 112.7 mm. The double-tube corebarrel contains a stationary inner barrel supported on a swivel. Flush fluid is pumped down the inside of the rods which run from the drilling rig at ground level to the top of the corebarrel. Once inside the barrel the flush fluid passes down between the inner and outer barrels and discharges through ports in the cutting face of the bit. The inner barrel is extended with a core catcher box which contains a split ring core catcher. When the barrel is pulled from the bottom of the hole, the catcher spring prevents loss of core by moving down the inside taper of the catcher box and progressively gripping the core more tightly if it slips downwards.

Double-tube swivel type corebarrels of large diameter can be used with great success not only to provide good quality core of sound rock, but also to provide samples of very stiff or hard clays. Once the core enters the inner barrel it is protected from erosion of the flush water and from the torsional effects of rotation. The top of the inner barrel is vented to prevent build-up of pressure over the top of the core. Should this vent become blocked, the pressure in the inner barrel may prevent core entry after as little as 0.5 m of coring, and in softer formations the core may be washed away or ground away.

Wireline drilling techniques, coupled with polymer mud, are now frequently used in the stiff and hard Eocene formations of the London area (the London clay, the Woolwich and Reading beds, and the Thanet sand), as an alternative to thin-wall tube sampling, when higher quality samples than can be obtained using the BS U100 tube sampler are required.

Corebarrels tend to have a larger area ratio and inside clearance than is generally accepted for drive samplers. The former is an advantage, because one of the problems when drilling in soft formations is

to keep the pressure between the bit and rock or soil low enough to prevent the barrel fracturing or displacing the material beneath it. The larger inside clearance, which might be 2.4% for an HWF barrel, can cause serious problems. Although the core is protected from erosion by the flush fluid once it enters the inner barrel it is still in contact with that fluid and shales, mudstones and clays may deteriorate significantly if water flush is in use. Because the core is not well supported in the inner barrel, the effects of vibrations will be severe.

It is clear that when the highest possible recovery is required, the normally accepted rules to obtain fast economical progress with a diamond drill cannot be followed. Low bit pressures may reduce bit life by polishing the diamonds, and the low rotational speeds necessary to prevent vibrations from damaging the core will reduce the penetration speed. The main disadvantage of the double-tube swivel type corebarrel is that considerable skill and experience are required to use it successfully. When soil conditions are difficult both equipment and technique must be chosen with care: flush fluid, rig stroke, barrel length, diameter and design, and bit type will all be important.

The correct flush fluid can slow or even prevent the disintegration of the core. A long- stroke rig helps to reduce erosion and softening of the core by reducing the need for rechucking of the drill rods, and also lessens the chances of blocking if the flush pump is stopped during rechucking. Short barrels (say 1.0— 1.5 m long) of large diameter may be preferable to long thin barrels, because the effects of vibration and softening will be reduced. The use of liners of either rigid plastic or flexible plastic (Mylar) helps to reduce the effects of flush fluid, and largely eliminates damage to the core during withdrawal from the inner barrel. They can reduce inside clearance, and their smooth surface allows easy entry of the core.

Much damage can be done to good core during its extraction from the barrel. It is not uncommon to see core removed by holding the corebarrel almost vertical on a wire rope, and repeatedly hitting the inner barrel with a hammer. This method is not only likely to damage the inner barrel, but also will often damage the core. There is little or no chance of the driller being able to maintain the pieces of core in the same relative orientation as they occupy in the barrel while he struggles to place them in a corebox. Core should be extruded with the corebarrel held horizontally, using a coreplug in the inner barrel. Pressure should be smoothly applied to the back of the coreplug so that the core is extended with a minimum of vibration into a plastic receiving channel of about the same diameter as the core. After extrusion both core and plastic channel should be wrapped in clear polythene sheet and securely taped before being placed in the corebox.

Iwasaki *et al.* (1977) and Seko and Tobe (1977) have carried out comparative trials between double-tube swivel type corebarrels and a variety of other sampling devices. Iwasaki *et al.* found that a double-tube swivel type corebarrel with a face discharge bit, modified with a built-in check valve rather than a spring core catcher, an inner tube stabilizer and a reduced inside clearance of about 1.4% would give better results than a Denison sampler (see later) for clays with undrained shear strengths in excess of 150 to 200 kN/m2. Seko and Tobe carried out comparative sampling trials in the very stiff and sometimes hard clays of the Tokyo area: they used the samplers listed in Table 7.2.

Table 7.2 Samplers used in comparative sampling trials

	Type of sampler	Diameter of core (mm)
1	Double-tube swivel type corebarrel with a tungsten bit	60
2	Denison sampler	70-80
3	Retractor barrel, without a core catcher	
4	Retractor barrel, with core catcher	
5	Wireline Denison sampler	
6	Single-tube corebarrel	
7	Thin-wall hammered open-drive sampler	

All the rotary samplers were used with mudflush. It was found that the double-tube swivel type corebarrel gave the best specimens, based on unconfined compressive strengths and modulus of elasticity values. Types 2, 4 and 5 were classed as second best, but thought unreliable, often causing serious disturbance. Surprisingly, the retractor barrel without the core catcher gave worse results than that with a spring core catcher. Significantly, the single-tube corebarrel and the hammered thin-walled open-drive sampler were described as 'entirely unsuitable'.

Protruding corebarrels

In order to reduce the effects of flush fluid and torsional forces on the core, a number of devices have been developed in which the inner barrel extends below the bottom of the rotating corebit. The first of these devices was developed by the Ministry of Railways in Japan in the mid-1930s (Hvorslev 1940; Iwasaki *et al.* 1977). Subsequently Johnson (1940) reported the development of a similar corebarrel with a protruding inner barrel for taking samples of dense but erodible soils in the Denison District, Texas.

Denison corebarrel

The Denison corebarrel (Fig. 7.17) is a triple-tube swivel type corebarrel, with a shoe with a sharp cutting edge threaded onto the inner barrel and extending below the cutting teeth of a tungsten corebit. The length of the corebit must be changed to alter the amount by which the shoe extends below the corebit. According to Hvorlev (1949) and Lowe (1960) a 50—75 mm inner barrel protrusion is suitable for relatively loose or soft soils, whilst the cutting edge should be flush with the corebit in 'very stiff, dense and brittle' soils. The Denison corebarrel uses a 'basket' type spring core catcher, where a number of curved, thin, flexible springs are fixed to a base ring by rivets, or by welding. According to Hvorslev (1949) the use of such thin springs means that the core catcher is frequently damaged and must be replaced. The inner barrel encloses a liner, often of brass. The original inner barrel design by Johnson (1940) had a 32% area ratio, and a 0.6% inside clearance. The use of a high area ratio means that samples of hard clays and dense sands or gravels will be greatly disturbed, and better sampled by a conventional retracted inner barrel type sampler. Very soft to firm clays can be more effectively sampled with a fixed piston sampler. According to Lowe (1960) the Denison sampler is designed for use in stiff to hard cohesive soils and in sands. It is rarely used in the UK, where stiff clays are sampled using the 100mm thick-walled open-drive sampler, and the undisturbed sampling of sands is rarely attempted.

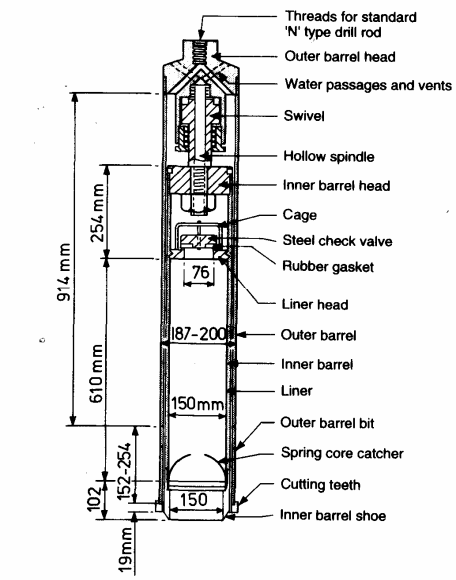


Fig. 7.17 Denison triple-tube corebarrel (Johnson 1940).

Retractor barrels

One of the problems facing the Denison corebarrel user is that the inner barrel protrusion must be pre-selected. To overcome this problem, several corebarrels have been developed which include spring mounted inner-barrels.

Pitcher sampler

The pitcher sampler (Terzaghi and Peck 1967; Morgenstern and Thomson 1971) is shown in Fig. 7.18. The inner barrel consists of a thin-walled sampler tube with a rolled and reamed cutting edge which is fixed to the inner head by set screws. The outer barrel has a tungsten insert corebit. The inner head is not fixed to the outer barrel; when the device is lowered to the bottom of the hole the head is supported immediately above the bit and flush fluid can be passed down the drill rods through the centre of the sample tube to remove any debris left at the bottom of the hole. Once the sample tube beds on to the soil at the bottom of the hole, the central tube on the top of the inner head mates with the outer barrel head. Flush fluid is now routed via the outside of the sampling tube, and the space above the sample is vented via the top of the sampler. The lead of the tube cutting edge is governed by the spring stiffness and the hardness of the soil.

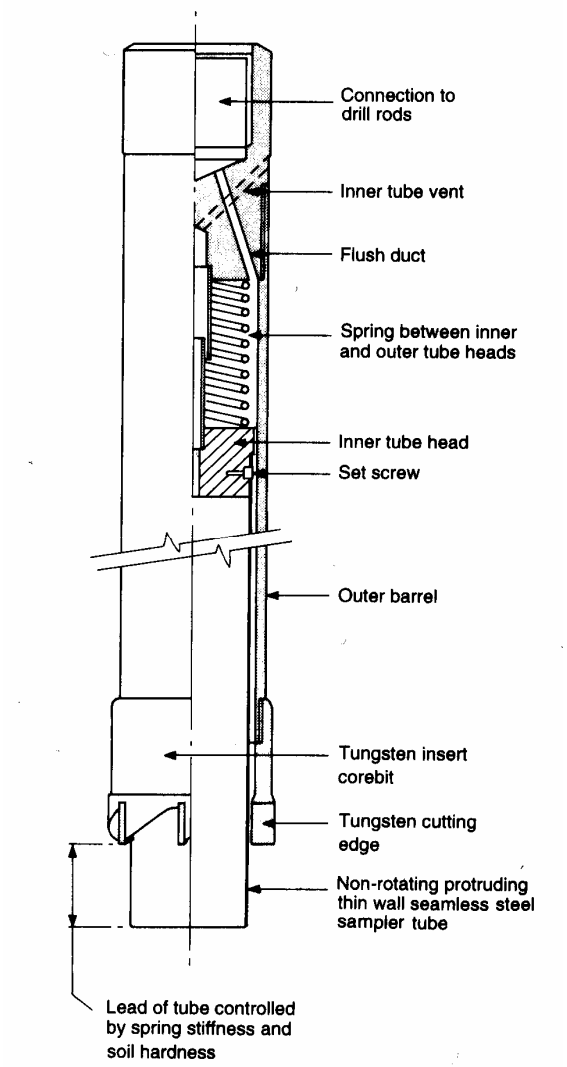


Fig. 7.18 Principal features of the Pitcher sampler.

In theory this type of sampler is ideally suited for drilling in soils with alternate hard and soft layers. In practice it has been found that hard friable soils, such as the weathered Keuper marl, can be sampled very successfully but frequently damage the rather light sampling tube. Thus this device is likely to be unsuitable for sampling inter-layered soil and rock, for example inter-layered clay and limestone. More rugged forms of retractor corebarrels have been developed in Australia by Triefus, and in France by Soletanche (Cambefort and Mazier 1961; Mazier 1974).

Mazier corebarrel

The Mazier corebarrel (Fig. 7.19) is a triple-tube swivel type retractor barrel, whose effectiveness (as with the Pitcher sampler) relies on the fact that the amount of inner barrel protrusion is controlled by a spring placed in the upper part of the device. The inner barrel contains a brass liner which can be used to transport samples to the laboratory, or for storage. The cutting shoe on the bottom of the inner barrel is substantial, making it much less easily damaged than a thin-walled seamless tube, but introducing the problems of disturbance when the high area ratio shoe travels ahead of the corebit.

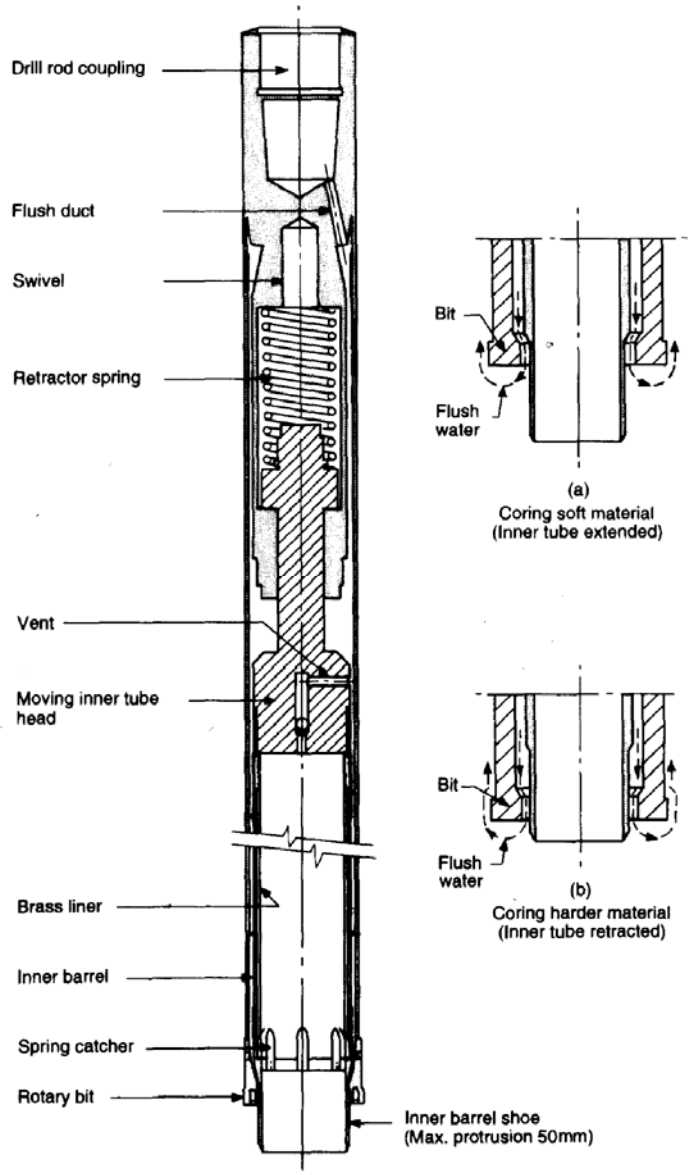


Fig. 7.19 Detail and principle of operation of the Mazier corebarrel.

Triefus triple-tube retractor corebarrel

The Triefus triple-tube retractor corebarrel uses the same principles as the Mazier device, but the maximum projection of the cutting shoe ahead of the corebit is much smaller (between 3mm and 16mm). This means that the absolute minimum of flush fluid should be used, in order to prevent washing and scour of soft or friable formations. Unlike the Mazier barrel, no core catcher is used and the base of the barrel must be sealed by dry blocking (i.e. drilling without flush fluid) at the end of the run. The liner tubes in the Triefus barrel can be either split steel, or solid transparent plastic, the latter allowing inspection of the core in the field without removal from its tube.

Additional major advantages of the Triefus barrel are that most of its components are interchangeable with those of their triple-tube standard corebarrels (allowing the standard barrel to be converted to a retractor barrel in the field), and that both these barrels are fitted with a core extractor plug (blow-out valve) to allow the core in its liner to be pushed gently from the barrel whilst in a horizontal position.

Retractor barrels find their best use in formations of variable hardness, where piston drive sampling cannot penetrate and when standard rotary coring provides insufficient protection to the soil. In general, thin-walled sampler retractor barrels such as the Pitcher sampler are susceptible to tube damage, while those with thick cutting shoes are much more likely to cause serious disturbance to the soil. Where soil conditions are relatively uniform and the soil is of sufficient undrained shear strength (>150—200kN/m2) the results of Iwasaki *et al.* (1977) and Seko and Tobe (1977) indicate that a double-tube swivel type rotary corebarrel can provide less disturbed samples than either protruding or retracting inner tube barrels, or driven thick-walled open-drive samplers.

SAND SAMPLING

‘Practical undisturbed’ samples, in Hvorslev’s terms, can be obtained from sand deposits: but despite the fact that the soil structure, water content, void ratio and constituents remain unaltered such samples are not normally suitable for compressibility testing. Indeed, Broms (1980) asserts that such samples cannot be obtained of a sufficiently high quality to allow determinations of potential liquefaction problems. The effects of total stress relief on granular soils usually result in very large reductions in effective stresses, and the properties of such soils tend to be highly dependent both on stress history and current effective stress level. But in addition, small changes in shear stress can also destroy the ‘memory’ of previous stress applications, leading to considerable reduction in stiffness, in sands.

Undisturbed sand sampling can be very expensive, and is normally only required in special circumstances, for example to obtain values of *in situ* density for earthquake liquefaction problems or for compressibility studies. In Japan, for example, the havoc created by earthquake damage has caused a considerable interest in the sampling of granular materials and has lead to widespread research and publication (for example, Yamada and Uezawa 1969: Hanzawa and Matsuda 1977; Isihara and Silver 1977; Seko and Tobe 1977: Tohno 1977 and Yoshimi *et al.* 1977).

Hvorslev (1949) outlined a number of techniques for sampling sand using:

1. thin-wall fixed piston samplers in mud-filled holes;
2. open-drive samplers under compressed air;
3. impregnation;
4. freezing;
5. core catchers.

Fixed piston sampling

A study of the effectiveness of thin-walled fixed piston sampling has been carried out at full-scale in

the laboratory by Marcuson *et al.* (1977). Using a 1.22m dia. x 1.83m high specimen of fine sand, and mud flush rotary drilling, they obtained samples using a 76.2mm dia. piston sampler with an area ratio of 11%. By comparison between soil densities when placed and after sampling it was found that samples of dense sand were slightly loosened and samples of loose sand densified, but the results suggest an accuracy within ±3.5% of the placed density for sands with relative densities of between 20% and 60%. Bearing in mind the difficulty of creating uniform samples (the authors suggest a variability of density at the time of placement of about + 3.2% to —1.6%) these results seem encouraging, and support the view of Friis (1961) regarding the value of thin-walled fixed piston sampling under drilling mud in sand.

The use of compressed air to displace water from around the sampler tube in the borehole, and thus reduce the losses of samples in sand was first suggested by Glossop to Bishop (1948) and independently by Vargas to Hvorslev (1949).

Bishop's sand sampler

Bishop's sand sampler (Bishop 1948; Nixon 1954; Serota and Jennings 1957) consists of a 63.5mm thin-walled open-drive sampler held by set screws to a head containing a rubber diaphragm check valve and vents (see Fig. 7.20). This assembly is mounted within a compressed air bell which is connected to an air pump at the ground surface. The sampler is used in the following manner.

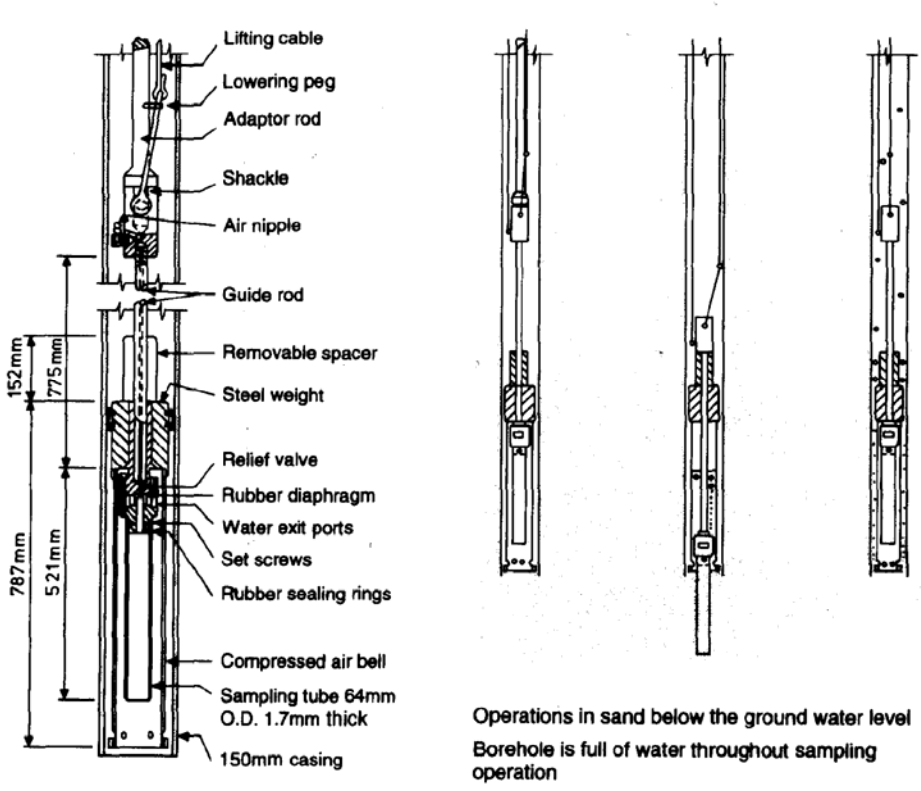


Fig. 7.20 The Bishop compressed air sand sampler (Bishop 1948).

1. Lower the sampler inside the compressed air bell to the base of a cleaned borehole.
2. Push the sample tube ahead of the bell into the soil, using the rods.
3. Remove the rods, and force compressed air into the bell via the relief valve in the sampler head. The relief valve vents to the inside of the bell at a pressure difference of about 150 kN/m2, and this pressure bears on the upper surface of the diaphragm, ensuring that it works efficiently.

4. Once air bubbles rising to the surface of the water in the borehole indicate that the water in the bell has been expelled, the sample tube is pulled from the soil into the bell, and then rapidly brought to ground surface using the lifting cable.
5. Remove the spacer, push the sample tube and head out of the bell, and release the sample tube set screws.
6. Cap the base of the tube and release the check valve before removing the sample tube from the head.

The Bishop sand sampler uses arching and the small capillary suctions set up at the sand/air interface to reduce sample losses. This principle has since been used by Yamada and Uezawa (1969).

Soil impregnation

Van Bruggen (1936) described the use of soil impregnation with a dilute emulsion of asphaltic bitumen, in order to impart cohesion to granular soils and thus allow them to be sampled. The bitumen was subsequently removed by washing through with a solution of carbon disulphide and acetone. Such a process would certainly change the properties of the soil. Hvorslev (1949) describes the use of chemical injection around the cutting shoe of an open-drive sampler to solidify soil and help to retain samples of granular material. Subsequently Karol (1970) and Borowicka (1973) have reported the use of various resin and silica grouts to prevent disturbance to the soil structure during sampling. Impregnation and injection are expensive and relatively difficult to use: the soil to be sampled cannot be impregnated unless the chemicals and so on, can be effectively removed at a later date, and in addition, most grouts and emulsions will not penetrate relatively impervious sand or silt soils. The method is therefore rarely used.

Freezing

In contrast, freezing has been widely used to seal the bottom of sample tubes (once driven), to prevent disturbance to the soil during transporting to the laboratory, and to freeze soil before sampling, (for example, Fahlquist 1941; Hvorslev 1949; Ducker 1969 and Yoshimi *et al*. 1977). These techniques are very expensive, and yet the need for undisturbed silt and sand samples has resulted in their continued use, particularly in the earthquake areas of the Far East and the USA. A relatively economical method of obtaining frozen sand samples is shown in Fig. 7.21. A 73mm thin-wall steel tube is inserted into the ground, while removing soil from its interior with an auger. Once the desired depth of freezing is reached, the lower end of the tube is sealed with cement grout and the plastic ‘freezing tube’ inserted down the centre of the steel tube. Circulation of an ethanol and crushed dry ice coolant at -40 to -60°C results in a frozen column of soil which can be extracted by pulling the steel tube from the ground. At ambient air temperatures of 23—30°C and a ground temperature of 22°C at 0.9 m depth, Yoshimi *et al.* (1977) acquired columnar samples 5.8 m long and 380mm dia. after 16 h of coolant circulation.

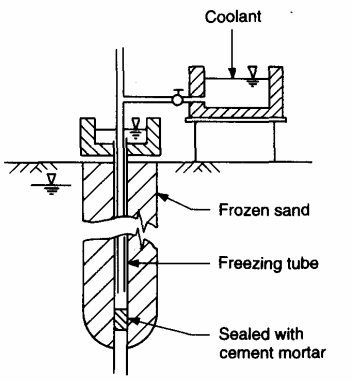


Fig. 7.21 Method of sand sampling by freezing adopted by Yoshimi, Oh-Oka (1977).

The value of samples obtained by freezing depends on the amount of density and soil structure change caused by the process. The amount of strain taking place during freezing increases with increasing relative density, decreasing applied pressure, and increased freezing time. Tests by Yoshimi *et al.* on specimens of soil placed in the laboratory and on samples obtained using the freezing tube technique described above indicate that soil adjacent to the freezing tube was significantly altered by freezing; loose sand was densified, but medium dense and dense sands loosened. Densities in the outer half of the sample, however, showed no alteration and indicated values mostly within ±2% of the average placed or field dry density.

Core catchers

Core catchers can be used with great success to retain granular soils, but their design may introduce considerable disturbance during the sampler drive. Spring systems such as used in rotary samplers are examples of the types of catching device which should be used with great caution. The catchers described by Yamada and Uezawa (1969) and Isihara and Silver (1977) are examples of systems which should lead to a minimum of disturbance.

SAMPLER SELECTION

For reasons of economy it is sensible to adopt the cheapest sampling techniques compatible with the aims of an investigation. To do this, the reasons for drilling and sampling must be clearly defined. In many cases only very simple methods of sampling will be necessary — the priority will be the proper identification of soil type and soil fabric, and there will be little need to obtain realistic soil parameters. In other instances, for example when complex computer analyses are to be undertaken during design, much higher quality samples will be required so that sophisticated laboratory tests can be carried out.

The German Standard DIN 4021 defined five quality classes of soil sample, and quality classes have also been discussed by Idel *et al.* (1969), and by Rowe (1972), and have been defined in BS 5930:1981. In principle they identify the range of samples shown in Table 7.3.

Table 7.3 Soil sample quality classes

Quality class	State of soil sample
1	No geometric distortion. Shear strength and compressibility are unaffected.
2	Geometric distortion. Density and water content unaffected.
3	Density altered. Water content and particle size distribution unaffected.
4	Water content and density altered. Particle size distribution unaffected.
5	Particle size distribution altered by loss of fines or grain crushing.

Based upon the research of the past decade, this classification no longer seems relevant and, indeed, it does not appear to have been used in practice. We can note, for example, that Quality class 1 samples cannot be obtained with routine tube samplers, and that the shear strength and compressibility of even the best quality block samples are likely to be affected as a result of changes in effective stress during sampling. It is difficult to envisage how water content can remain unaffected when density changes, given that most soils are likely to be saturated when *in situ*. And inadequate particle size distributions may result as much from a poor choice of sample size as from poor sampling technique.

In the UK the typical ‘routine’ soil sampling associated with light percussion drilling is as follows.

1. *All soils*. Small disturbed sample (jar sample) immediately each new stratum is detected. Water samples from every water strike.
2. *Sand.* Standard penetration test (SPT), with a jar sample taken from the split spoon, immediately on entering the stratum, and at 1.5 m intervals thereafter. If no sample is recovered by the split spoon a large disturbed sample (bulk bag) should be taken from the same level. Bulk bags should be taken at 1.5m intervals midway between the SPT tests.
3. *Sand and gravel, or gravel*. Standard penetration test with 60° cone (SPT cone), immediately on entering the stratum, and at 1.5m intervals thereafter. A large disturbed sample (bulk bag) should be taken from the level of the SPT (cone) test.
4. *Chalk, marl or silt*. Alternate 100mm thick-walled open-drive samples (UlOOs) and SPT tests with jar samples taken from the split spoon sampler, at 1.5 m intervals. In hard marls and cemented silts an ‘H’ size double-tube swivel type corebarrel with air flush is sometimes used.
5. *Hard clay.* U100 samples may be possible, but otherwise SPT tests are carried out. Exceptionally, mud flush and rotary coring is used.
6. *Stiff clay*. U100 sample immediately upon entering the stratum and then at 1.5 m intervals. A small disturbed sample is taken from the cutting shoe of the U100 samples, and at 1.5 m intervals midway between the U100 samples.
7. *Very soft clay or peat.* 54mm or 100mm dia. thin-walled fixed piston samples, either continuously in a shallow deposit, or at 1 m intervals in a deep deposit.
8. *Rock.* Continuous rotary core from a double-tube swivel type corebarrel, with water flush. The minimum size is usually ‘N’.

This scheme leaves the ‘variation survey’, that is the assessment of soil variability, as well as many of the decisions about sampling, and the recognition of strata changes, to the drilling foreman. He is the only person to see the majority of the soil profile, and must watch the disturbed material produced by the drilling process most carefully so that subtle (but perhaps significant) features are not to be missed. This type of ground investigation, and the routine testing practices associated with it can now be viewed as barely adequate. The ground profile is not properly examined, the samples obtained are typically of low quality, and the testing carried out may often be unnecessary, or inappropriate for the engineering needs of the project.

When proper design of ground investigation is carried out there is a need to match the sophistication of sampling (and *in situ* testing) to the sophistication of the analysis and design of the project. Empirical and semi-empirical methods of design are certainly acceptable, and should be used in conjunction with the appropriate sampling and test methods upon which they were originally based. But more fundamental methods of analysis, for example for the constitutive modelling of soil behaviour via finite element or finite difference analyses, will require high-quality sampling and testing methods. From the range of sampling methods described, the methods shown in Table 7.4 are the best available.

Table 7.4 Recommended sampling methods

Soil type	Recommended sampling technique	Likely disturbance
Very soft, soft, firm, or sensitive clays	Laval open-drive overcored sampler	Minor destructuring
	Sherbrooke down-hole block sampler	Reduction in effective stress due to borehole fluid penetration
Inter-layered sand, silt, clay	Delft sampler	Major loss of effective stress. Some destructuring
Firm, stiff and very stiff clays	Thin-walled hydraulically jacked open-drive tube samples	Minor destructuring, with significant increases in effective stress
Very stiff and hard clays, mudrocks, and stoney clays	Wireline coring, using bentonite mud or polymer muds with anti-swelling agents, or double-tube swivel type corebarrel with bentonite mud flush	Significant decrease in effective stress
Sand	Piston sampling in mud-filled borehole	Total loss of effective stress. Major destructuring. Density approximately maintained
Gravel	Sampling from pits	Only particle size and density can be obtained
Weak rocks, chalk	Triple-tube swivel-type corebarrel with mud or msf flush, or retractor barrel	Minor core loss Discontinuities opened
Decomposed granite	Treifus or Mazier retractor barrel	Minor core loss. Effective stress loss
Hard rock	Double-tube swivel-type corebarrel	Discontinuities opened

Chapter 8

Laboratory testing

INTRODUCTION

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

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

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

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

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

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

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

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

THE PURPOSE OF SOIL TESTING

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

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

AVAILABLE TESTS

This chapter sets out to describe individual test techniques in detail: texts such as Lambe (1951), Bishop and Henkel (1962), Akroyd (1964), Vickers (1978), Head (1980), Head (1982), Head (1986), BS 1377:1990 and ASTM Part 19, should be referred to for the methods used in each test. Soil tests are loosely brought into two groups in this section; the first provides information to allow the

classification of soil into arbitrary groups while the second includes all tests which provide parameters which may be used in geotechnical calculation and design (Table 8.1).

Table 8.1 Soil classification tests and test parameters

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

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

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

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

Soil classification tests

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

Particle size distribution tests

BS 1377:1990 gives four methods for determining the particle size distribution of soils (part 2, clauses 9.2—9.5). The coarse fraction of the soil (>0.06mm approximately) is tested by passing it through a

series of sieves with diminishing apertures. The particle size distribution is obtained from records of the weight of soil particles retained on each sieve and is usually shown as a graph of 'percentage passing by weight' as a function of particle size (Fig. 8.1).

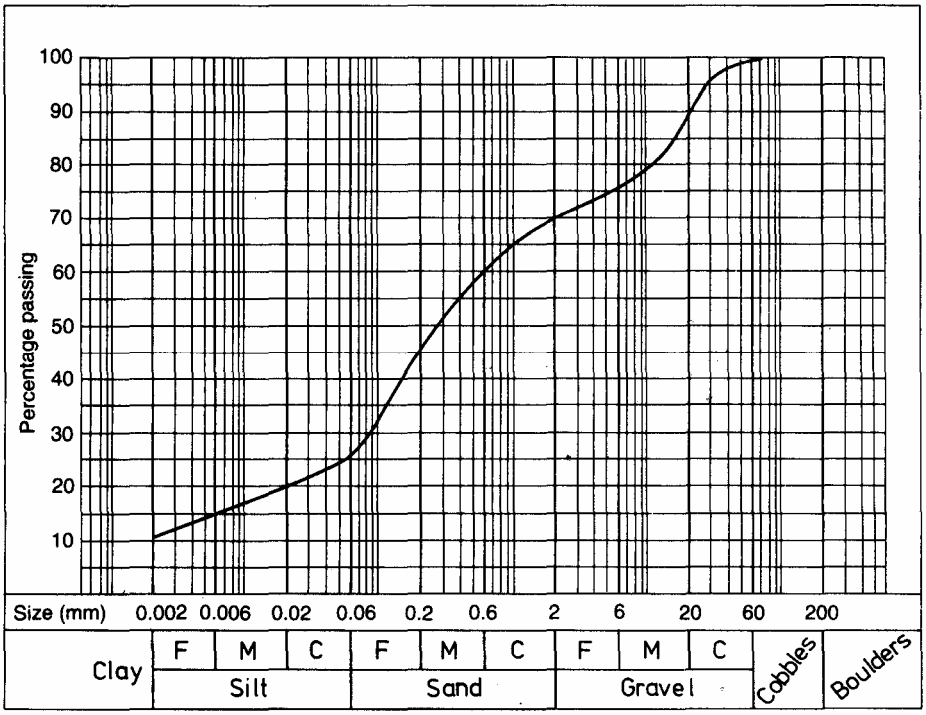


Fig. 8.1 Typical particle size distribution.

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

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

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

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

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

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

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

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

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

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

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

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

Plasticity tests

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

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

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

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

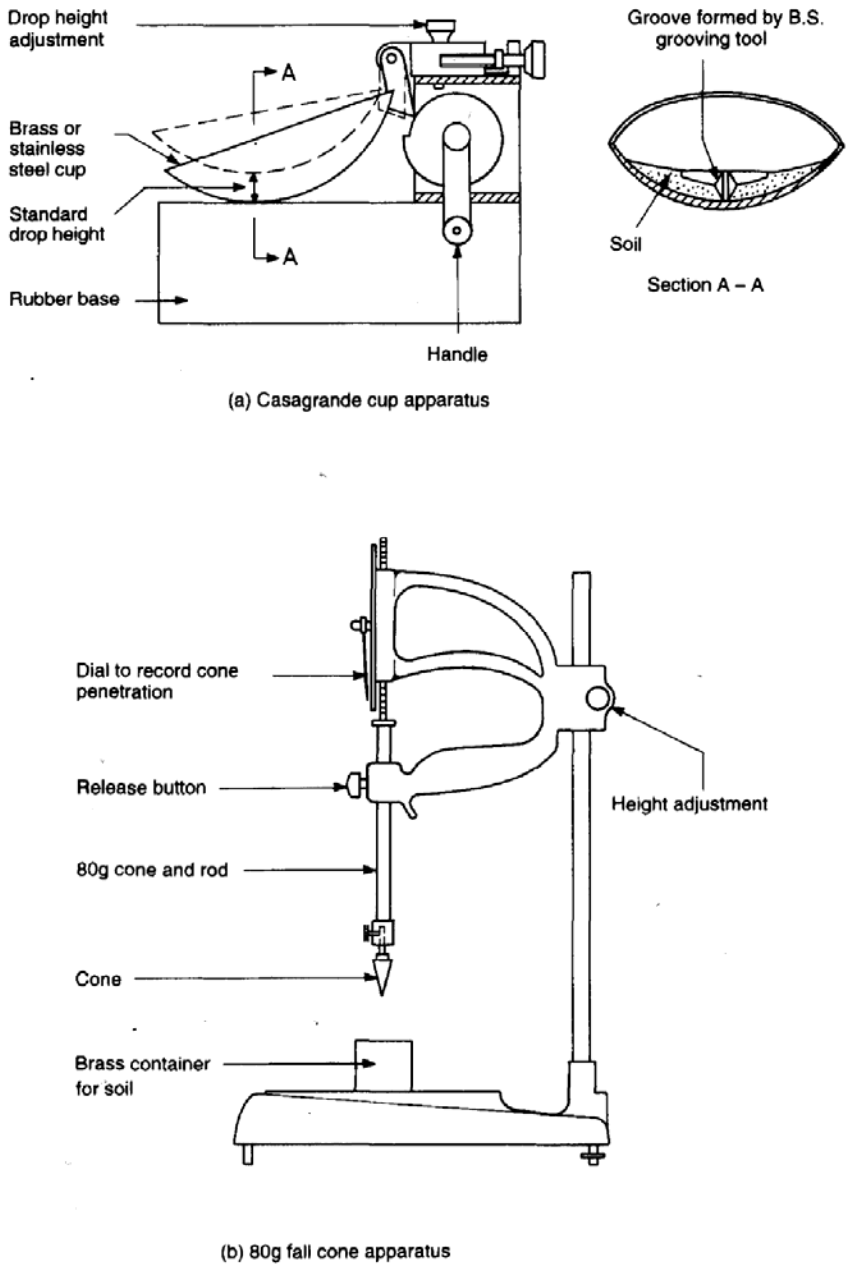


Fig. 8.2 Casagrande cup and cone penetrometer for liquid limit testing.

Table 8.2 Results of comparative testing programme

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

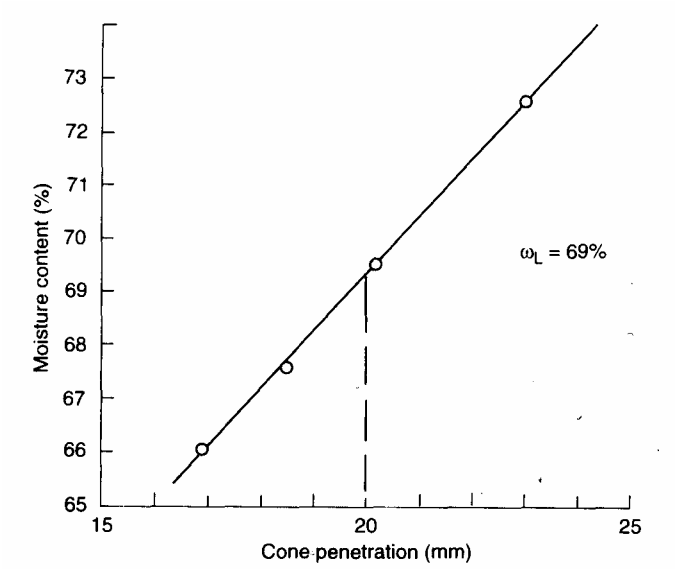


Fig. 8.3 Liquid limit result by four-point cone method.

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

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

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

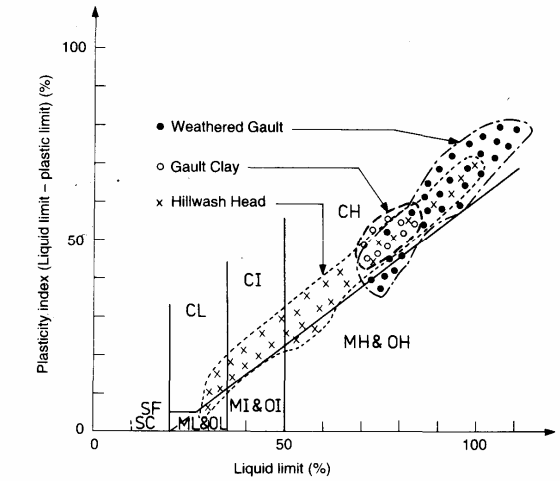


Fig. 8.4 Casagrande plot showing classification of soil into groups.

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

Table 8.3 Relative costs of plasticity tests

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

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

Compaction tests

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

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

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

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

its moisture content determined. The process was repeated for several increasing moisture contents, and a compaction curve (i.e. dry density as a function of moisture content) was obtained (Fig. 8.5).

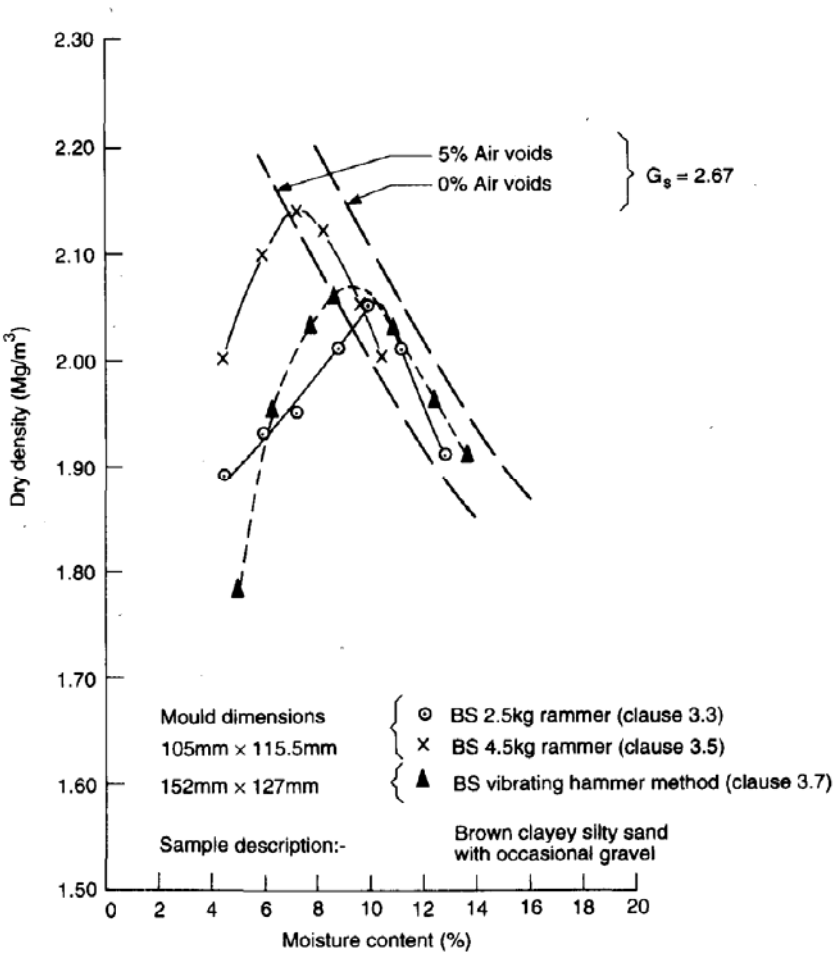


Fig. 8.5 Compaction curves.

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

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

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

Table 8.4 Comparison of compaction tests

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

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

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

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

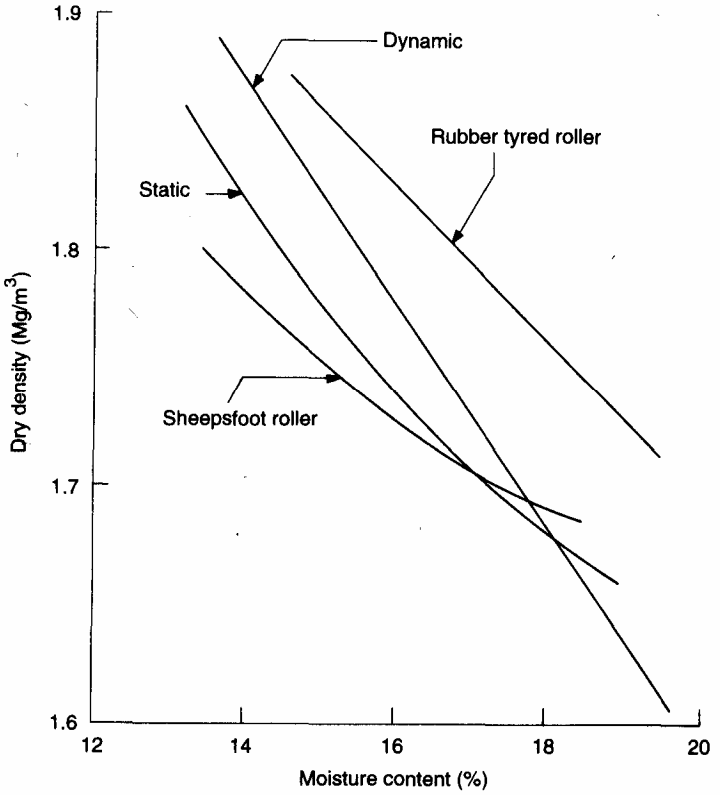


Fig. 8.6 Lines of optimum moisture content/maximum dry density for laboratory compaction methods and two types of field compaction (Foster 1962).

Particle density (specific gravity) determination

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

The method still used by most test houses to determine the particle density of fine-grained soil utilizes a 50m1 density bottle (BS 1377:part 2:1990 clause 8.3). Unfortunately there is no simple means of knowing when all the air has been removed from the bottle and hence the soil must be de-aired under vacuum. The use of de-aired water will help but it is still necessary to leave the sample in the density bottle under vacuum for several hours. The major difficulty with this test is the provision of a satisfactory vacuum and measuring the length of time required to remove the air completely. These factors can clearly lead to errors in specific gravity determinations. Krawczyk (1969) found that the difficulties of de-airing the soil could be overcome by shaking the sample instead of placing it under vacuum. The advantages of shaking are that the shaking action is easily standardized and the removal of air is more rapid than by the application of a vacuum. Krawczyk proposed that the test should be carried out in a 1 litre gas jar to make the same test suitable for fine-, medium- and coarse-grained soils and the shaking action provided by an end-over-end shaker. This alternative method has been included in the British Standard (BS 1377:part 2:1990, clause 8.2) and should be treated as the preferred method, since in providing a more reliable technique of de-airing the soil it yields more repeatable results.

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

Table 8.5 Comparison of particle density results

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

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

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

Tests for geotechnical parameters

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

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

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

Strength tests

The principal tools available for strength determination in a good UK geotechnical testing laboratory are the California Bearing Ratio (CBR) apparatus, the Franklin Point Load Test apparatus (Franklin *et*

al. 1971; Broch and Franklin 1972), the laboratory vane apparatus and various forms of direct shear and triaxial apparatus.

California bearing ratio (CBR) test

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

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

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

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

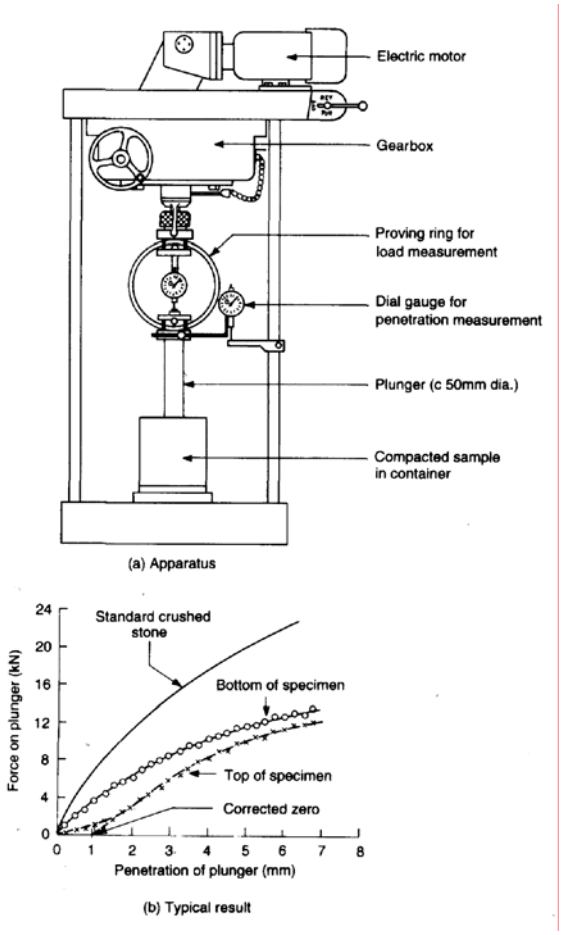


Fig. 8.7 CBR apparatus and typical result.

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

Franklin point load test

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

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

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

Laboratory vane test

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

Direct shear test

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

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

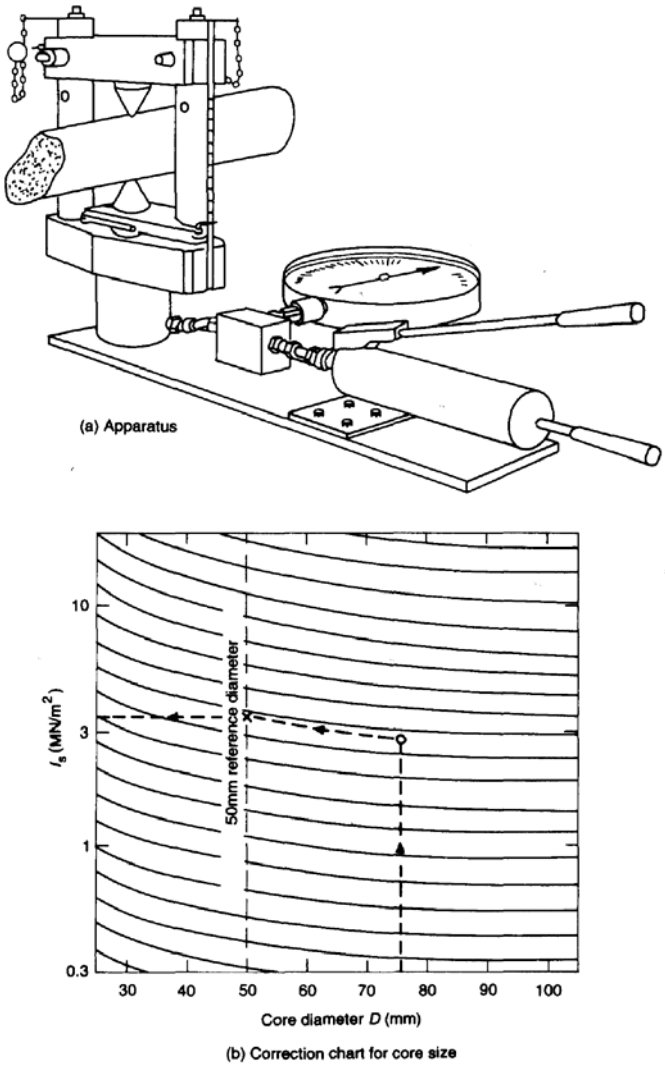


Fig. 8.8 Point load apparatus (Broch and Franklin 1972; Brown and Phillips 1977).

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

In the UK, shear box tests are now used mainly to determine residual shear strength parameters for the analysis of pre-existing slope instability (Skempton 1964; Skempton and Petley 1967). In this application the technique of cut-plane shear box testing described by Petley (1966) gives results which have been found to be satisfactory, based on back-analysis (for example see Foster (1980)). A specimen of clay is placed in the shear box and allowed to swell for 24h under the weight of the load hanger. Following this, the specimen is consolidated under the required normal pressure and measurements of vertical compression are made. The two halves of the box are then separated

sufficiently to allow a cheese wire to cut smoothly through the specimen. The two halves of the specimen are then separated, and the soil surfaces smoothed by rubbing a glass plate lubricated by distilled water over the surfaces. When smooth, the lower half of the soil specimen is raised by packing it with a few layers of filter paper: the box is reassembled, and after applying a normal stress, the specimen is subjected to large displacements on the preformed shear surface by repeatedly reversing the travel of the box. The maximum shear stress obtained for each stage of shearing should be plotted against the logarithm of cumulative displacement, and shearing should continue until this curve levels out. The lowest maximum shear stress values (in the final shear stage of each test) are plotted against their imposed normal stresses to obtain the residual effective strength parameters (c'_r (normally zero) and φ'_r) for a soil. Typical values are given in Table 8.6.

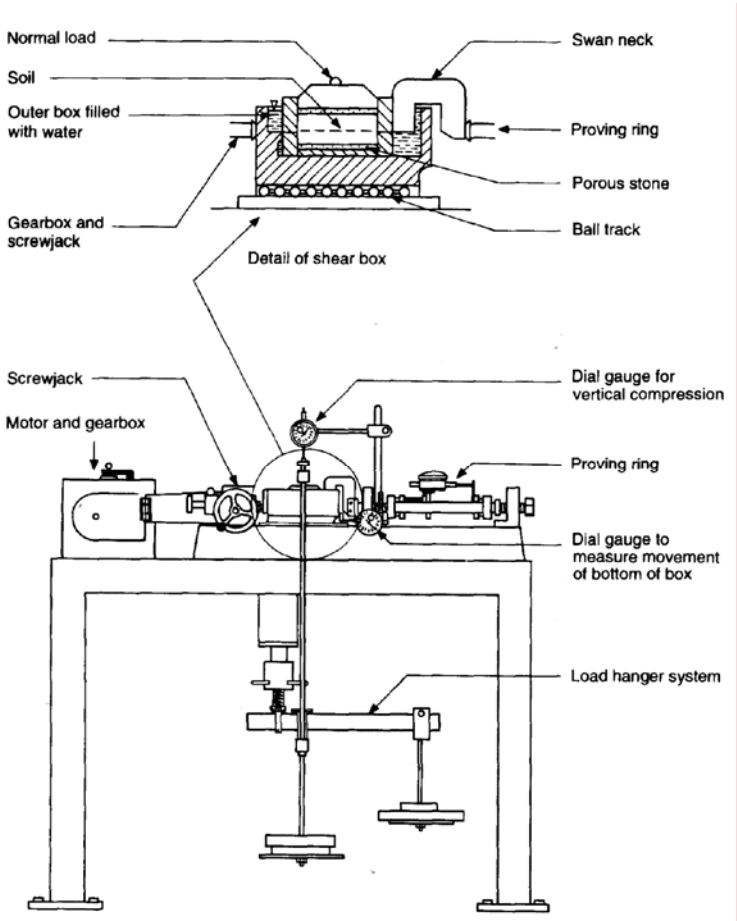


Fig. 8.9 Bishop direct shear box.

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

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

Table 8.6 Effective strength parameters for some UK soils

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

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

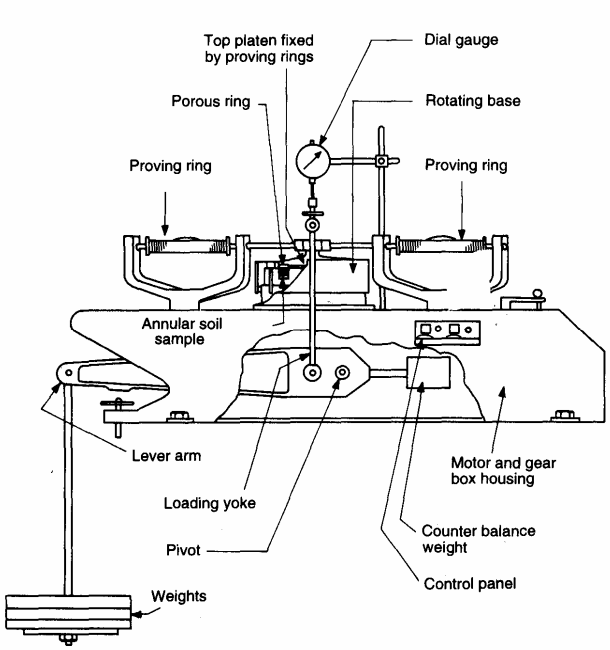


Fig. 8.10 Bromhead ring shear apparatus (Bromhead 1979).

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

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

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

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

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

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

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

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

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

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

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

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

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

Triaxial test

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

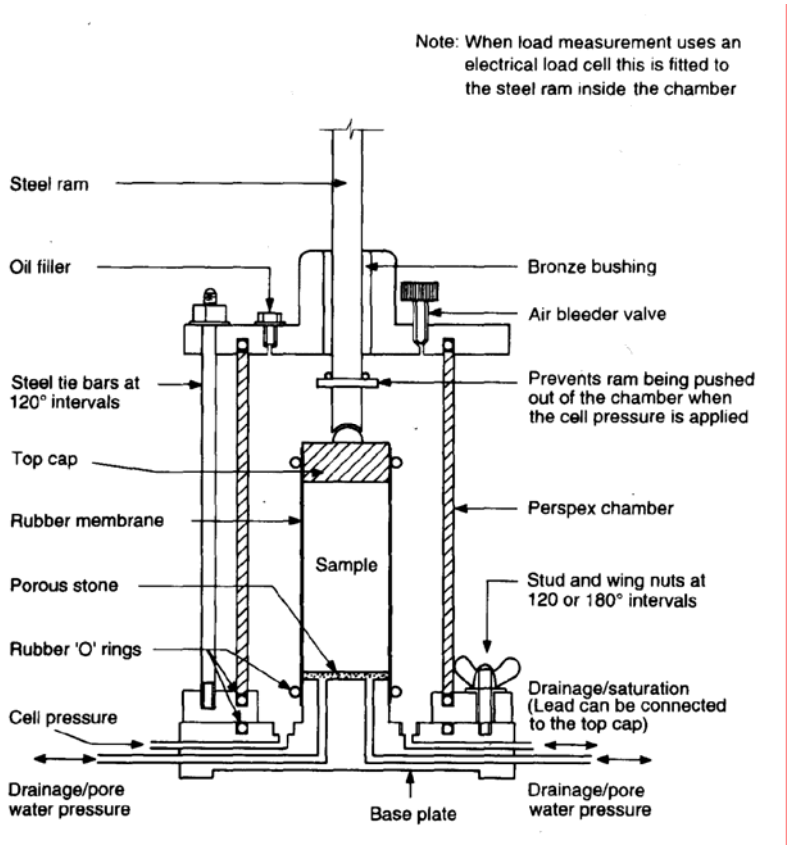


Fig. 8.11 Triaxial cell.

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

When the consolidation cell pressure and back pressure are applied to the specimen, readings of volume change are made using a volume change device in the back pressure line. The speed at which volume change takes place depends on the effective pressure increment, the coefficient of consolidation of the soil and the drainage conditions at the specimen boundaries. Normally, pore

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

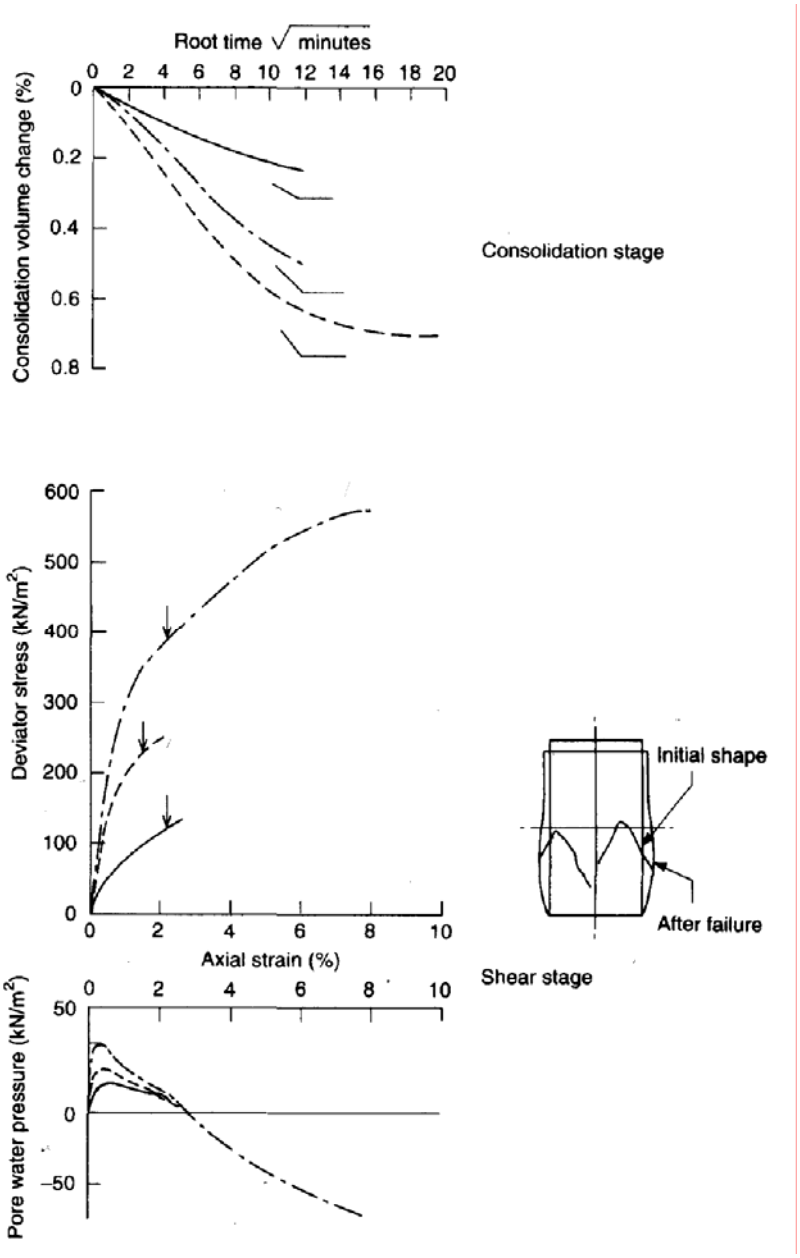


Fig. 8.12 Consolidation and shear stage results for a consolidated undrained triaxial compression test with pore pressure measurement. (Arrows denote principal stress ratio failure, $(\sigma_1'/\sigma_3')_{max}$.)

For drainage from one end only:

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

For drainage from both ends and radial:

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

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

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

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

For drainage from one end only:

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

For drainage from both ends and radial boundary:

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

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

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

$t_f = 0.53\ t_{100}$ for drainage from one end (8.13)

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

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

Once consolidation is complete, the specimen may be isolated from the back pressure and the rate of vertical movement of the compression machine platen set. For this, the minimum time to failure is divided into the estimated axial sample deformation at failure (or at the time of the first valid readings). Soil will normally fail at axial strains of between 2 and 20%, and the actual figure used is largely based on experience of testing similar soil types. During the shear stage the vertical stress is

increased by the loading ram, and measurements are made at regular intervals of deformation, ram load and pore pressure. These are converted to graphs of principal stress difference (σ_1- σ_3) and pore pressure as a function of strain (Fig. 8.12), and failure is normally taken as the point of maximum principal stress difference. The effective stress Mohr circles are plotted for the failure conditions of the three specimens, and the gradient and intercept of a straight line drawn tangential to these circles defines the effective strength parameters c' and φ' (Fig. 8.13).

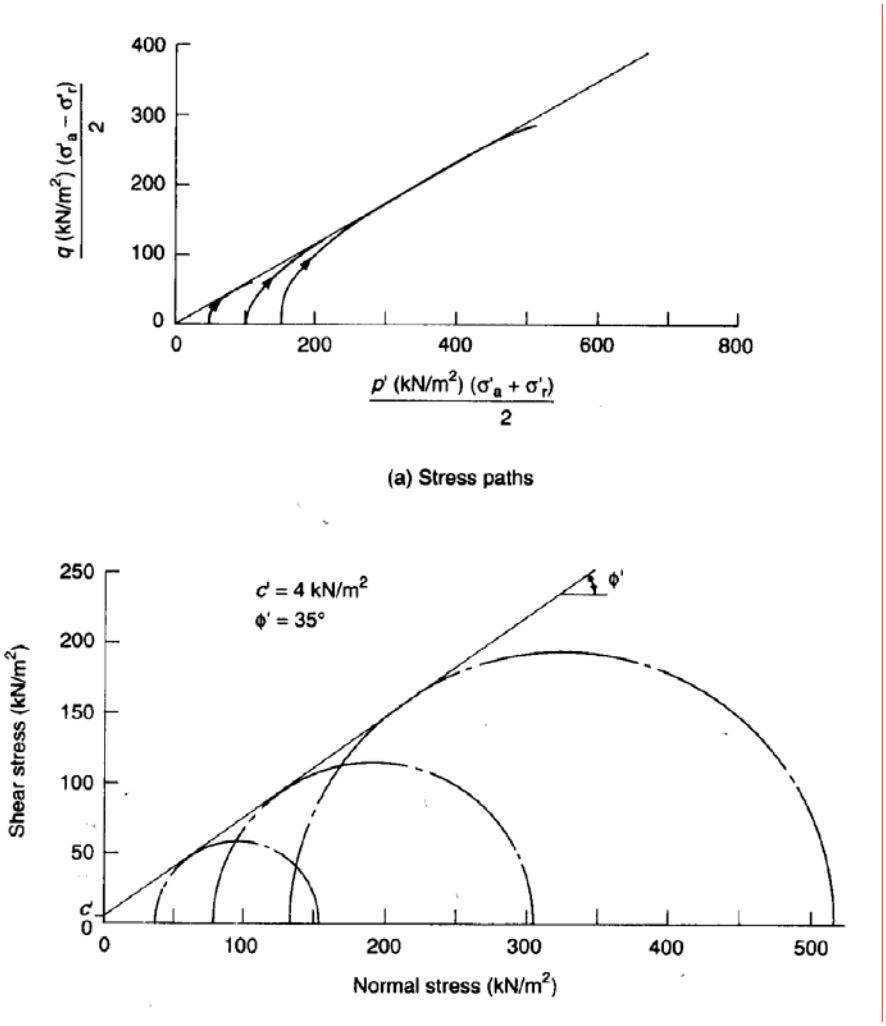


Fig. 8.13 Stress paths and Mohr circles at failure for a consolidated undrained triaxial compression test with pore pressure measurement.

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

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

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

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

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

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

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

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

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

Stiffness tests

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

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

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

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

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

conventional measurements of axial deformation of triaxial specimens, made outside the triaxial cell, introduce significant errors in the computation of strains.

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

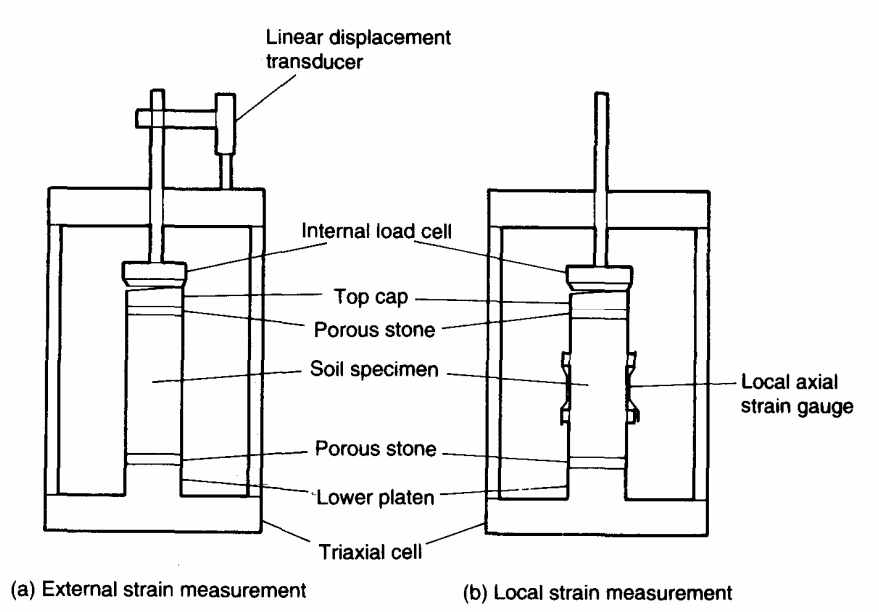


Fig. 8.14 Schematic diagram illustrating external and local strain measurement in the triaxial apparatus.

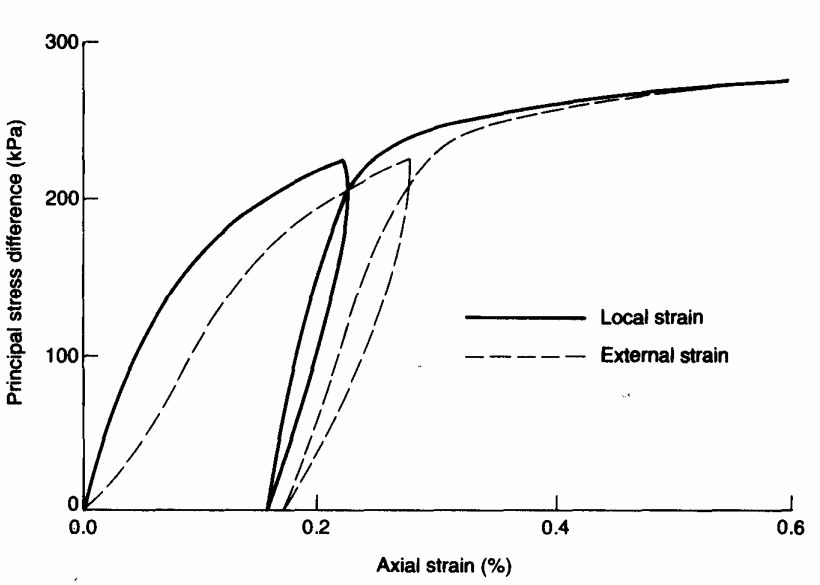


Fig. 8.15 Comparison of local and external strains (Clayton and Khatrush 1986).

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

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

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

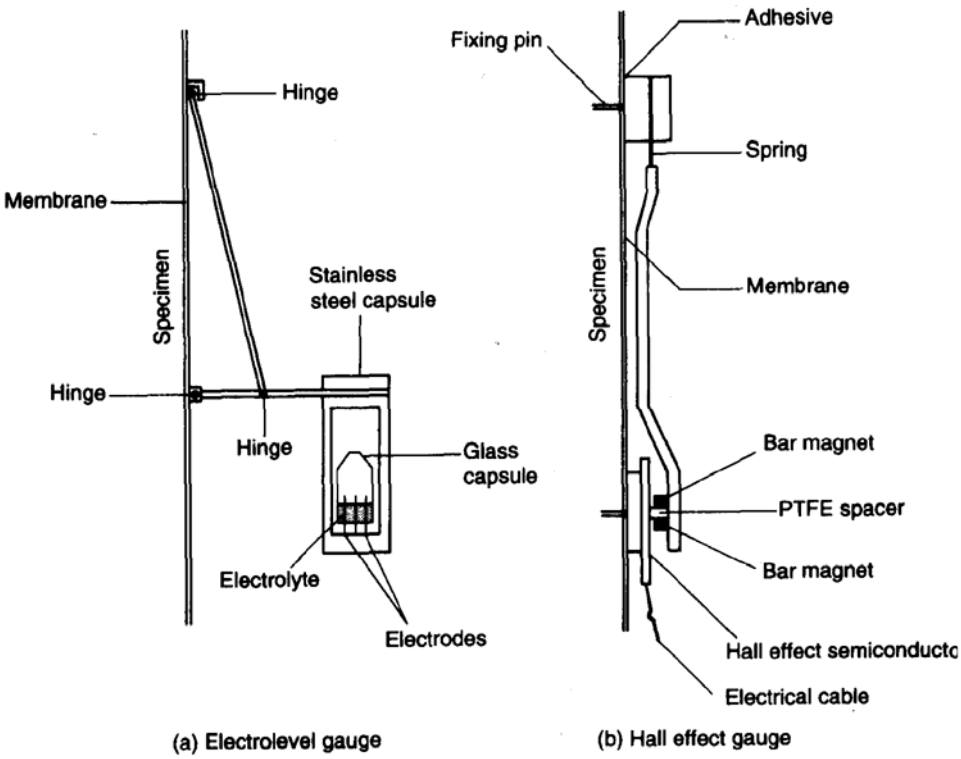


Fig. 8.16 Principle of operation and detail of electrolevel and Hall effect local strain gauges (Jardine *et al.* 1984, and Clayton and Khatrush, 1986).

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

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

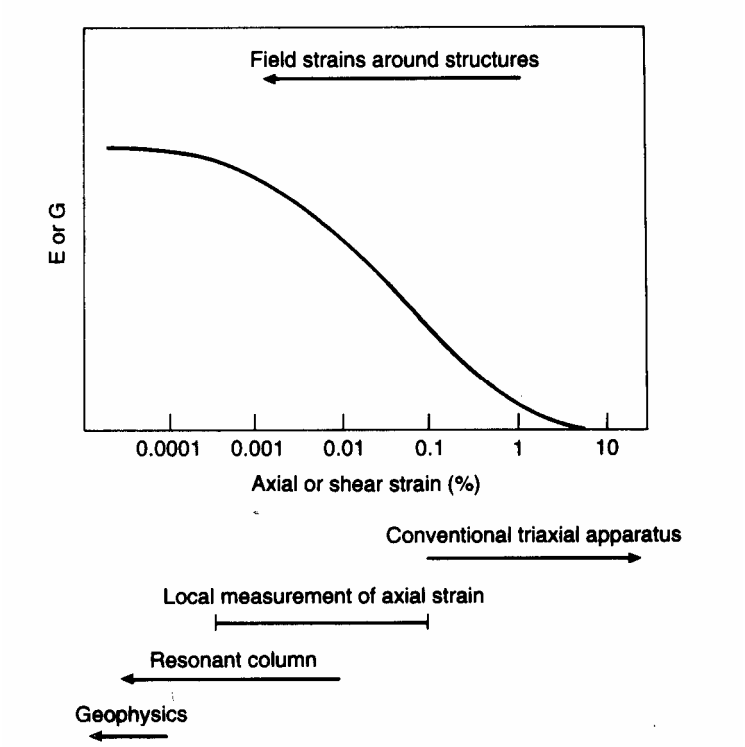


Fig. 8.17 Typical relationship between stiffness and strain for soils.

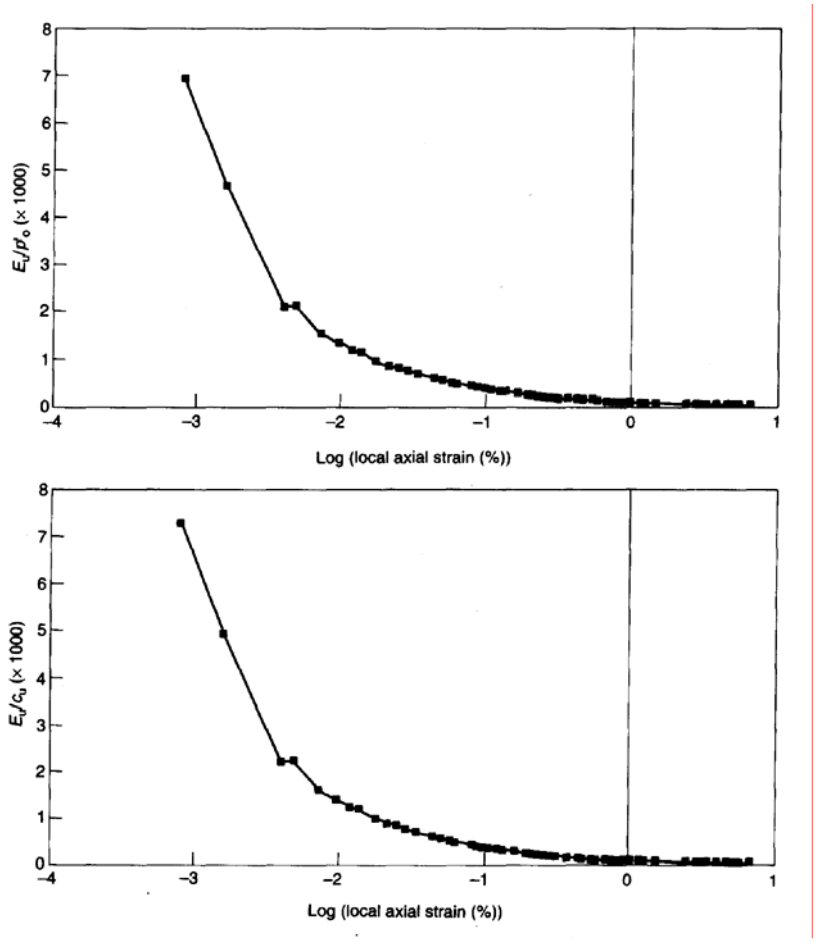


Fig. 8.18 Normalized stiffness—log strain curves.

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

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

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

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

CONSOLIDATION TESTS

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

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

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

Casagrande oedometer test

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

The most common specimen size is 76mm dia. x 19mm high, since this allows the highly disturbed

edges of a 102 mm dia. sample to be pared off during specimen preparation. Where the specimen preparation process may be prevented by the presence of stones, the specimen diameter must be equal to that of the sampler.

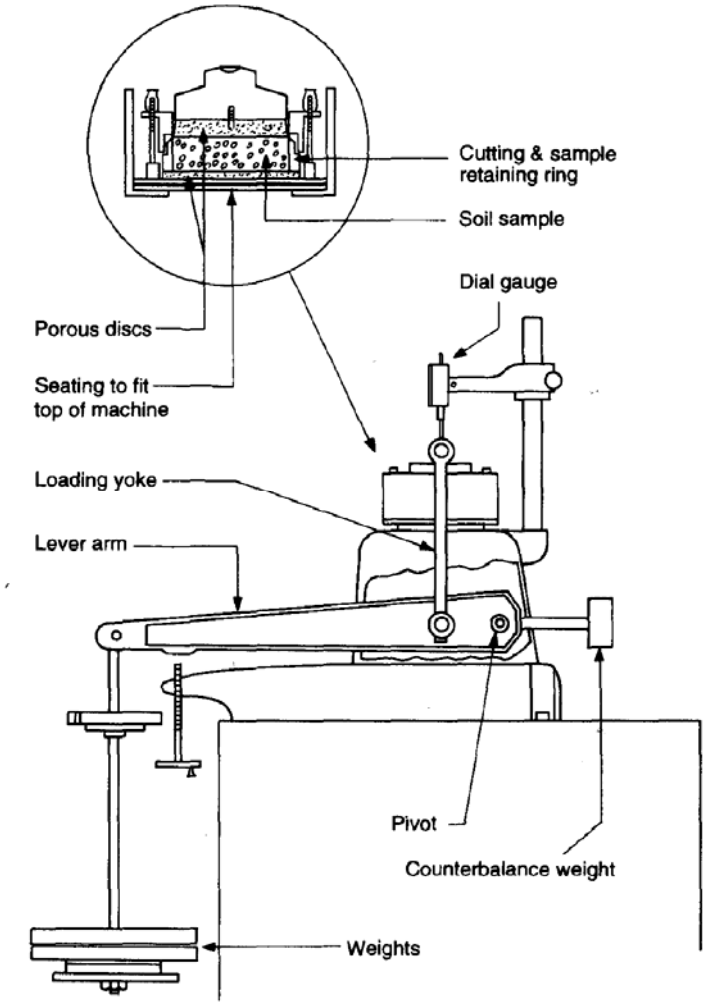


Fig. 8.19 Casagrande oedometer apparatus.

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

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

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

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

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

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

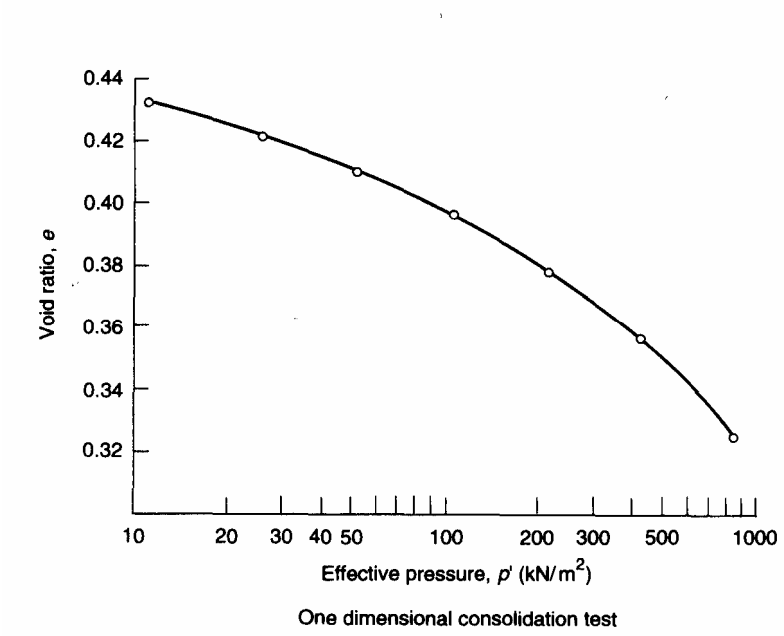


Fig. 8.20 Oedometer test result.

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

In hard clays and soft rocks the mass compressibility is much affected by the compressibility of the

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

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

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

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

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

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

Triaxial dissipation test

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

When testing compacted soils the initial stages of the test involve undrained application of cell pressure, to allow an assessment of the pore pressure parameter B. Most natural soils will be saturated

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

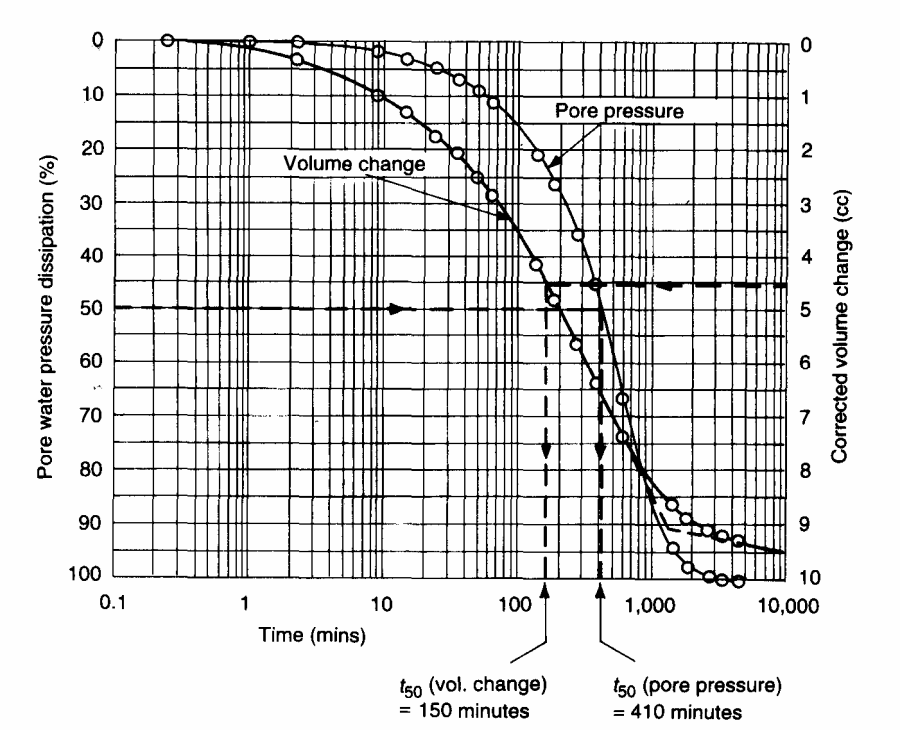


Fig. 8.21 Typical triaxial dissipation test result.

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

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

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

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

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

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

The triaxial dissipation test is time consuming (no filter drains can be used), but relatively

straightforward. Pore pressure measurement should be carried out by electrical pressure transducer and de-airing of the triaxial system should be thorough.

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

Hydraulic oedometer test

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

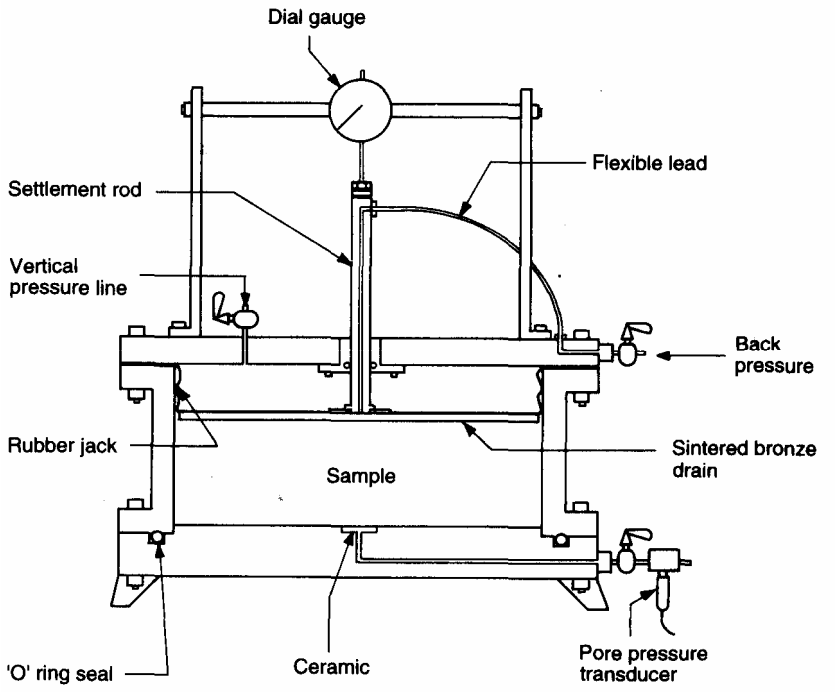


Fig. 8.22 The hydraulic oedometer (Rowe and Barden 1966).

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

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

The hydraulic oedometer cell can be used with at least four types of specimen drainage: drainage may

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

Permeability tests

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

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

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

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

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

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

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

Chemical tests

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

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

Table 8.7 Available chemical tests

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

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

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

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

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

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

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

Sulphate content, chloride content, and organic matter content in aggregate or materials intended for

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

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

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

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

ACCURACY AND MEASURING SYSTEMS

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

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

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

Chapter 9

In situ testing

INTRODUCTION

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

When seen in terms of such practical factors as the range of soil types that may be tested, the simplicity of a test and its cost — factors which are most likely to be considered as important by practising engineers, penetration tests (and particularly dynamic penetration tests) are much more

attractive than more sophisticated and analytically 'correct' tests, such as the self-boring pressuremeter. this then is the problem facing the engineer planning a site investigation: should cheap, rugged, simple tests be carried out, or should the risks associated with more sophisticated tests be taken, with the aim of obtaining more fundamental, and often more accurate, parameters? The answer must depend on the precision with which engineering calculations are to be made, and the types of soil to be expected, as detailed in Table 9.1.

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

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

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

PENETRATION TESTING

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

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

The Standard Penetration Test (SPT)

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

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

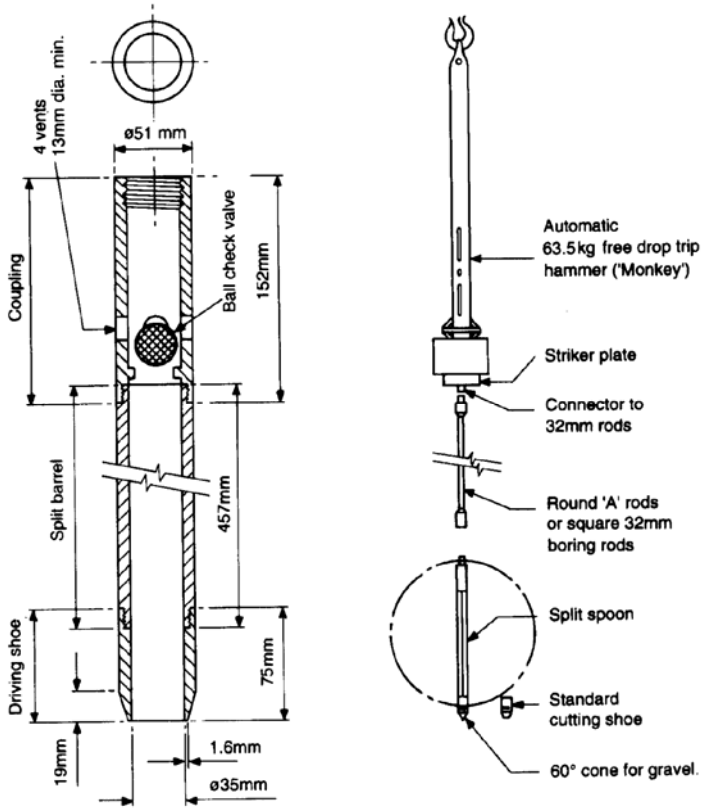


Fig. 9.1 Equipment for the standard penetration test.

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

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

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

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

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

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

Effects of test apparatus

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

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

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

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

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

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

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

Table 9.2 Stiffnesses and weights of various SPT rods

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

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

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

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

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

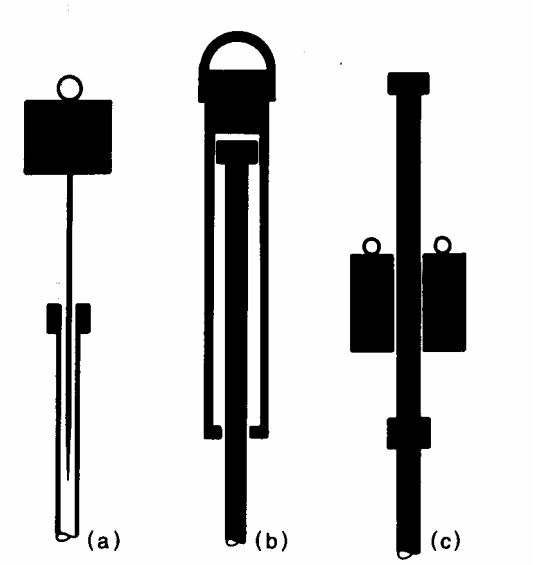


Fig. 9.2 Sections through American SPT slip-rope hammers: (a) pinweight hammer; (b) safety hammer; (c) donut hammer (after Riggs 1986).

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

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

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

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

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

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

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

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

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

Table 9.3 Measured SPT rod-energy ratios

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

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

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

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

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

From the figures in Table 9.2 it can be estimated that corrections of the order of +40 to —30% may

need to be applied to N values. This will certainly be necessary when the results are to be used in a precise way, for example in assessing the liquefaction potential of sands.

Effects of borehole disturbance

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

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

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

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

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

Interpretation and use

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

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

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

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

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

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

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

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

The cone penetration test (CPT)

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

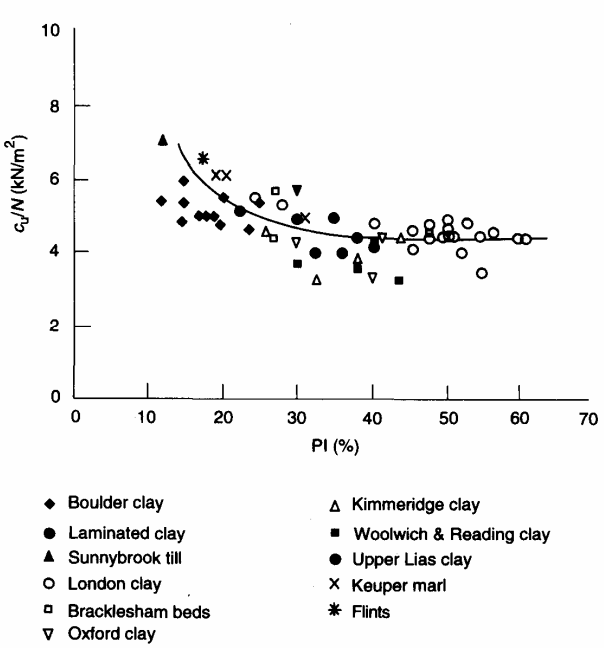


Fig. 9.3 Ratio of undrained shear strength (c_u) determined on 100mm diameter specimens to SPT N, as a function of plasticity (Stroud 1974).

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

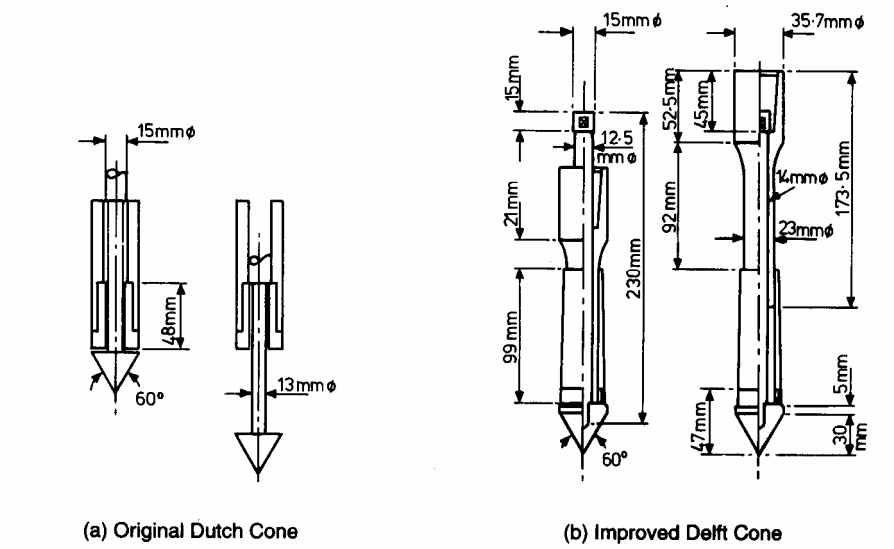


Fig. 9.4 Original dutch cone and improved mechanical Delft cone (Lousberg *et al.* 1974).

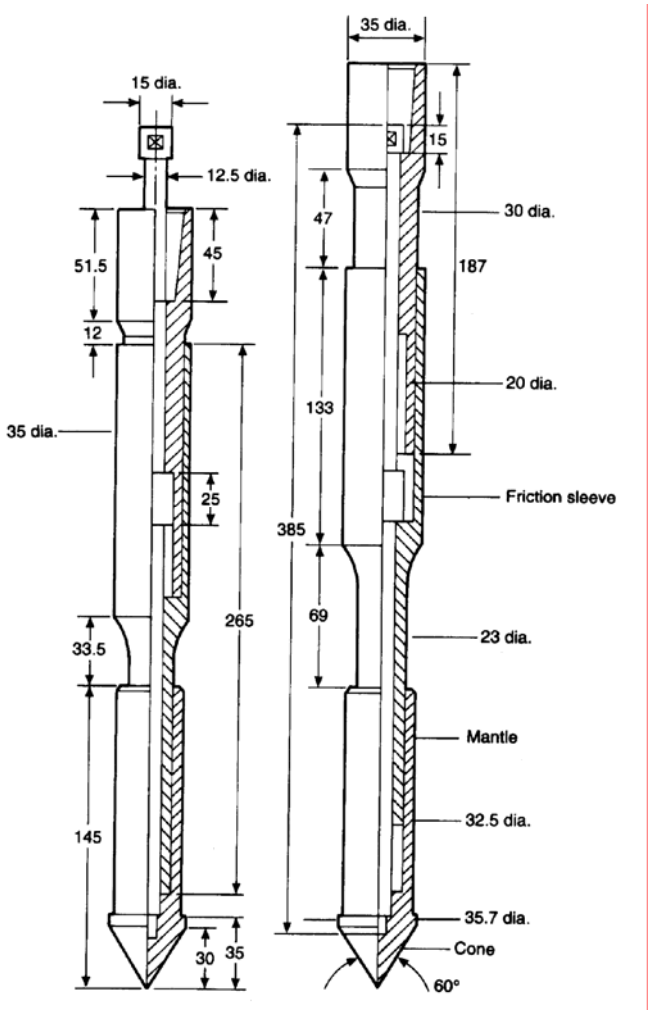


Fig. 9.5 Begemann mechanical friction cone (left, fully closed; right, fully extended) (Meigh 1987)

Mechanical cone testing

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

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

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

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

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

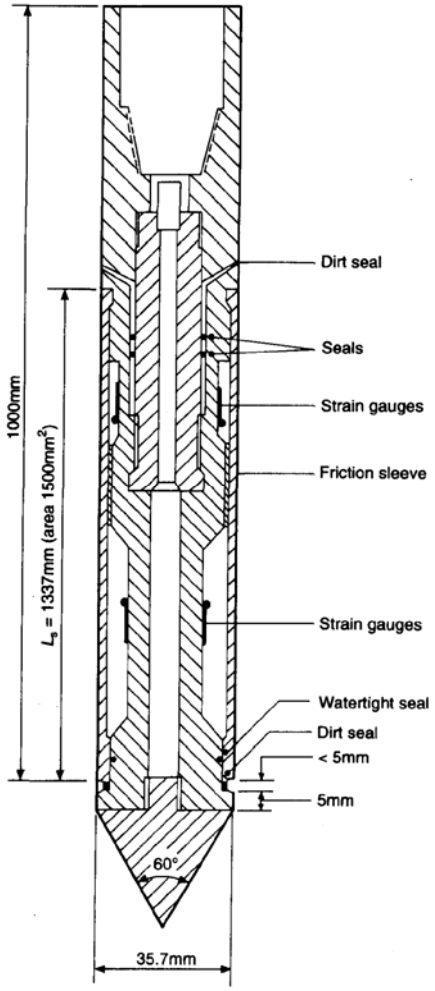


Fig. 9.6 Electric friction cone (largely after Meigh 1987).

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

At high cone resistances, loads as high as 10 tonne may be need to be applied to the cone. At 30m depth the compression of the inner rods may be of the same order as the 8cm stroke used in a Delft cone and, although the top of the inner rods is pushed downwards by the correct amount, the cone will not then advance ahead of the outer rods. This effect will obviously be more serious when a Begemann

cone is in use, because the available stroke is only 4cm. In addition, in deep soft soils, corrections should be made to mechanical cone data to compensate for the mass of the rods.

Electric cone testing

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

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

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

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

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

The piezocone

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

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

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

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

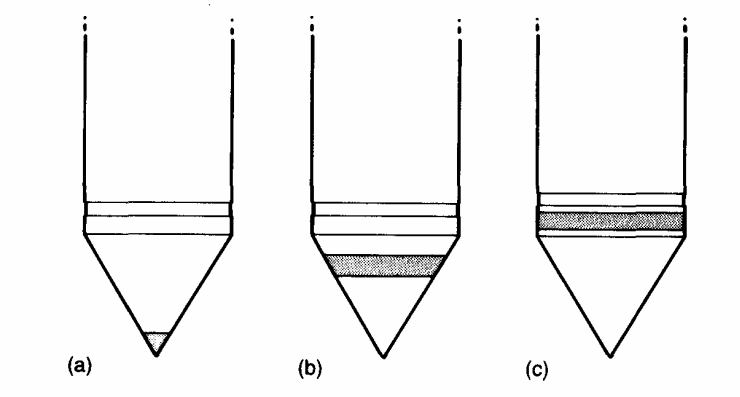


Fig. 9.7 Positions of porous tips on piezocones.

The seismic cone

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

Standards and reference test procedure

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

Interpretation and use

The basic measurements made by a cone are:

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

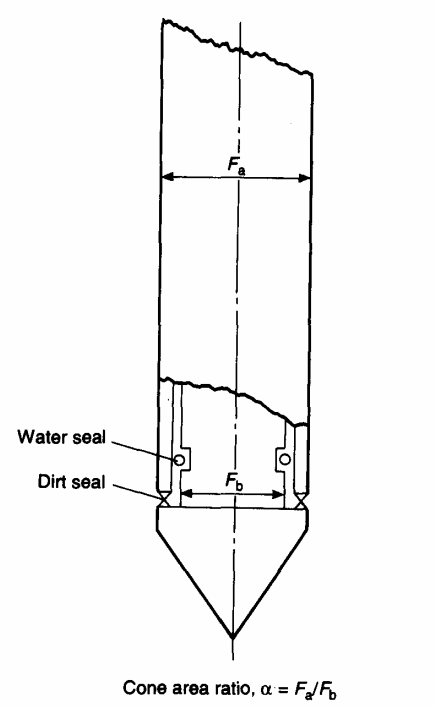


Fig. 9.8 Definition of cone area ratio, α.

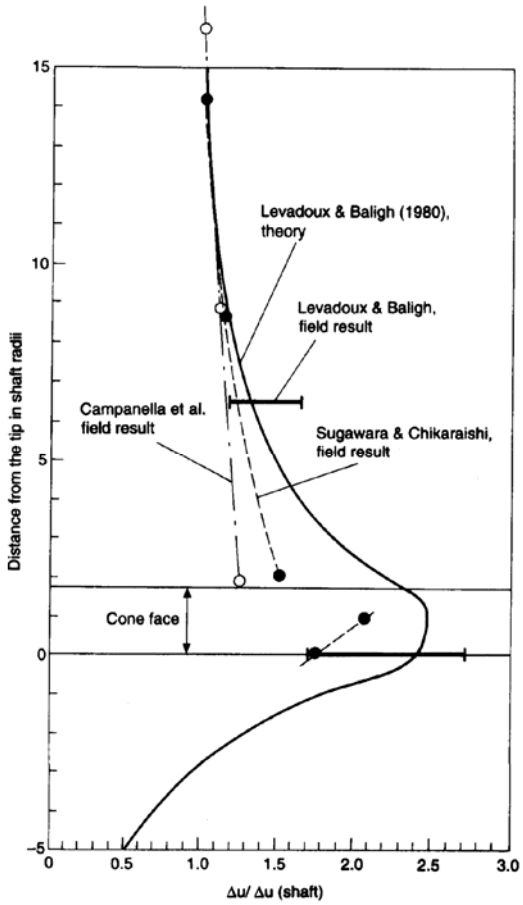


Fig. 9.9 Distribution of excess pore pressure over the cone (Coutts 1986).

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

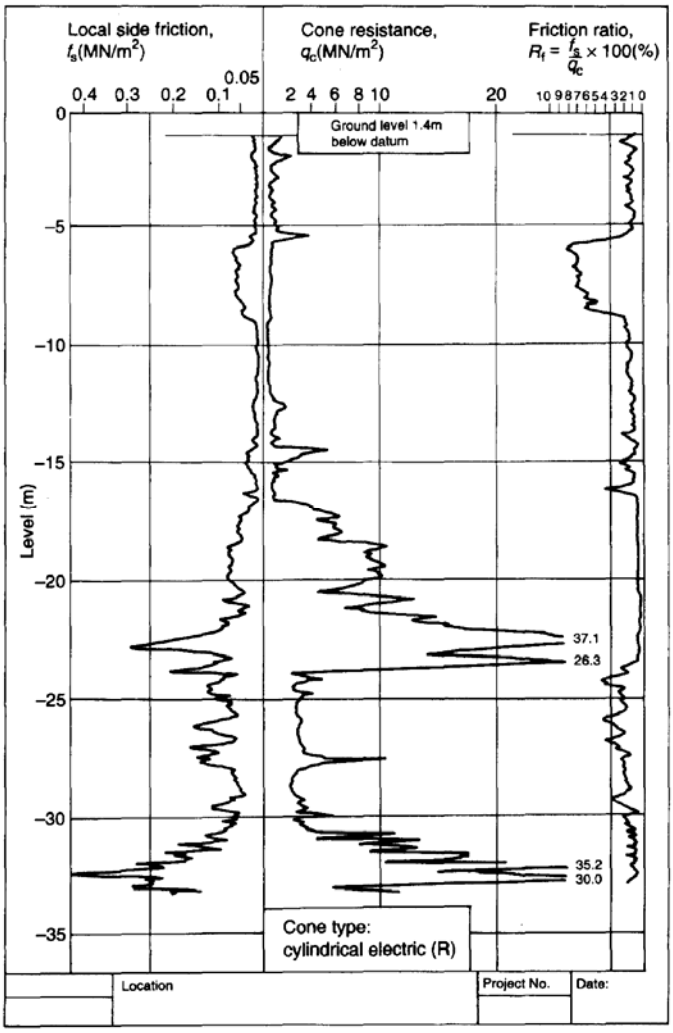


Fig. 9.10 Typical record of a friction cone penetration test (te Kamp, 1977, from Meigh, 1987).

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

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

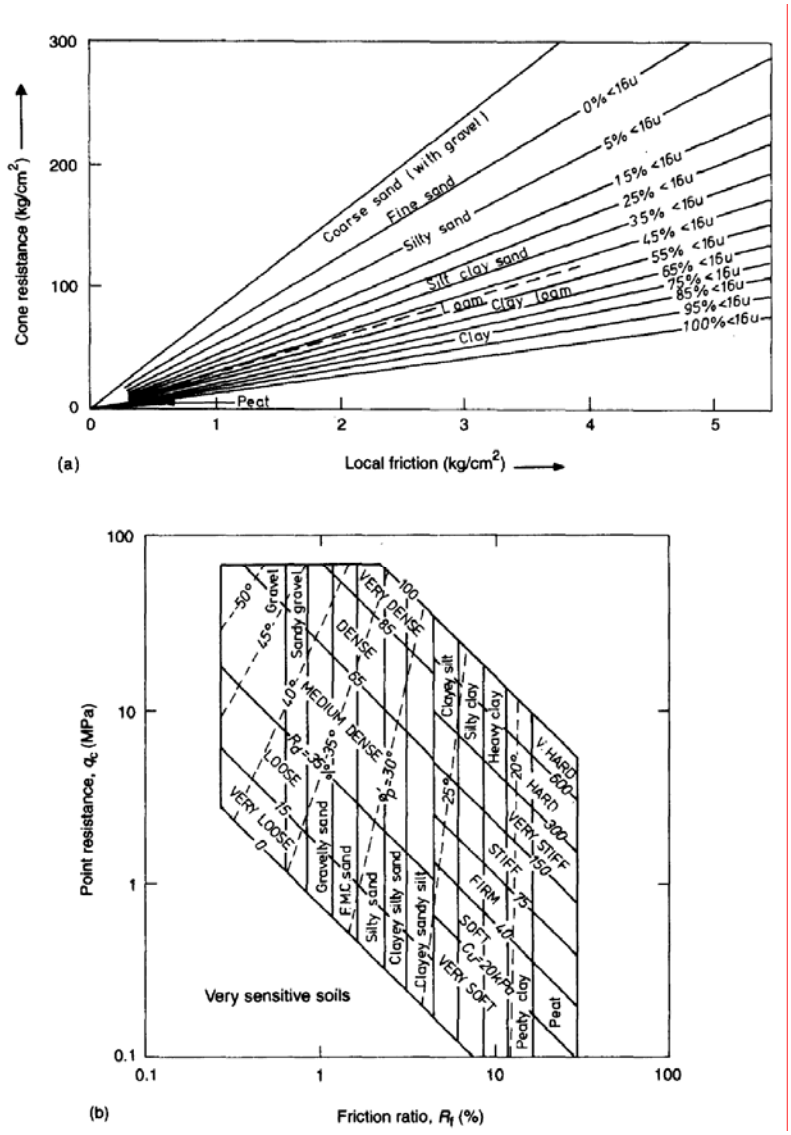


Fig. 9.11 (a) relationship between soil type, cone resistance and local friction (Begemann 1956); (b) soil identification chart for a mechanical friction cone (Searle 1979).

Table 9.5 Diagnostic features of soil type

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

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

$$M = \alpha_M q_c \tag{9.12}$$

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

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

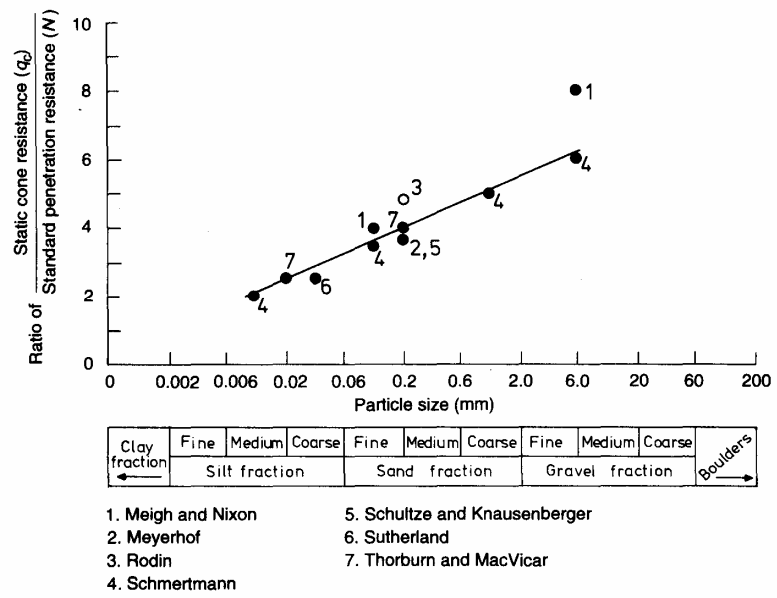


Fig. 9.12 Ratio of (CPT q_c) (SPT N) as a function of D50 particle size of the soil (Thorburn, 1971).

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

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

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

so that the undrained shear strength may be calculated from:

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

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

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

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

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

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

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

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

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

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

Probing

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

STRENGTH AND COMPRESSIBILITY TESTING

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

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

self-boring form) used in the UK for special projects in overconsolidated clays, to determine undrained strength, shear modulus, and coefficient of earth pressure at rest, K_0.

3. *The plate loading test.* This is used primarily to obtain the stiffness of granular soils and fractured weak rocks.
4. *The Marchetti dilatometer.* This is not yet used commercially in the UK but, at the time of writing, is becoming more widely used in other parts of the world.

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

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

The field vane test

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

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

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

Standard testing

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

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

Apparatus

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

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

Table 9.6 USA specifications for vane blades

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

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

Table 9.7 UK specifications for vane blades

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

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

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

Test procedure

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

Once the vane has been pushed into the ground, it is rotated at a slow rate, preferably using a purpose-built test apparatus with an inbuilt geared drive (Fig. 9.13). Torsional force is measured, and is then

converted to unit shearing resistance by assuming the geometry of the shear surface, and the shear stress distribution across it.

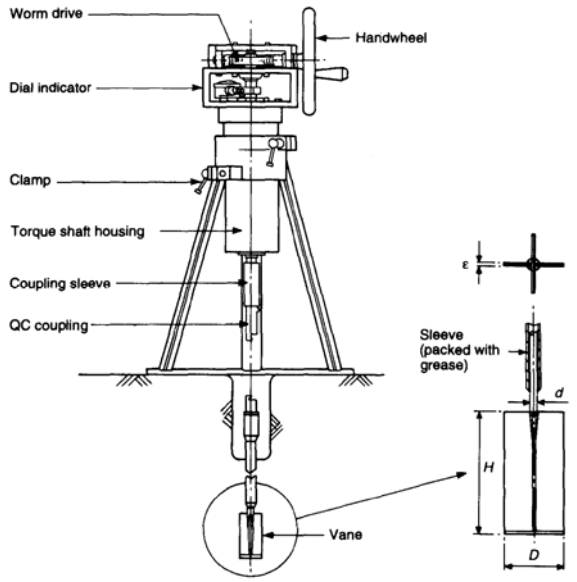


Fig. 9.13 Farnell model 274 field vane apparatus.

The test procedure is as follows.

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

Interpretation

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

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

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

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

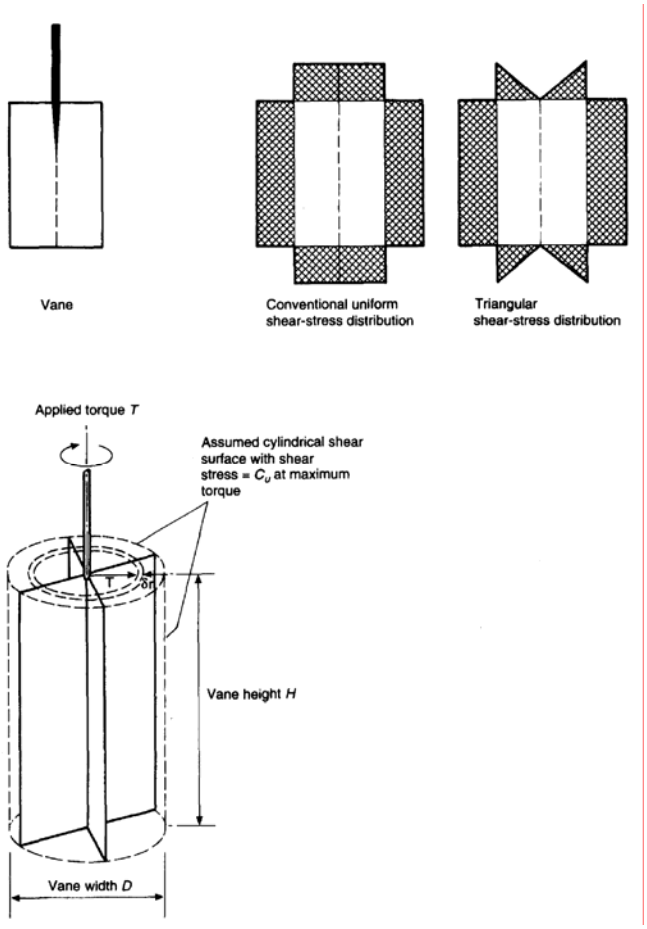


Fig. 9.14 Assumed geometry of shear surface for conventional interpretation of the vane test.

For a vane blade where H = 2D:

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

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

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

For a vane blade where H = 2D:

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

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

Discussion

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

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

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

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

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

shear strength along a cylindrical surface. Hence, the occurrence of progressive failure may influence the final strength value.

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

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

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

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

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

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

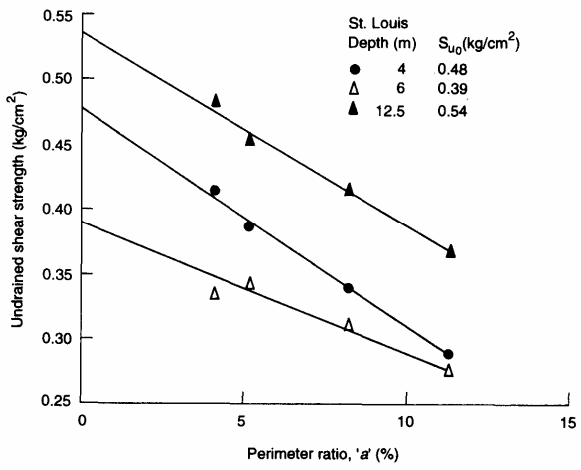


Fig. 9.15 Effect of perimeter ratio on undrained shear strength of Champlain clay (La Rochelle *et al.* 1973).

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

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

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

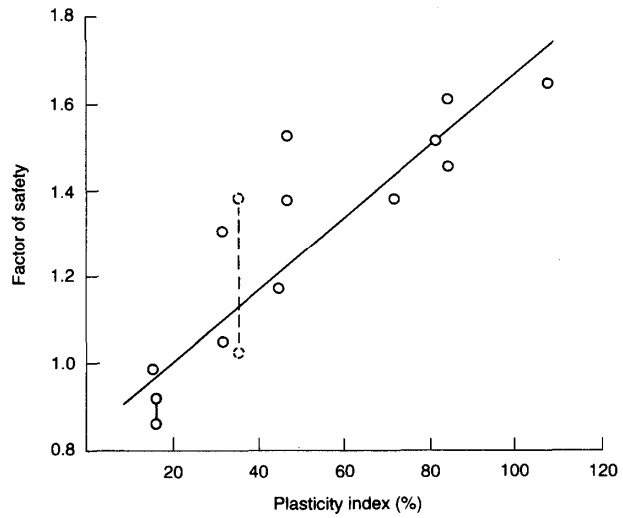


Fig. 9.16 Calculated factors of safety for failed embankments, based on (Bjerrum 1972).

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

Further developments

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

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

(cylindrical) and horizontal planes. Aas (1965, 1967) developed a method for assessing the degree of anisotropy of a soil by using rectangular vanes of differing heights.

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

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

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

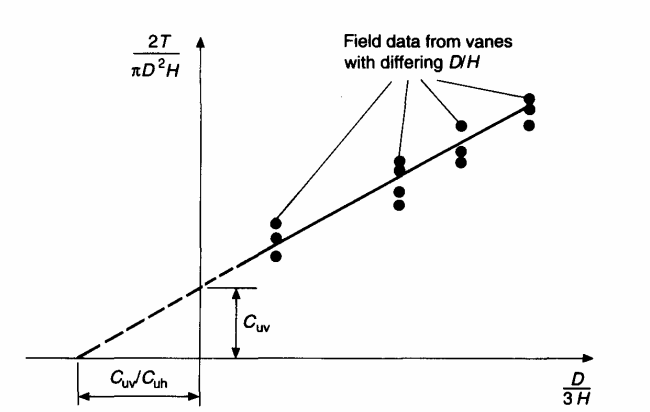


Fig. 9.17 Method of determining undrained strength anisotropy (Aas 1965, 1967).

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

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

whilst for a triangular distribution:

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

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

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

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

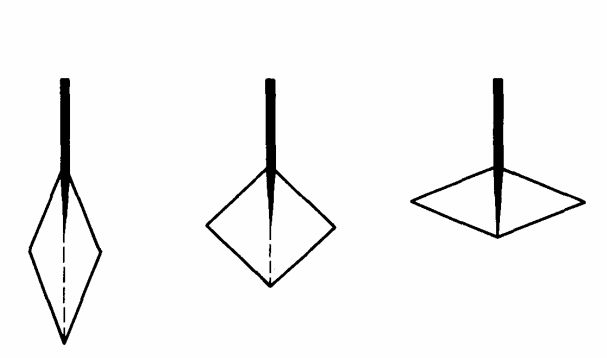


Fig. 9.18 Diamond shear vanes.

Pressuremeter testing

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

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

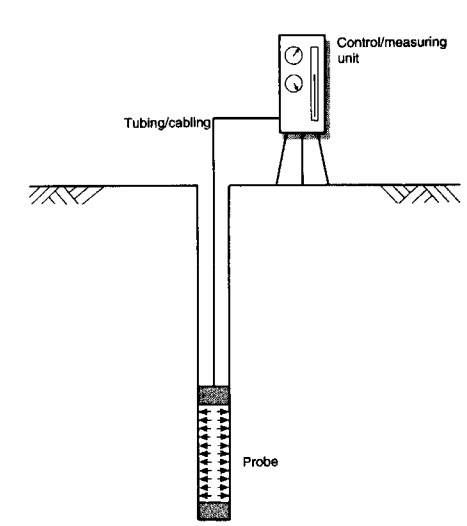


Fig. 9.19 Basic components of the pressuremeter.

Types of pressuremeter

Three principal types of pressuremeter are in use.

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

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

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

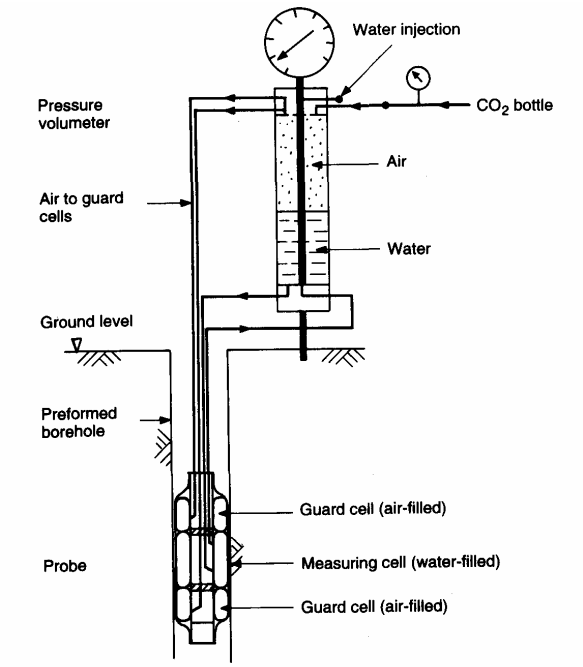


Fig. 9.20 Diagrammatic sketch of the Ménard pressuremeter (Gibson and Anderson 1961).

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

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

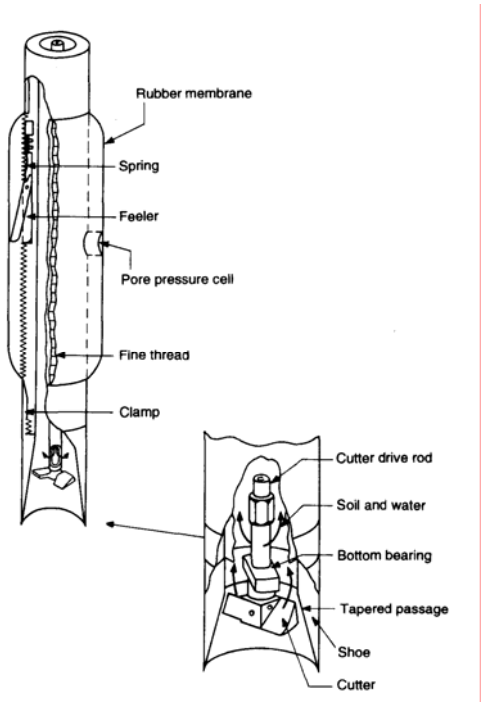


Fig. 9.21 The Cambridge self-boring pressuremeter (after Windle and Wroth 1977).

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

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

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

Factors affecting the amount of disturbance caused by insertion are:

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

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

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

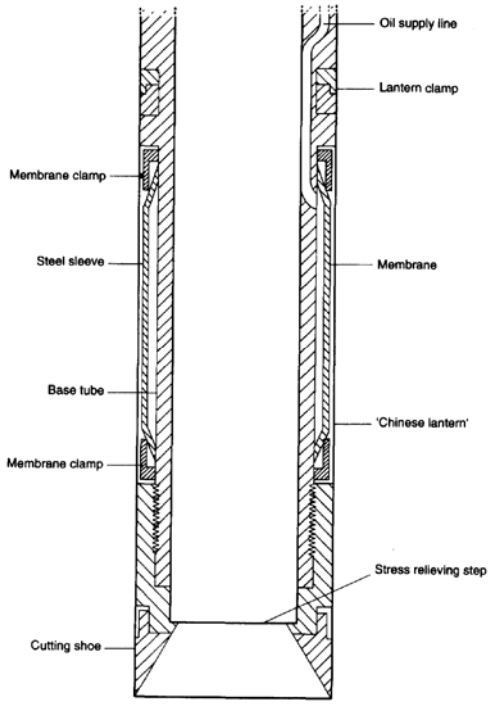


Fig. 9.22 The push-in pressuremeter.

Test methods — borehole pressuremeters

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

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

which records volume change, in the pressure-volumeter. Pressure is provided by means of a CO2 bottle.

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

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

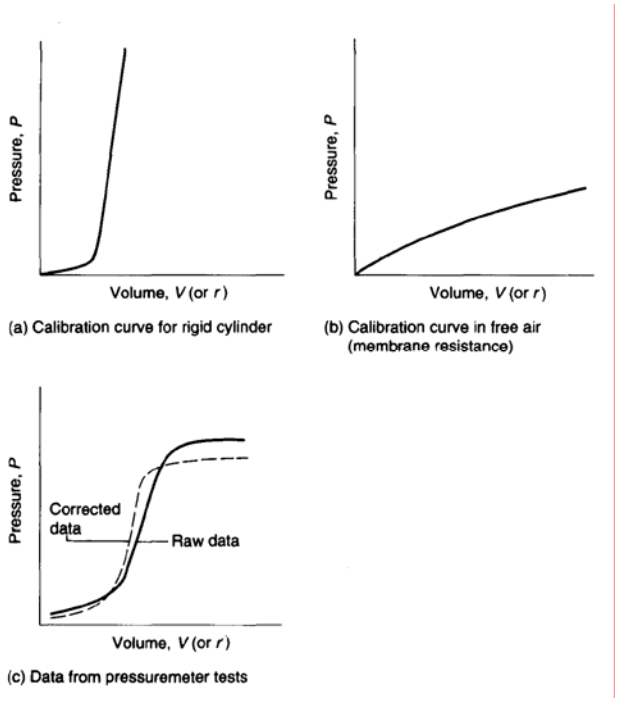


Fig. 9.23 Pressuremeter calibration plots and data correction (Mair and Wood 1987).

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

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

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

The pressuremeter curve can be divided into three phases:

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

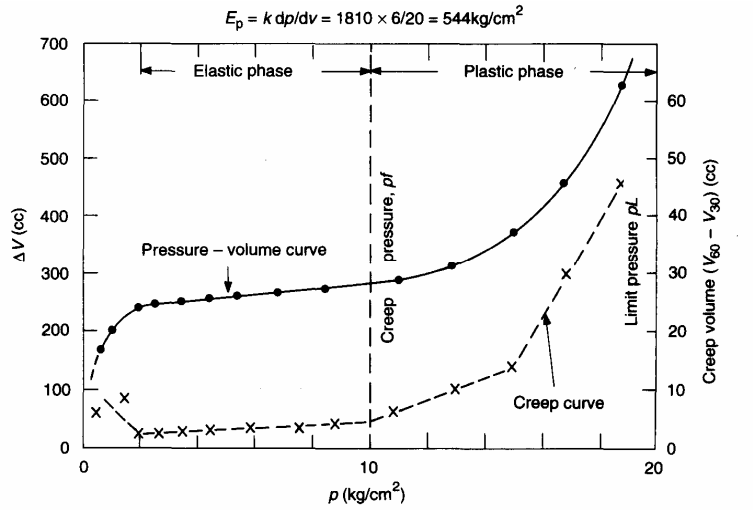


Fig. 9.24 Pressuremeter test curve on mudstone (Meigh and Greenland 1965).

Test method — Cambridge self-boring pressuremeter

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

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

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

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

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

Results and interpretation

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

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

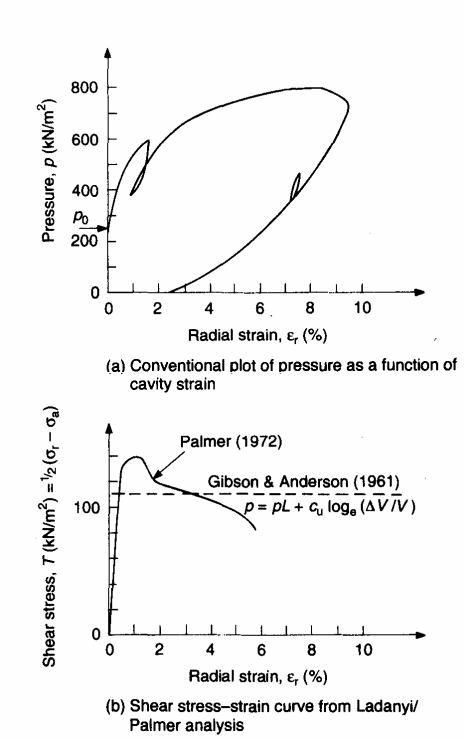


Fig. 9.25 Results from a Cambridge self-boring pressuremeter test in Gault clay (Windle and Wroth 1977).

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

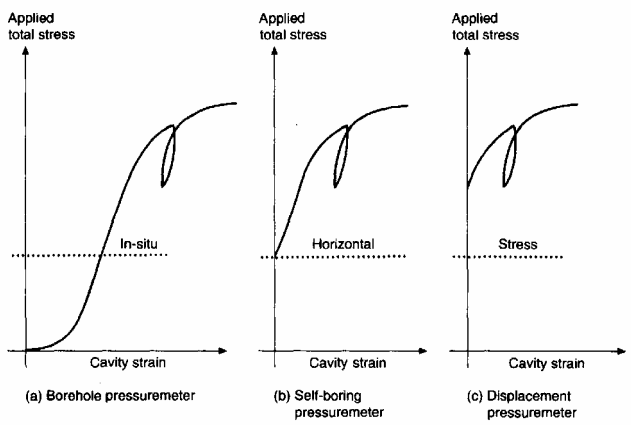


Fig. 9.26 Idealized results from three types of pressuremeter.

Semi-empirical approach

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

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

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

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

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

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

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

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

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

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

From elastic analysis:

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

and therefore

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

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

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

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

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

Analytical approach

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

CLAYS

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

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

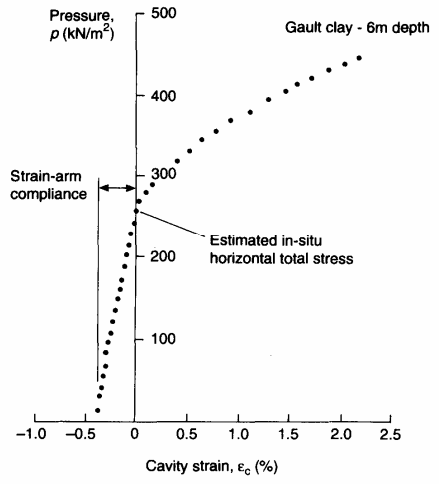


Fig. 9.27 Illustration of lift-off method for determining in-situ horizontal stress (from Dalton and Hawkins 1982).

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

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

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

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

This simplification should not be made at larger cavity strains.

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

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

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

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

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

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

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

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

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

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

The limit pressure can therefore be estimated by plotting the corrected data for the last part of the test as change in volume over total volume, $\Delta V/V$ as a function of corrected cell pressure, p. When plotted

in this way the data should form an approximately straight line—Fig. 9.28. By extrapolating to $\Delta V/V(=V_m/(V_0 + V_m))= 1$, and in the case of a self-boring pressuremeter, by first converting cavity strain through the expression:

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

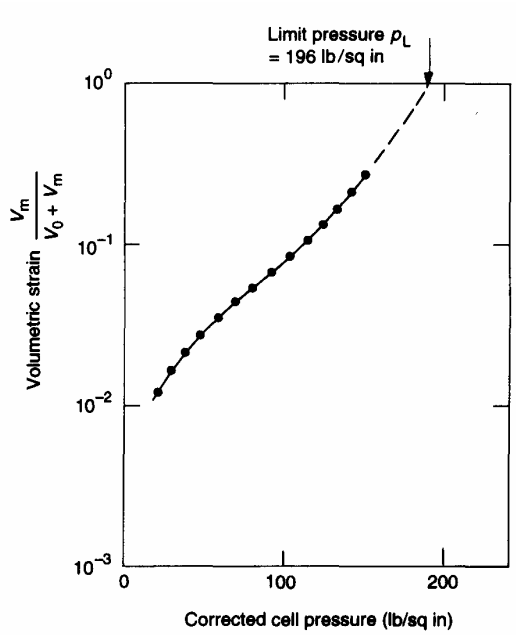


Fig. 9.28 Method of extrapolating to limit pressure (Gibson and Anderson 1961).

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

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

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

ROCKS

In rocks, the self-boring pressuremeter has recently been developed to improve penetration (by using a full-face drilling bit), and to give greater sensitivity to cavity strain (by incorporating Hall effect sensors) (Clarke and Allan 1989). This is still a relatively untried tool, however. In contrast, the borehole pressuremeter has long been used to obtain stiffness values in weak rocks and saprolitic soils. Meigh and Greenland (1965) showed good agreement between ultimate bearing pressure obtained from small-diameter plate tests and limit pressure values, with reasonable agreement between modulus values derived from plates and from the pressuremeter. However, the small size of the pressuremeter tests, and the dominant effects of fracture stiffness and orientation on rock mass stiffness suggest that this may often not be the case. Recently, Haberfield and Johnston (1993) have argued that the only parameter that can be determined with any reliability in weak rock is the shear modulus, and that for

this to be reliable joint spacing must be less than about 20mm (for a typical pressure- meter of approximately 75 to l00mmdia.). With current technology, they argue that it is doubtful that an accurate estimate of in situ stress can be obtained, and that strength properties are best measured by other methods.

Plate loading tests

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

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

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

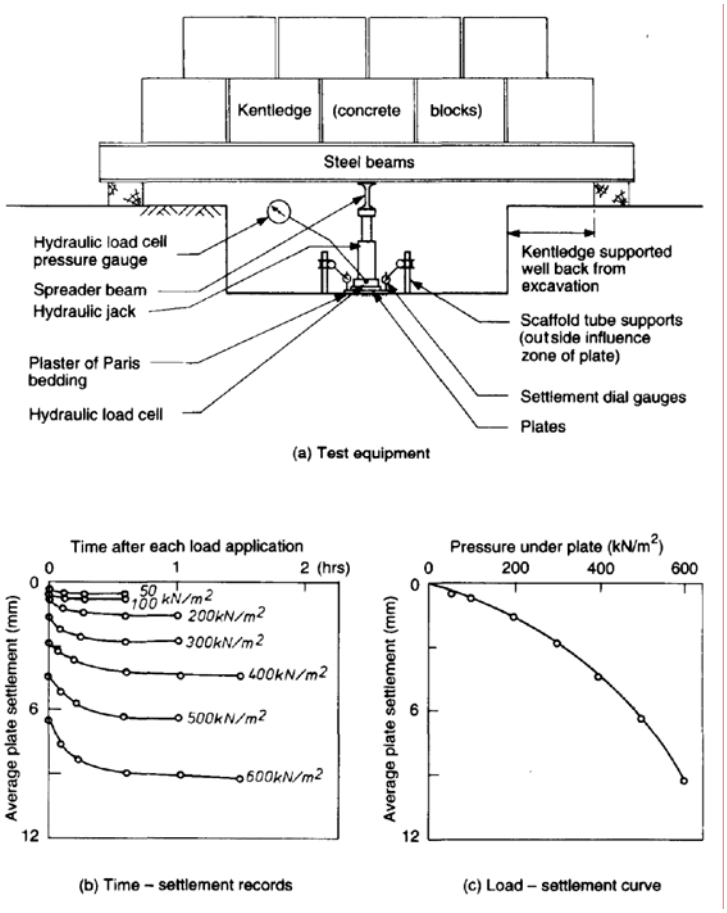


Fig. 9.29 Plate loading test layout and result.

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

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

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

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

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

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

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

Terzaghi and Peck proposed:

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

where ρ_B = settlement of a footing of width B, ρ_1 = settlement of a 1 ft plate, which leads to a maximum settlement ratio of 4, however big the footing. Bjerrum and Eggestad (1963) investigated Terzaghi and Peck's relationship for settlement ratio, and found a considerable scatter, (Fig. 9.30). It can be seen that Terzaghi and Peck's relationship is close to Bjerrum and Eggestad's lower extreme', while Bjerrum and Eggestad's upper bound approximates to $(\rho_B / \rho_1) = B$ for B <l0ft (3m) and gives a settlement ratio of 30 for B 100 ft (30 m). Bjerrum and Eggestad found no differences in correlation for footings on dense, medium or loose sand, but considered that the upper extreme represented very loose, slightly organic sand while foundations on dense sand would give points between the average and lower extremes. This evidence is not supported by more recently published work which gives even higher settlement ratios. D'Appolonia *et al.* (1970) give values of settlement ratio for dense sands which fall above the extreme of Bjerrum and Eggestad, and Sutherland (1975) gives values taken from Levy and Morton (1975) which are even higher.

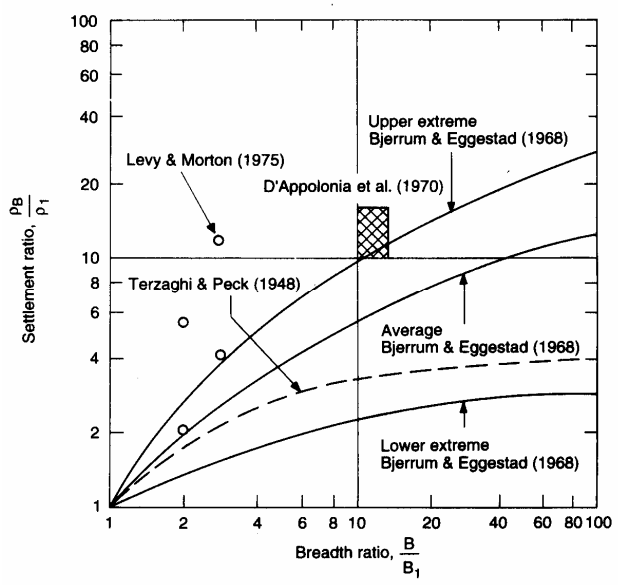


Fig. 9.30 Correlation between plate bearing tests and settlement of foundations (Sutherland 1975).

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

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

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

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

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

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

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

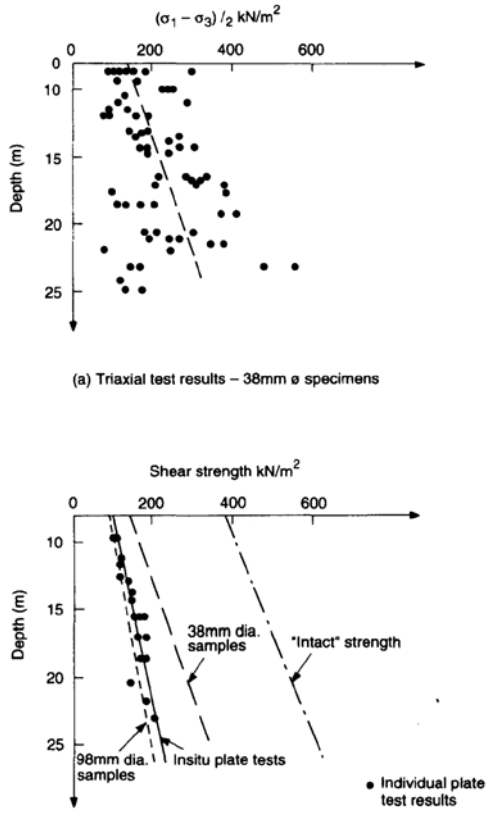


Fig. 9.31 Comparison of plate tests and laboratory test results in London clay (Marsland 1972).

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

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

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

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

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

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

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

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

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

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

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

Smaller plate-loading tests are routinely carried out down-hole in some countries (for example, South Africa) as part of investigations which rely upon visual description. This combination has proved particularly valuable above the water table in hard, gravelly or unsaturated and saprolitic soils, all of which can be very difficult to sample and test. The plate test is carried out across a large-diameter hole

which is formed using a auger-piling rig. An engineer or geologist is lowered down the hole to describe the ground and produce a borehole record. Test depths are then selected, and diagonally opposed faces are hand trimmed to provide flat areas upon which the small-diameter (100, 200 or 300 mm) plate test will bear. Details can be found in Wrench (1984).

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

The Marchetti dilatometer (DMT)

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

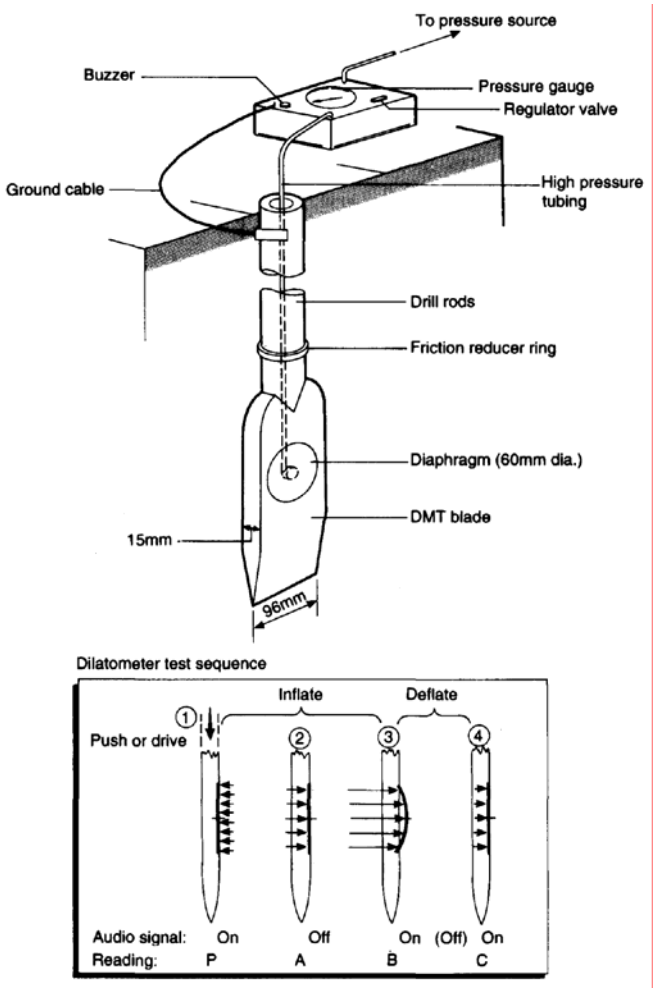


Fig. 9.32 Marchetti dilatometer.

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

Equipment

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

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

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

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

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

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

Test method

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

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

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

The blade is pushed into the soil at between 10mm/s and 30mm/s. Penetration resistance is measured

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

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

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

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

Reduction of test data

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

$$p_0 = 1.05\ (A - z_m + \Delta A) - 0.05\ (B - z_m - \Delta B) \tag{9.46}$$

$$p_1 = B - z_m - \Delta B \tag{9.47}$$

$$p_2 = 1.05\ (C - z_m + \Delta A) - 0.05\ (B - z_m - \Delta B) \tag{9.48}$$

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

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

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

$$q_D = \frac{P_D}{A_D} \tag{9.49}$$

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

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

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

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

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

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

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

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

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

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

Results, interpretation and use

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

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

SOIL PROFILING AND IDENTIFICATION

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

DETERMINATION OF SOIL PARAMETERS

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

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

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

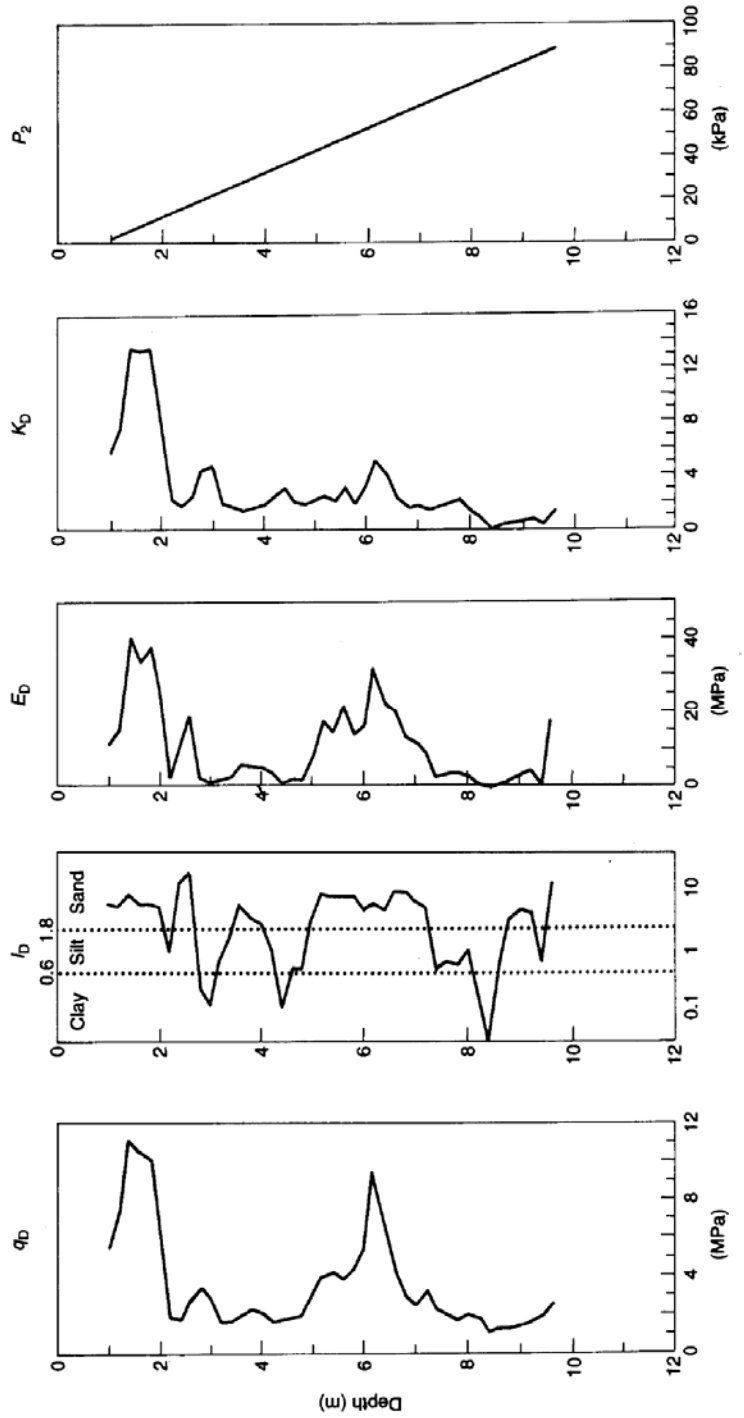


Fig. 9.33 Result of DMT sounding in clayey silts and silty sands (data from Schmertmann 1986).

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

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

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

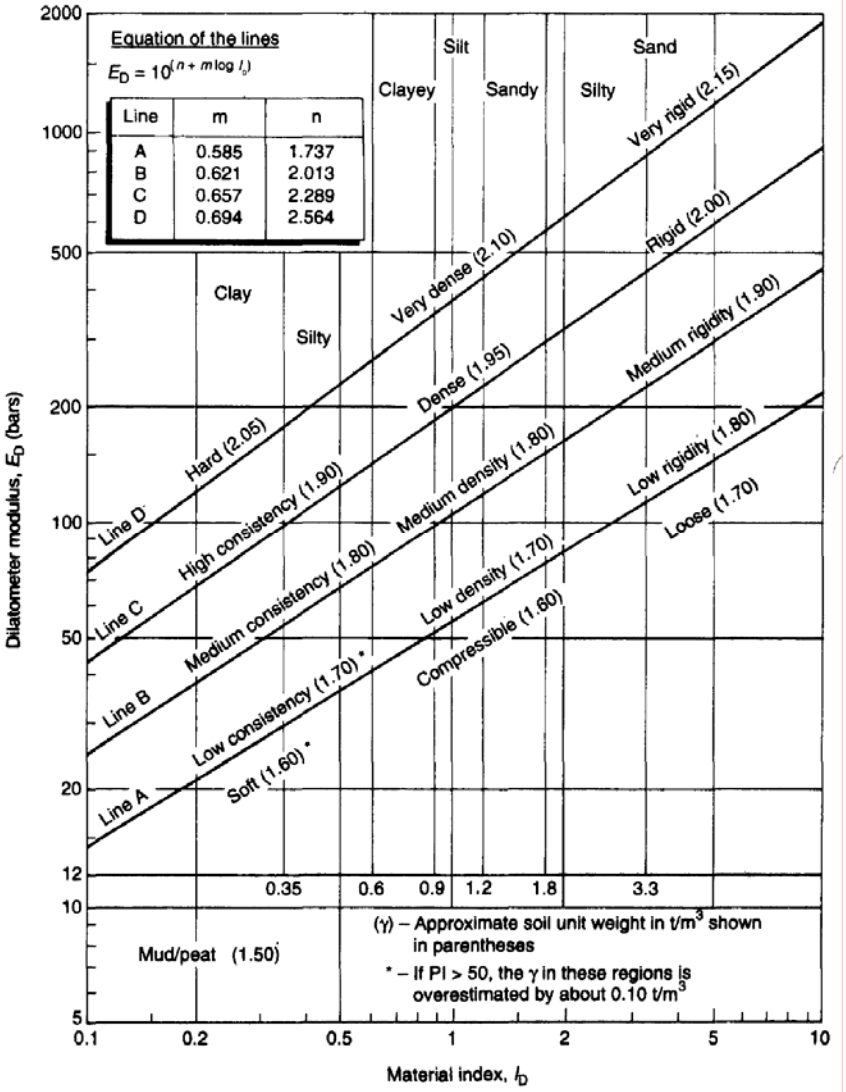


Fig. 9.34 Chart for determination of soil description and unit weight (from Marchetti and Crapps 1981).

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

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

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

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

ESTIMATION OF IN-SITU PORE PRESSURE AND HORIZONTAL STRESS

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

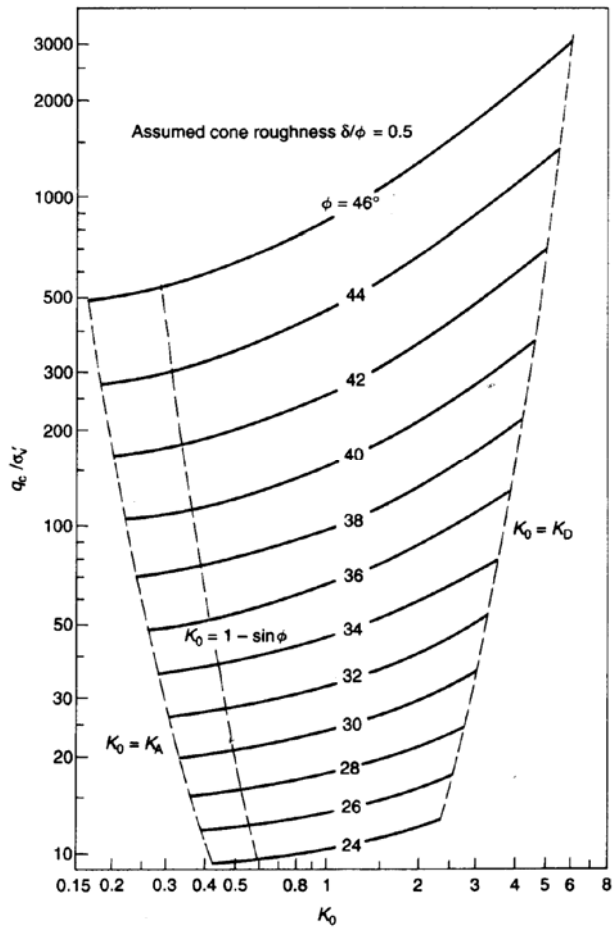


Fig. 9.35 Chart for determining effective angle of friction from CPT q_c and K_0 (from DMT data) (Marchetti 1985).

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

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

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

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

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

DESIGN

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

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

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

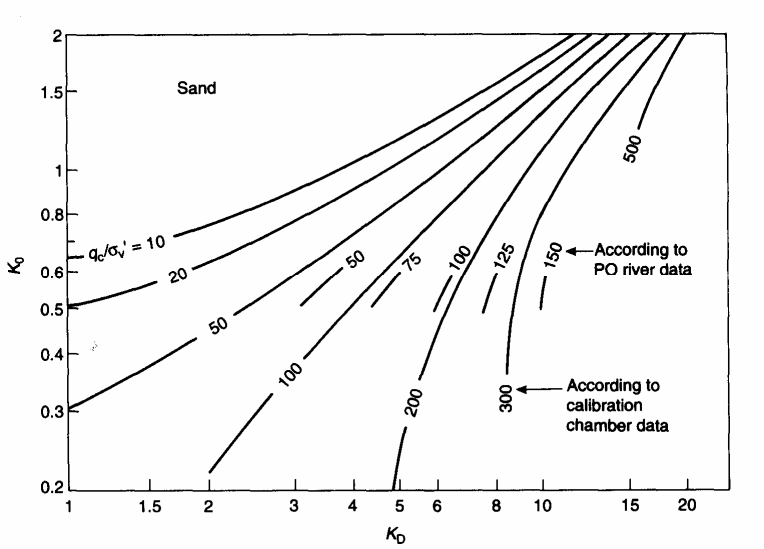


Fig. 9.36 Correlation between K_0 and DMT K_D for sand (Marchetti 1985).

Table 9.9 Design using the Marchetti dilatometer (DMT)

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

PERMEABILITY TESTING

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

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

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

Rising or falling head tests

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

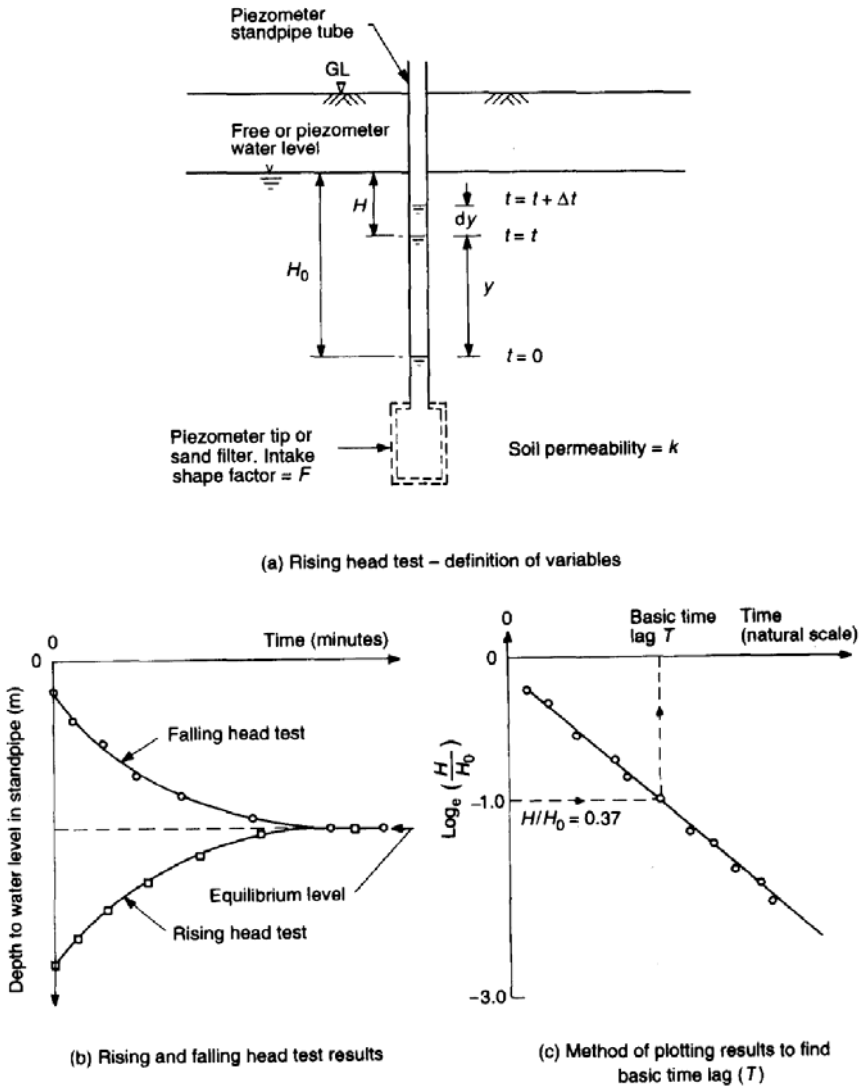


Fig. 9.37 Rising or falling head tests.

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

Assumptions

Soil does not swell or consolidate. Other test errors, such as those due to air in the soil or pipes, do not

occur. There is no smear. At time t, the driving head = H. Therefore, from Darcy's law the rate of flow into the piezometer is given by:

$$q = FkH = Fk(H_0 — y) \tag{9.57}$$

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

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

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

therefore, combining with eqn. 9.57:

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

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

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

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

therefore:

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

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

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

When the time equals the basic lag, then:

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

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

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

Where in situ tests are carried out, but the groundwater or piezometric level cannot be determined it

may be found by inserting trial values of H_0 in the above equations, and repeatedly plotting the graph of loge (H/H_0) vs. time. When the correct value of H_0 is inserted, a straight line will result: incorrect values yield curves.

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

Piezometer type Coefficient of permeability(cm/s)	Soil type						
	Sand			Silt		Clay	
	10^{-2}	10^{-3}	10^{-4}	10^{-5}	10^{-6}	10^{-7}	10^{-8} k
204 mm dia. borehole (flat bottomed)	0.37	3.69	36.90	369.00			
Casagrande piezometer l50mmdia. X 9l4mm long with 10mm bore standpipe			0.02	0.22	2.20	22.00	220.00
Closed hydraulic piezometer with 100 m of tubing				0.03	0.25	2.50	25.00

Once H_0 is known, the shape factor must be calculated to allow the coefficient of permeability to be determined from the basic time lag. Hvorslev (1951) gives shape factors for a variety of geometries, but during most site investigations only a few cases are normally used (Table 9.11).

Table 9.11 Shape factors (from Hvorslev (1951))

Geometry	F
Cased borehole (diameter, D) soil flush with bottom of casing, in uniform soil	2.75D
Cased borehole, soil flush with bottom of casing. Soil above base of hole impermeable	2D
Cased borehole, with uncased length L, in uniform soil, or cylindrical piezometer	$\dfrac{2\pi L}{\log_e\left[\dfrac{L}{D}+\sqrt{\left(1+\left(\dfrac{L}{D}\right)^2\right)}\right]}$

If F is known, then the coefficient of permeability, k, can be found because:

$$k = \frac{A}{FT} \tag{9.62}$$

Constant head testing

Constant head testing is required in all soils where stress changes will result in significant consolidation or swelling. When clay is subjected to an in situ permeability test the effective stresses in the soil are modified by the increase in pore water pressure normally applied. As the soil swells is takes in water, and thus test records normally indicate a higher permeability than, in fact, exists.

Gibson (1963, 1966, 1970) and Wilkinson (1968) have considered the use of the constant head test in clay strata. The object of the test is to find the rate of flow under steady seepage conditions, after swelling has occurred.

Under constant head conditions, the rate of water flow (q) at various times (t) after the test start is plotted as a function of ($1/\sqrt{t}$), (see Fig. 9.38). As time passes swelling reduces and q decreases. After some time it should be possible to extrapolate to find the rate of flow at infinite time ($q_{t=\infty}$), the steady flow. The test results may plot concave up or down, depending on the A value of the soil (Gibson 1966), and generally they will not give a straight line on the ($1/\sqrt{t}$) plot.

The coefficient of permeability may be found from Hvorslev's equations. For example for a cylindrical piezometer:

$$q = \frac{2\pi LkH}{\log_e\left[\frac{L}{D} + \sqrt{\left(1+\left(\frac{L}{D}\right)^2\right)}\right]} \tag{9.63}$$

Alternatively, Maasland and Kirkham (1959) have proposed:

$$q = \frac{3\pi LkH}{\log_e\left[\frac{1.5L}{D} + \sqrt{\left(1+\left(\frac{1.5L}{D}\right)^2\right)}\right]} \tag{9.64}$$

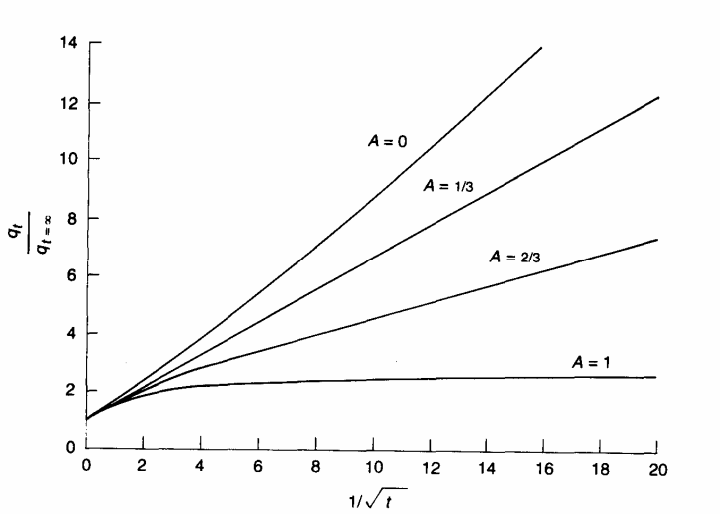


Fig. 9.38 Effect of Skempton's 'A' value on in-situ permeability test results (Gibson 1970).

It has been suggested that coefficient of consolidation values can theoretically be obtained from the slope of the q vs. (1/√t) curve, but since this will typically be curved (depending on the A value, see Gibson (1970)), it will be better to obtain them by combining coefficient of permeability values with coefficient of compressibility values obtained from laboratory tests at the same effective stress levels.

Gibson (1966) has considered the effect of the permeability of the piezometer tip and any surrounding filter sand on the measured value of soil permeability. In the above equation, it is assumed that the permeability of the piezometer installation is infinite, and Gibson has concluded on theoretical grounds that this assumption will be reasonable only if the piezometer ceramic and any surrounding filter sand are at least ten times more permeable than the surrounding soil. Gibson gives two examples of soil permeability limits as shown in Table 9.12.

Table 9.12 Soil permeability limits

Type of high air entry ceramic	Max. k soil (m/s)
Aerox 'Cellaton' Grade 6 ceramic	10^{-9}
Doulton Grade P6A ceramic	10^{-10}

Wilkinson (1968) has considered the effects of smear and trapped air on the results of the test. Air trapped in the piezometer pocket or leads during piezometer construction may lead to high initial flow

rates, but should not seriously effect predictions based on $q_{t=\infty}$. On the other hand, smear may have a very serious effect, particularly where the piezometer is pushed directly into the soil and no sand pocket is used. This means that 'drive-in' piezometers may not be successfully used to determine in situ permeability in soils exhibiting fabric.

Further errors may arise due to leakage past grout seals used to isolate the top of the sand pocket from the upper part of the borehole. Vaughan (1969) considers that leakage effects are only of major consequence when the soil permeability is low, and of the order of 10^{-10}m/s, or less.

The use of high pressures during constant head tests may lead to 'hydraulic fracture', a process whereby the water pressures rise to such a level that they exceed the in situ total stresses. In theory if the soil is normally consolidated (i.e. $K_0 < 1$) vertical cracks will be formed, but in heavily overconsolidated soil cracking will be horizontal because vertical stress levels are smaller than those on the vertical plane (Bjerrum *et al.* 1972).

The following maximum increases in water pressure are suggested:

In situ coefficient of earth pressure at rest (K_0)	0.3	0.5	0.7	1.0
$Max.\left(\frac{increae\ of\ water\ pressure}{initial\ effective\ stress}\right)\left(\frac{\Delta u}{p_0}\right)_{max.}$	0.4	0.7	0.9	1.0

When the water pressure increases above these levels cracks develop in the soil, and the apparent permeability rapidly rises through several orders of magnitude, giving totally misleading results (Fig. 9.39).

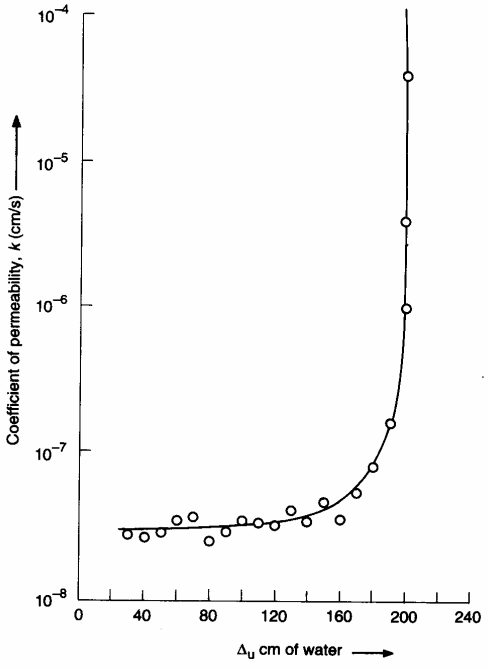


Fig. 9.39 Hydraulic fracture leading to permeability increase (Bjerrum *et al.* 1972).

Of course, the constant head permeability test requires a field apparatus to provide the constant head, and to measure the rate of flow. Two configurations are in use, the more sophisticated using a water cylinder pressurized by an air/water bladder, and flow measurement by variable area conical float flowmeters. Constant air pressure is supplied via a 12 V electrical compressor and a diaphragm-type pressure regulator. This type of apparatus is only suitable for measuring permeabilities in certain

restricted ranges, depending on the specific design, and in addition problems may occur where the test has to be continued for a long period and the total flow volume exceeds 3—4 litres.

A simpler but less precise method of test may easily be built as shown in Fig. 9.40 which basically consists of a large (501) polythene water container, connected by push couplings and a tap to a 100cc glass burette. The head is maintained constant by topping up the polythene drum to a mark, and flow measurement is achieved by turning off the tap and noting the volume change in the burette over a measured time. This type of test is theoretically less accurate than using the more sophisticated type, because the head does not remain constant. It has the advantage however, that large bore pipes can be used throughout, thus allowing measurement of a wide range of permeabilities. With other systems, the restrictions in the constant pressure system may sometimes be greater than the effect of soil permeability.

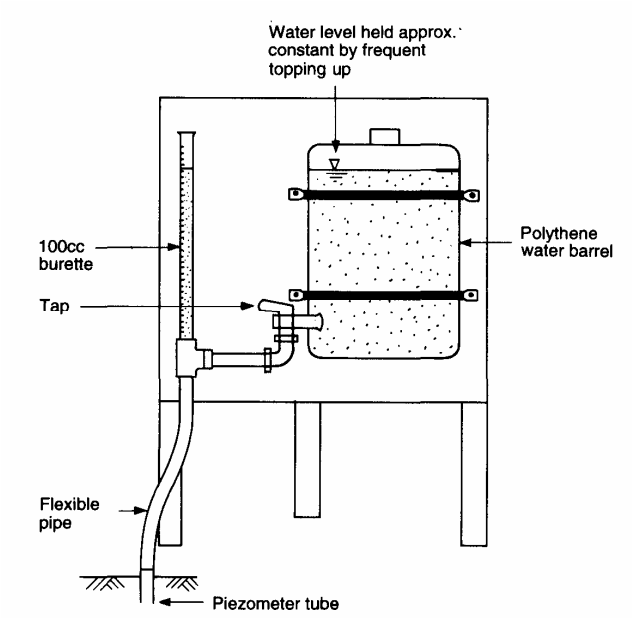


Fig. 9.40 Simplified field constant head apparatus.

The packer or 'Lugeon' test

The rock equivalent of the soil constant head permeability test is the packer test, also sometimes known as the Lugeon test (Lugeon 1933). The test may be carried out in the base of a drillhole using a single inflatable packer to seal off the test section, or after the hole is complete, testing may be carried out at a variety of depths using a double packer to seal the test section top and bottom.

The construction of the packers is critical if leakage is to be avoided, and the longer the packers used, the more effective will be the test. Details of the construction of packers developed at Imperial College, London, may be found in Harper and Ross- Brown (1972), Hoek and Bray (1974), and Pearson and Money (1977).

The test is carried out by lowering the packer or packers to the required depth and inflating them using gas pressure supplied from a nitrogen bottle. The length of each packer should be at least five times the borehole diameter, when expanded; recent researchers have used length to diameter ratios of between 20 and 40. The test section is often about 3m long.

The packers are supported on drill rods, which are also used to supply water under pressure to the test section, and at the top of the borehole the rods are connected via a water swivel or 'gooseneck' to a 'Christmas tree' and flush pump (Fig. 9.41).

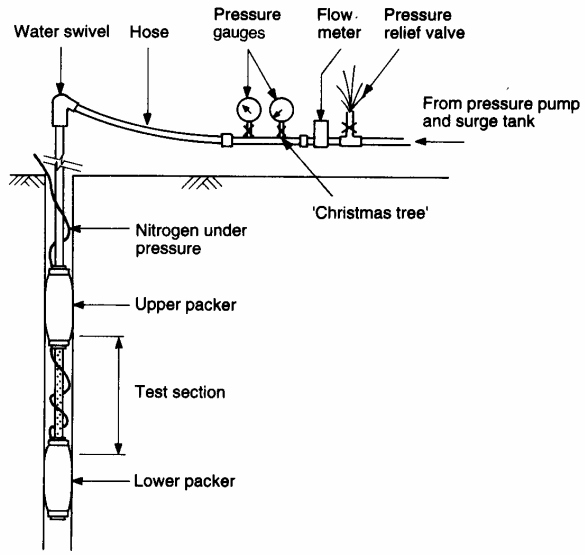


Fig. 9.41 Basic equipment for the double packer test.

The flush pump is capable of producing water at high pressure, but even though this type of pump is often fitted with a gearbox the rate of flow cannot be controlled sufficiently to allow a constant pressure to be applied to the test section. In addition, many pressure pumps run on the piston principle and the output pressure varies with the position of the piston. The 'Christmas tree' therefore needs to include a pressure relief valve, and may contain a surge tank to smooth the pressure from the pump. In addition, at the end coupled to the swivel hose one or two Bourdon pressure gauges and a volumeter are included to allow the measurement of water flow and pressure in various ranges.

The test is carried out in stages, being cycled up to a maximum head and then down again. Typically a maximum head is specified to avoid hydraulic fracture. The allowable net dynamic head (H_t) is often specified as:

$$\frac{overburden\ pressure\ at\ test\ depth}{unit\ weight\ of\ water} = H_t$$

The test is normally carried out using stages such as 1/3, 2/3, 1, 2/3 and 1/3 of the maximum allowable gauge pressure on the 'Christmas tree'. At each pressure stage, the pressure is held constant and the volume measured over a period of 5 min. If the volume measured over two consecutive 5 min periods differs by more than 10%, then measurement should be made for a further 5min period before the pressure is changed. The permeability is calculated from the volume of flow and the net dynamic head applied to the test section.

The net dynamic head (H_t) is:

$$H = (H_p + H_m + H_w) - H_c \tag{9.65}$$

where H_p = pressure head (from the pressure gauge), H_m = head due to the height of the pressure gauge above the ground level at the top of the drillhole, H_w = distance to the groundwater from the top of the drillhole, and H_c = head loss in the test equipment.

H_c, the head loss, must be obtained by calibration of every piece of equipment between the pressure gauge and the test section as a routine before each test. The flexible swivel hose, the swivel, rods and perforated section should be connected, laid out on the ground, and tested by pumping water at different rates while recording the pressure required to sustain the flow. The results may be plotted either with H_c as a function of flow (Fig. 9.42), or H_c as a function of packer stem length. Failure to

calibrate properly may lead to errors in permeability of about one order of magnitude, particularly in highly permeable rocks. In rocks with a permeability of less than 1 x 10^{-7} m/s head losses in the equipment are not likely to be significant.

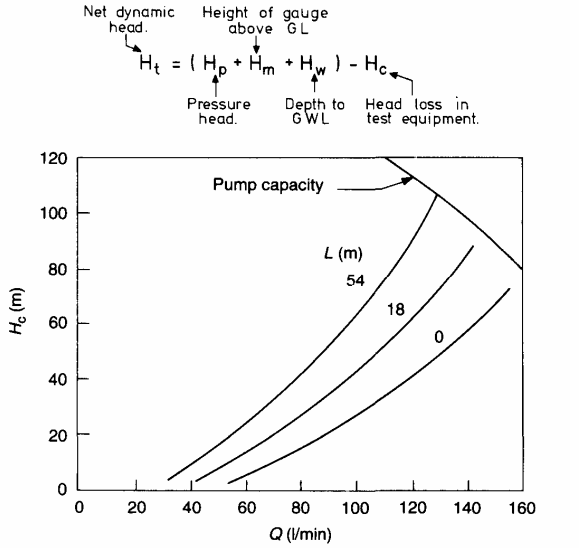


Fig. 9.42 Calibration curves for packer test equipment with various rod lengths (Dick 1975).

The formulae for determining the coefficient of permeability from packer test results are given in the United States Bureau of Reclamation *Earth Manual* (1963) as:

$$k = \frac{Q}{2\pi LH_t}\log_e\left(\frac{L}{r}\right) \qquad \text{for L}\geq 10\text{r} \tag{9.66}$$

and

$$k = \frac{Q}{2\pi LH_t}\sinh^{-1}\left(\frac{L}{2r}\right) \qquad \text{for 10r}>\text{L}\geq\text{r} \tag{9.67}$$

where k = permeability, Q = constant rate of flow into the hole, L = test length, and r = radius of hole tested.

Hoek and Bray (1974) also give the solution for tests carried out with rock joints normal and parallel to the length of the test section. In this case:

$$k = \frac{\left(\frac{mL}{r}\right)}{\frac{Q\log_c}{2\pi LH_t}} \tag{9.68}$$

where $m = (k/k_p)^{½}$, k = permeability at right angles to borehole, and k_p = permeability parallel to the borehole, which if cross-flow is ignored equals the intact rock permeability. For most applications, Hoek and Bray consider a reasonable value of k/k_p to be 10^6, whence $m = 10^3$. This has the effect of increasing the value of the permeability calculated from the USBR equations by about half an order of magnitude. Interpretation in terms of the 'Lugeon coefficient' is less contentious in deep deposits since this is, by definition, 'the water absorption measured in litres per metre of test section per minute at a pressure of 10 kg/cm^2 (= 1000 kN/m^2).

The pressure restrictions necessary to prevent hydraulic fracture in shallow deposits, however, require extrapolation of low pressure test results. Thus:

$$\frac{Q_2}{Q_1} = \left(\frac{p_2}{p_1}\right)^n \tag{9.69}$$

where Q = rate of flow caused by a pressure p, where n = 1 for laminar flow or 0.5 for turbulent fissure flow. Lancaster-Jones (1975) concludes that under normal conditions, flow tends to be turbulent.

Where cycled tests are performed, usually with an ABCBA pressure pattern, results are sometimes presented graphically either as: (1) a head/permeability diagram; or (2) a head/flow ('Lugeon') diagram.

Examples of the interpretation of these diagrams are given by Lugeon (1933), Little *et al.* (1963), Morgenstern and Vaughan (1963), Muir Wood and Caste (1970), Dick (1975) and Pearson and Money (1977). In practice, results typically fall into three groups (Fig. 9.43).

1. The 'ideal' case: Darcy's law dictates that flow will be directly proportional to pressure, and therefore predicts a horizontal line on the head/permeability plot. On the Lugeon diagram, a straight line passing through the origin should be found.
2. Permeability appears to rise with increasing pressure.
3. Permeability appears to fall with increasing pressure.

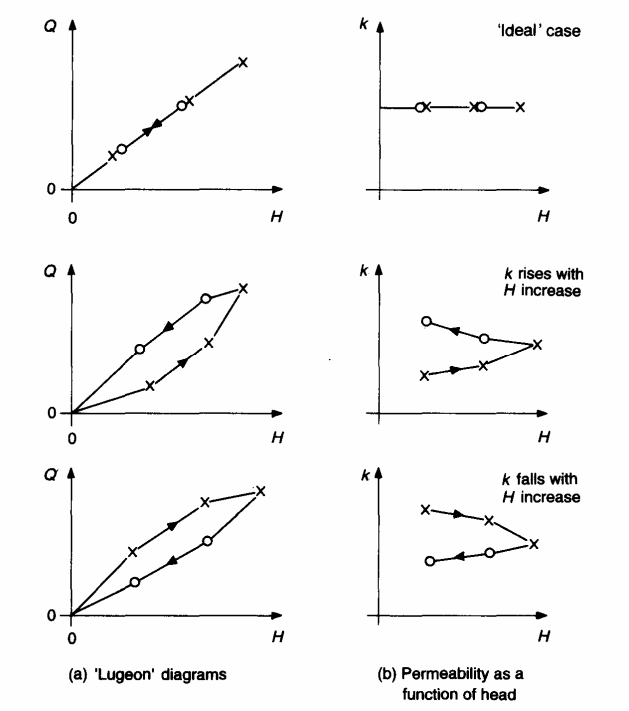


Fig. 9.43 Typical results of packer tests.

Pearson and Money (1977) have observed that these anomalies can also be divided into two other groups: the effects of flow of water in the rock mass, and effects of the test system or technique. The former are unavoidable, but the latter can usually be overcome by careful test technique, coupled with improved instrumentation.

The effects of water flow may lead to either increasing or decreasing permeability with increasing

pressure. Increasing permeability will result from erosion of fractures, or dilation as a result of test water pressures. This latter effect can lead to very large increases in apparent permeability if test pressures rise to hydraulic fracture levels. Decreasing permeability is normally associated with either turbulent flow, or siltation or clogging of fissures. Fissure siltation may occur as a result of migration of fines within the rock mass, but is often associated with the use of dirty test water.

A further effect which appears to give decreasing permeability during the decreasing pressure stages of a test has been described by Little *et al.* (1963) as a 'back pressure' effect. The increasing pressure stages act to charge the rock fissures with high pressure water which reduces the head drop between the test section and the rock mass; flow in the later stages of the test, when the applied head is being reduced, leads to apparently low permeabilities because the true head gradient cannot be assessed. There is little point in conducting head decrease stages if this effect is observed.

The effects of test system and technique have been partly discussed in previous sections. Quite clearly packer leakage and the use of dirty water are highly undesirable, and the losses in the system must be assessed if head measurement is to take place at the top of the hole. In addition to these problems, basic systems such as are shown in Fig. 9.41 suffer from a variety of other defects.

1. *Long-term surging.* The use of piston pumps will lead to very rapid surging, as noted above, but long-term surging can also occur if petrol engines are used to drive pumps.
2. *Air injection.* Slightly faulty suction hosing in the pump system may lead to air being pushed into the test section.
3. *Flow measurement.* Basic systems typically use either a reciprocating chamber or an impeller to activate a mechanical counter. These devices measure total flow, rather than rate of flow and cannot detect sudden changes in flow rate which may indicate the onset of faults such as packer leakage. In addition, such devices tend to stick at low flow rates.

In addition to the effects above, the action of forming the drillhole may lead to a considerable amount of smear over the test section, particularly in soft argillaceous rocks. Under these conditions, 'pumping-in' tests may be expected to yield much lower values of permeability than tests which are based on extraction (such as the rising head test). A further complicating effect arises when tests are conducted above the groundwater level. The fracture pattern of the rock mass gives it a storage capacity which leads to initially high flows, and these flows may take a very considerable time to level off.

The packer test in its present usage is undoubtedly far from perfect, and the equipment is rather difficult to use successfully. Various authors have advocated the use of electronic measuring equipment to measure the actual pressure in the test section and the rate of flow (for example, Pearson and Money (1977)). It is likely that the introduction of such equipment would bring considerable advantages, but the added complexity of the equipment would undoubtedly increase both the cost and difficulty of the test.

References and standards

AAS, G. (1965) ‘A study of the effect of vane shape and rate of strain on the measured values of in situ shear strength of soils’, *Proc. 6th Int. Conf. Soil Mech. and Found. Eng.,* Montreal, Volume 1, pp. 141—145.

AA5, G. (1967) ‘Vane tests for investigation of anisotropy of undrained shear strength of clays’. *Proc. Geot. Conf.,* Oslo, Volume 1, pp. 3—8.

ABBISS, C.P. (1979) ‘A comparison of the stiffness of chalk at Mundford from a seismic survey and large-scale tank test’, *Géotechnique,* 29, 461—468.

AB05HI, H., YOSHIKUMI, H. and MARUYAMA, S. (1970) ‘Constant loading rate consolidation test’, *Soils and Foundations (Japan),* 10 (1), 43—56.

AGARWAL, K.B. (1968) ‘The influence of size and orientation of sample on the undrained strength of London Clay’, PhD thesis, University of London.

AICHISON, G.D. (1973) ‘Twenty-five years of application of soil survey principles in the practice of foundation engineering’, *Geoderma,* No. 10, pp. 99—112.

AKR0YD, T.N.W. (1957) *Laboratory testing in soil engineering,* Soil Mechanics Ltd, Lone don, 233 pp.

AKROYD, T.N.W. (1964) *Laboratory Testing in Soil Engineering,* Soil Mechanics Ltd, London.

ALLEMEJER, K.A. (1973) ‘Application of pedological soil surveys to highway engineering in Michigan’, *Geoderma,* No. 10, pp. 87—98.

ALLEN, T. (1975) *Particle Size Measurement,* Chapman and Hall, London.

ALLUM, J.A.E. (1966) *Photogeneology and Regional Mapping,* Pergamon Press, London.

ANDERSON, W.F. (1974) ‘The use of multistage triaxial tests to find the undrained strength parameters of stony boulder clay’, *Proc. Inst. Civ. Eng.,* 57, 367—372.

ANNAN, A.P. and COSWAY, SW. (1992) ‘Ground penetrating radar survey design’, Annual Meeting of SAGEEP, Chicago.

ANON (1970) ‘Using a remotely controlled borehole camera’, *Ground Engineering,* 3 (5), 20—21.

ANON (1974) ‘Properties of chalk revealed by deep inspection pits’, *Ground Engineering,* 7 (3), 50.

ANON (1977) ‘The logging of rock cores for engineering purposes: Engineering Group Working Party Report’, *Q.J. Eng. Geol.,* 10 (1), 45—5 1.

API BUL Dl 1 (1965) *Glossary of Drilling Fluid and Associated Terms,* American Petroleum Institute, Dallas, Texas.

API RP 1 3B (1969) *Recommended Practice — Standard Procedure for Testing Drilling Fluids,* American Petroleum Institute, Dallas, Texas.

APTED, J.P. (1977) ‘Effects of weathering on some geotechnical aspects of London clay’, PhD thesis, University of London.

ARCHIE, G.E. (1942) ‘The electrical resistivity log as an aid in determining some reservoir characteristics’ *Trans. Am. Inst. Mine Engng.,* 146, 54—62.

ARMAN, A., POPLIN, J.K. and AHMAD, N. (1975) ‘Study of the vane shear’, *ASCE Proc. Speciality Conf. on In-Situ Measurement of Soil Properties,* Raleigh, Volume 1, pp. 93—120.

Association of Ground Investigation Specialists (AGIS) (1979) ‘Specification for ground investigations’, *Ground Engineering,* 12((5), 56—57.

ATKINSON, J.H. and SALLFORS, G. (1991) ‘Experimental determination of stress—strain— time characteristics in the laboratory and in-situ tests’, *Proc. 10th European Conf Soil Mech. and Found. Eng.,* Florence, Volume 3, Balkema, Rotterdam, pp. 915—956.

AUFMUTH, R.E. (1974) ‘Site engineering indexing of rock: Field testing and instrumentation of rock’, *American Society for Testing and Materials, Special Technical Publication, No. 554,* pp. 81—99.

BAGUELIN, F. *et al.* (1972) ‘Expansion of cylindrical probes in cohesive soils’, *Proc. ASCE, J. Soil*

Mech. Found. Div., 98 (SM11), 1129—1 142.
BAGUELIN, F., JEZEQUEL, J-F. and SHIELDS, D.H. (1978) *The Pressuremeter and Foundation Engineering*, Trans Tech Publications, Cleveland, Ohio, USA.
BALDI, *G.*, HIGHT, D.W. and THOMAS, G.E. (1988) 'A reevaluation of conventional triaxial test methods', in R.T. Donoghe, R.C. Cheney, and ML. Silver (eds) *Advanced Triaxial Testing of Soil and Rock*, ASTM STP977, pp. 2 19—263.
BALLARD, R.F., STOKOE, K.H. and MCLEMORE, R. (1983) 'Proposed test methods for crosshole seismic testing', *ASTM Geotech. Testing J., 6*, 210—219.
BALIGH, M.M. (1985) 'Strain path method', *J. Geotech. Div., Am. Soc. Civ. Engrs.*, 111 (GT9), 1108—1136.
BALIGH, MM., AZZOUZ, AS, and CHIN, C.T. (1987) 'Disturbance due to ideal tube sampling', *J. Geotech. Engng. Div., Am. Soc. Civ. Engrs*, 113 (GT7), 739—757.
BARKER, R.D. and WORTHINGTON, P.F. (1972) 'Location of disused mineshafts by geophysical methods', *Civil Engng and PubI. Wks Rev., 67* (788) 275—76.
BARLA, G., SHARP, J.C. and RABAGLIATA (1993) 'Stress strength and deformability assessment for the design of large storage caverns in a weak Eocene chalk', *Proc. MIR '90, 3rd Conf. on Meccanica e Ingegneria delle Rocce (Mechanics and Engineering of Rocks),* Torino, 26—30 Nov., pp. 22-1—22-21.
BARR, M.V. (1977) 'Downhole instrumentation — a review for tunnelling ground investigation', *CIRIA Technical Note 090*, Construction Industry Research and Information Association, London.
BARR, M.V. and HOCKING, G. (1976) 'Borehole structural logging employing a pneumatically inflatable impression packer', *Proc. Sym. on Exploration for Rock Engineering*, Johannesburg.
BARTON, N. (1973) 'Review of a new shear-strength criterion for rock joints', *Engineering Geology, 7*, 287—332.
BARTON, N., LIEN, R. and LUNDE, J. (1974) 'Analysis of rock mass quality and support practice in tunnelling and a guide to estimating support requirements', *Internal Report No. 106*, Norwegian Geotechnical Institute, Oslo.
BA5TIN, E.S. and DAVIS, C.A. (1909) 'Peat deposits in Maine'. *US Geol. Survey, Bull. No. 376*, p. 61.
BAZARAA, A.R.S.S. (1967) 'Use of the standard penetration test for estimating settlement of shallow foundations on sand', PhD thesis, University of Illinois, USA.
BEAUMONT, J.E. (1979) 'Remote sensing survey techniques', *The Highway Engineer*, 26 (4), 2-14.
BEGEMANN, H.K.S. (1961) 'A new method for taking of samples of great length', *Proc. 5th Int. Conf. Soil. Mech and Found. Eng.*, Paris, Volume 1, pp. 437— 440.
BEGEMANN, H.K.S. *(1965)* 'The friction jacket cone as an aid in determining the soil profile', *Proc. 6th Int. Conf. Soil. Mech. and Found. Eng.*, Montreal, Volume 1, pp. 17—20.
BEGEMANN, H.K.S. (1969) 'The Dutch static penetration test with the adhesion jacket cone', *Lab. Grondmech. Delft, Meded.*, 12 *(4)*, 69—100; 13 (1), 1—86.
BEGEMANN, H.K.S. (1971) 'Soil sampler for taking undisturbed sample 66mm in diameter and with a maximum length of 17m'. *Proc. Speciality Session on Quality in Soil Sampling, 4th Asian Conf.*, Int. Soc. Soil. Mech. and Found. Eng., Bangkok, pp. 54—57.
BEGEMANN, H.K.S. (1974) 'The Delft continuous soil sampler', *Bull. Int. Assoc. of Engineering Geology*, No. 10, pp. 35—37.
BELCHER, D.J. (1946) 'Engineering applications of aerial reconnaissance', *Bull. Geol. Soc. Amer.*, 57, pp. 727—34.
BERESFORD, MW. and HURST, J.G. (1971) *Deserted Medieval Villages*, Lutterworth Press, Guildford.
BERESFORD, M.W. and ST JOSEPH, J.K.S. (1979) *Medieval England: An Air Survey*, 2nd edn, Cambridge University Press.
BERGDAHL, U. and BROMS, B.B. (1967) 'New method of measuring in situ settlements', *Proc. ASCE, J. Soil Mech. Found. Engng. Div.*, 93 (SM5), *5* 1—57.
BERRE, T., SCHJETNE, K. and SOLLIE, S. (1969) 'Sampling disturbance of soft marine clays', *Proc. Speciality Session I on Soil Sampling, 7th Int. Conf. Soil. Mech. and Found Eng.*, Mexico, pp.

21—24.
BIENIAWSKT, Z.T. (1973) 'Engineering classification of jointed rock masses', in *The Civil Engineer in South Africa,* pp. 335—343.
BIENIAWSKI, *Z.T.* (1975) 'The point load test in geotechnical practice', *Engineering Geology,* 9, 1—11.
BIENIAWSKI, Z.T. (1989) *Engineering Rock Mass Classifications.* J. Wiley & Sons, New York, 25lpp.
BISHOP, A.W. (1948) 'A new sampling tool for use in cohesionless sands below ground water level', *Géotechnique,* 1, 125—3 1.
BISHOP, A.W. and GIBSON, RE. (1964) 'The influence of the provisions for boundary drainage on strength and consolidation characteristics of soils measured in the triaxial apparatus', *ASTM Spec. Tech. Pub. No. 361, Laboratory Shear Testing of Soils,* ASTM, Ottawa, pp. 435—451.
BISHOP, A.W., GREEN, G.E., GARGA, C.E., ANDRESEN, A. and BROWN, J.D. (1971) 'A new ring shear apparatus and its application to the measurement of residual strength', *Géotechnique,* 21, 273—328.
BISHOP, A.W. and HENKEL, D.J. (1962) *The Measurement of Soil Properties in the Triaxial Test,* 2nd edn, Edward Arnold, London.
BISHOP, A.W. and LITTLE, A.L. (1967) 'The influence of the size and orientation of the sample on the apparent strength of the London Clay at Maldon, Essex', *Proc. Geot. Conf,* Oslo, Volume 2, pp. 89—96.
BISHOP, A.W. and VAUGHAN, P.R. (1962) 'Selset reservoir: design and performance of the embankment', *Proc. Inst. Civ. Eng.,* 21, 305—346.
BISHOP, A.W., WEBB, DL. and LEWIN, P.!. (1965) 'Undisturbed samples of London clay from the Ashford Common shaft: strength-effective stress relationships', *Géotechnique,* 15, 1—31.
BJERRUM, L. (1972) 'Embankments on soft ground', *Proc. ASCE Speciality Conf. on Performance of Earth and Earth-Supported Structures,* Purdue University, USA, Volume 2, pp. *1—54.*
BJERRUM, L. (1973) 'Problems of soil mechanics and construction on soft clays and structurally unstable soils', State-of-the-art report, Session IV, *Proc. 8th Int. Conf. Soil. Mech. and Found. Eng.,* Moscow, Volume 3, pp. 111—159.
BJERRUM, L. and EIDE, 0. *(1956)* 'Stability of strutted excavations in clay', *Geotechnique, 6,* 32—47.
BJERRUM, L., KENNEY, T.C. and KJAERNSLI, B. (1965) 'Measuring instruments for strutted excavations', *Proc. ASCE J. Soil. Mech. Found. Engng. Div.,* 91 (SM 1), 111—141.
BJERRUM, L., NASH, J.K.T.L., KENNARD, R.M. and GIBSON, R.E. (1972) 'Hydraulic fracturing in field permeability testing', *Géotechnique,* 22 (2), 319—332.
BLYTH, F.G.H. and DE FREITAS, M.H. (1984) *A Geology for Engineers,* 7th edn, Edward Arnold, London.
Bois, P., LA PORTE, M., LAVAERGNE, M., THOMAS, G. (1971) 'Essai de determination automatique des vitessess sismiques par mesures entre puits', *Geophysical Prospecting,* 19, 42—83.
BOROWICKA, H. (1973) 'Rearrangement of grains by shear test with sand', *Proc. 8th Int. Conf. Soil Mech. and Found. Eng.,* Moscow, Volume 1.1, pp. 71—77.
BOULTON, G.S. (1972) 'Modern arctic glaciers as depositional models for former ice sheets', *J. Geol. Soc.,* 128, 361—393.
BRATCHELL, G.E., LEGGATT, A.J. and SIM0N5, N.E. (1975) 'The performance of two large oil tanks founded on compacted gravel at Fawley, Southampton, Hampshire', *Proc. BGS Conf. on Settlement of Structures,* Cambridge, Pentech Press, London, pp. 3—9.
BRETRING, A.E. (1936) 'Soil studies for the Storstrom Bridge, Denmark', *Proc. 1st Int. Conf Soil Mech. and Found. Eng.,* Harvard, Volume 1, pp. 314—327.
BRIAND, J.L. and MIRAN, J. (1992) *The flat dilatometer test,* US Department of Transportation, Federal Highway Administration Pub. No. FHWA-SA-9 1-044, p. 102, Washington, DC.
BRIDGE, T. and ELLIOTT, S.R.L. (1967) 'The application of soil mechanics to railway engineering', *Proc. Inst. Civ. Eng.,* 37, 243—46.
British Geotechnical Society (1994) *The Geotechnical Directory of the United Kingdom,* 4th edn

(1994—1995) The British Geotechnical Society, at The Institution of Civil Engineers, London.
BRITTO, A.M. and KUSAKABE, O. (1984) 'On the stability of supported excavations', *Canadian Geotech. J.*, 21, pp. 338—348.
BROADBENT, C.D. and RIPPERE, K.H. (1970) 'Fracture studies at the Kimberley Pit', *Proc. Symp. Planning Open Pit Mines*, Johannesburg, A.A. Balkema, Amsterdam, pp. 171—79.
BROCH, E. and FRANKLIN, J.A. (1972) 'The point-load strength test', *Int. J. Rock Mech. Mi Sci.*, 9, 669—697.
BROMHEAD, E.N. (1979) 'A simple ring shear apparatus', *Ground Engineering*, 12 *(5)*, 40, 42, 44.
BR0M5, B.B. (1980) 'Soil sampling in Europe: State-of-the-art'. *Proc. ASCE, J. Geot. Eng. Div.*, 106 (GTI), 65—98.
BROMS, B.B. and HALLEN, A. (1971) 'Sampling of sand and moraine with the Swedish Foil Sampler', *Proc. Speciality Session on Quality in Soil Sampling, 4th Asian Conf., Int. Soc. Soil Mech. and Found. Eng.*, Bangkok, pp. 49—53.
BUCHANON, S.J. (1936) Discussion on 'Exploration of soil conditions and sampling operations', *Proc. 1st Inst. Conf. Soil. Mech. and Found. Eng.*, Harvard, Volume 3, p. 29.
BUCHANON, S.J. (1938) 'Levees in the Lower Mississippi Valley', *Trans. ASCE*, 103, 1378— *139* 5.
BULLOCK, S.J. (1978) 'The case for using multichannel seismic refraction equipment and techniques for site investigation', *Bulletin of the Association of Engineering Geologists*, 14 (1), 19—35.
BURLAND, J.B. and BURBIDGE, M.C. (1982) 'Settlement of foundations on sand and gravel', *Proc. ICE*, Part 1, 78, 1325—1371.
BURLAND, J.B. and DAVIDSON, W.D. (1976) 'A case study of cracking of columns supporting a silo due to differential foundation settlement', *Building Res. Station, Current Paper CP42/76*, Building Research Station.
BURLAND, J.B. and LORD, J.A. (1970) 'The load deformation behaviour of Middle Chalk at Mundford, Norfolk: a comparison between full-scale performance and in-situ and laboratory measurements', *Proc. Conf. on In situ Investigations in Soils and Rocks*, British Ge6technical Society, London, pp. 3—15.
BURLAND, J.B., MOORE, J.F.A. and SMITH, P.D.K. (1972) 'A simple and precise borehole extensometer', *Géotechnique*, 22, 174—177.
BURLAND, J.B., SILLS, G.C. and GIBSON, R.E. (1973) 'A field and theoretical study of the influence of nonhomogeneity on settlement', *Found. and Soil Tech.*, 3, 250—256.
BURLAND, J.B. and SYMES, M.A. (1982) 'A simple axial displacement gauge for use in the triaxial apparatus', *Géotechnique*, 26, 371—375.
BURTON, A.N. (1969) 'Air photograph interpretation in site investigation for roads', *Roads and Road Constr., 47* (555), 72—76.
BURWASH, A.L. and WEISNER, W.R. (1984) 'Classification of peats for geotechnical engineering purposes', *Proc. Cold Regions Eng. Specialty Conf*, Can. Soc. for Civil Eng., Montreal, Volume 2, pp. 979— 998.
BURWASH, A.L. and WEI5NER, W.R. (1987) Discussion on Hobbs (1986) Q. *J. Eng. Geol.*, 20, 97.
CADLING, L. and ODENSTAD, S. (1950) 'The vane borer', *Proc. Royal. Swed. Geot. Inst.*, No. 2.
CAMBEFORT, H. and MAZIER, G. (1961) 'Recherches des écoulements d'eau privileges set prélévements intacts de sols cohérents hétérogénes', *Proc. 5th Int. Conf. Soil Mech. and Found. Eng.*, Paris, Volume 1, pp. 441—445.
CARTER, T.G. (1983) 'The site investigation and engineering characterization of glacial and glacilacustrine materials', PhD thesis, University of Surrey.
CASAGRANDE, A. (1932) 'The structure of clay and its importance in foundation engineering', *J. Boston Soc. Civ. Eng.*, 19, 168—221.
CA5AGRANDE, A. (1936) 'The determination of preconsolidation load and its practical significance', *Proc. 1st Int. Conf. Soil Mech. and Found. Eng.*, Harvard, Volume 3, pp. 60—64.
CASAGRANDE, A. (1947) 'Classification and identification of soils', *Proc. ASCE*, 73 (6),

783—810.
CASAGRANDE, A. (1948) ‘Classification and identification of soils’, *Trans. ASCE,* 113, 901— 992; reprint of Casagrande (1947) including discussion.
CASAGRANDE, A. (1960) ‘Karl Terzaghi — his life and achievements’, in *From Theory to Practice in Soil Mechanics,* John Wiley, New York. pp. 3—21.
CHANDLER, R.J. (1988) ‘The in-situ measurement of the undrained shear strength of clays using the field vane’, in A.F. Richards (ed.) *Vane Shear Strength Testing in Soils. Field and Laboratory Studies,* ASTM STP 1014, ASTM, Philadelphia, pp. 13—44.
CHANDLER, R.J., HARWOOD, A.H. and SKINNER, P.J. (1992) ‘Sample disturbance in London clay’, *Géotechnique,* 46 (4), *577—585.*
CHAVES, J.R. and SCHUSTER, R.L. (1964) ‘Use of aerial colour photography in materials surveys’, *Highway Research Record,* No. 63, pp. 1—9.
Chelsea Speliological Society (1990) ‘Database on underground sites in south east England’, *Records of the Chelsea Speliological Society,* Volume 18 (ISSN 0309-409X), Chelsea Speliological Society, Chelsea Community Centre, World’s End Place, Chelsea, London SW 10.
CHENEY, J.E. (1974) ‘Techniques and equipment using the surveyor’s level for accurate measurement of building movement’, *Proc. Symp. on Field Instrumentation in Geotechnical Engineering,* Butterworths, London, pp. 85—99.
CHERNYSHEV, S.N. and DEARMAN, W. (1991) *Rock Fractures,* Butterworth-Heinemann, London, 272 pp.
CHILD, G.H. (1986) ‘Soil descriptions — Quo Vadis?’ in A.B. Hawkins (ed.) *Site Investigation Practice: Assessing BS 5930,* Eng. Geol. Special Pub. No. 2, Geological Society, pp. 73—81.
CHIN, C.T. (1986) ‘Open-ended pile penetration in saturated clays’, PhD thesis, MIT, Cambridge, Mass., USA.
CLAPHAM, H.G. (1978) Personal communication on the failure of a temporary cut at Wraysbury Reservoir.
CLARKE, B.G. and ALLAN, P.G. (1989) ‘A self-boring pressuremeter for testing weak rock’, *Proc. 12th Int. Conf Soil Mech. and Found. Eng.,* Volume 1, pp. 21 1—214.
CLAYTON, C.R.I. (1986) ‘Sample disturbance and B55930’, in A.B. Hawkins (ed.) *Site Investigation Practice: Assessing BS 5930,* Eng. Geol. Special Pub. No. 2, Geological Society, pp. 33—39.
CLAYTON, C.R.I. (1990) ‘SPT energy transmission: theory measurement and significance’, *Ground Engineering,* 23 (10), 33—42.
CLAYTON, C.R.I. (1993) *The Standard Penetration Test (SPT): Methods and Use,* Construction Industry Research and Information Association, Funder Report CP/7, 129 pp.
CLAYTON, C.R.I., EDWARDS, A. and WEBB, D.L. (1991) ‘Displacements in London clay during construction’, *Proc. X Eur. Conf. on Soil Mech. and Found. Eng,* Florence, Volume II, Balkema, Rotterdam, pp. 791—796.
CLAYTON, C.R.I., HIGHT, D.W. and HOPPER, R.J. (1992) ‘Progressive destructuring of Bothkennar clay: implications for sampling and reconsolidation procedures’, *Géotechnique,* 42 (2), 2 19—239.
CLAYTON, C.R.I. and JUKES, A.W. (1978) ‘A one point cone penetrometer liquid limit test?’, *Géotechnique,* 28, 469—472.
CLAYTON, CR1. and KHATRUSH, S.A. (1986) ‘A new device for measuring local strains on triaxial specimens’, *Géotechnique,* 36 (4), 593—597.
CLAYTON, C.R.I., SIM0N5, N.E. and INsT0NE, S.J. (1988) ‘Research on dynamic penetration testing of sands’, *Proc. ISOPT— 1,* Volume 1, pp. 415— 422.
COLLEY, G.C. (1963) ‘The detection of caves by gravity measurements’, *Geophysical Prospecting,* 1 (2), 1—9.
C0NNOR, I.G. (1980) ‘A study of soil structure interaction with particular reference to sugar silos’, MSc dissertation, University of Surrey.
COOK, K.L. and VAN NOSTRAND, R.G. *(1954)* ‘Interpretation of resistivity data over filled sinks’, *Geophysics,* 19 (4), 761—90.
COOLING, L.F. (1942) ‘Soil mechanics and site exploration’, *J. Inst. Civ. Eng.,* 18, 37—61.
COOLING, L.F. and GOLDER, H.Q. (1942) ‘The analysis of the failure of an earth dam during construction’, *J. Inst. Civ. Eng.,* 19, 38—55.

COOLING, L.F. and M. SMITH, B.A. (1936) 'Exploration of soil conditions and sampling operations', *Proc. 1st. Int. Conf Soil Mech. and Found. Eng.,* Harvard, Volume 1, p. 12.
COUTTS, J.S. (1986) 'Correlations between piezocone results and laboratory data from six test locations', MSc dissertation, University of Surrey.
CRAWFORD, C.B. (1964) 'Interpretation of the consolidation test', *Proc. ASCE, J. Soil Mech. Found. Div,* 90 (SM5), 87—102.
CRAWFORD, O.G.S. and KEILLER, A. (1928) *Wessex from the Air*, Oxford University Press.
CRESPELLANI, T. and VANNUCCHI, G. (1994) 'Dynamic properties of soils', *Proc. 10th European Conf. on Soil Mech. and Found. Eng.,* Florence, Volume 4, Balkema, Rotterdam, 1509—1517.
CUMMING, J.D. and WICKLUND, A.P. (1980) Diamond Drill Handbook, 3rd edn, 2nd revision, J.K. Smit, Ontario, Canada.
DALTON, J.C.P. and HAWKINS, P.G. (1989) 'Fields of stress — some measurements of in-situ stress in a meadow in the Cambridgeshire countryside', *Ground Engineering,* 15(4), *15—*23.
D'APPOLONIA, D.J., D'APPOLONIA, E. and BRISETTE, R.F. (1970) 'Discussion on "Settlement of Spread Footings on Sand", *Proc. ASCE, J. Soil Mech. and Found. Engng. Div., 96* (S.12), 754—761.
DARAMOLA, 0. (1978) 'The influence of stress history on the deformation of sand', PhD / thesis, University of London (Imperial College).
DARRACOT, B.W. and LAKE, M.I. (1981) 'An initial appraisal of ground probing radar for site investigation in Britain', *Ground Engineering,* 14 (3), 14—18.
DARRACOTT, B.W. and MCCANN, D. (1986) Planning geophysical surveys in A.B. Hawkins (ed.) *Site Investigation Practice: Assessing BS 5930,* Eng. Geol. Special Pub. No. 2, Geological Society, pp. 85—89.
DA SILVERIA, A.F., RODRIGUES, F.P., GROSSMAN, N.F. and MENDES, F. (1966) 'Qualitative characterisation of the geometric parameters of jointing in rock masses', *Proc. 1st Congress Int. Soc. Rock. Mech.,* Lisbon, pp. 225—233.
DEARMAN, W.R. (1974) 'The characterisation of rock for civil engineering practice in Britain', *Centenaire de Ia Societe Geologique de Belgique,* Colloque Geologic de l'Ingenieur, Liege, pp. 1—75.
DEARMAN, W.R. BAYNES, F.J. and PEARSON, R. (1977) 'Geophysical detection of disused mineshafts in the Newcastle-Upon-Tyne area, N.E. England', Q. *J. Eng. Geol.,* 10 (3), 257—270.
DE BEER, E.E. and MARTENS, A. *(1957)* 'Method of computation of an upper limit for the influence of heterogeneity of sand layers on the settlement of bridges', *4th Int. Conf. Soil Mech. and Found. Eng.,* London, Volume 1, pp. 275—281.
DECOURT, L. (1990) 'The Standard Penetration Test, state-of-the-art report', *Proc. 12th Int. Conf. on Soil Mech. and Found. Eng.,* Rio de Janeiro.
DE MELLO., V.F.B. (1969) 'Foundation of buildings in clay', *Proc. 7th Int. Conf. Soil Mech. and Found. Eng.,* Mexico, State-of-the-art volume, pp. 49—136.
DE MELLO, V.F.B. (1971) 'The standard penetration test', *Proc. 4th PanAmerican Conf. Soil Mech. and Found. Eng.,* San Juan, Puerto Rico, Volume 1, pp. 1—86.
DEERE, D.U. (1964) 'Technical description of rock cores for engineering purposes', *Rock Mechanics and Engineering Geology,* 1(1), 17—22.
DEERE, D.U. and MILLER, R.P. (1966) 'Engineering classification and index properties for intact rock', *Technical Report No. AFWL-TR-65-116,* Air Force Weapons Lab., Kirtland Air Force Base, New Mexico,
Department of Transport (1976a) *Specification for Road and Bridge Works,* HMSO, London.
Department of Transport (l976b) *Notes for Guidance on the Specification for Road and Bridge Works,* HMSO, London.
DE RUITER, J. (1971) 'Electric penetrometer for site investigations', *Proc. ASCE, J. Soil Mech. and Found. Engng. Div., 97* (5M2), *457-472.*
DICK, R.C. (1975) 'In situ measurement of rock permeability: Influence of calibration error on test results'. *Bull. Assoc. Eng. Geol.,* 12 (3), 193—211.

DIMITRIJEVIC, M.D. & PETROVIC, R.S. (1965) *The use of sphere projection in geology.* Geoloski Zavod, Ljubljana.
DINES, K.A. and LYTLE, R.J. (1979) 'Computerised geophysical tomography', *Proc. IEEE, 67* (7), 1064—1073.
DIX, C.H. (1939) 'Refraction and reflection of seismic waves II: Discussion of the physics of refraction prospecting'. *Geophysics,* 4 *(4),* 238—41.
DOBRIN, M.B. (1960) *Introduction to Geophysical Prospecting,* McGraw-Hill, New York, *446* pp.
DOHR, G. (1975) *Applied Geophysics.' Introduction to Geophysical Prospecting,* Pitman, London.
DUCKER, A. (1969) 'Method for extraction of undisturbed frozen cores', *Proc. Speciality Session 1 on Soil Sampling, 7th Int. Conf. Soil Mech. and Found. Eng.,* Mexico, pp. 19—20.
DUMBLETON, M.J. (1968) 'The classification and description of soils for engineering purposes: a suggested revision of the British system', *Ministry of Transport, RRL Report LR 182,* Road Research Laboratory, Crowthorne, Berks.
DUMBLETON, M.J. (1981) 'The British soil classification system for engineering purposes: its development and relation to other comparable systems', *TRRL Report LR 1030,* Transport and Road Research Laboratory, Crowthorne, Berks.
DUMBLETON, M.J. and NIXON, I.K. (1973) 'The soil classification system proposed for use in the revised Code of Practice on Site Investigations', Informal discussion, *Proc. ICE,* Part 2, 55, 997—999.
DUMBLETON, M.J. and WEST, G. (1966) 'Some factors affecting the relation between clay minerals in soils and their plasticity', *Clay Minerals, 6* (3), 179—193.
DUMBLETON, M.J. and WEST, **G.** (1970) *Report LR 369, Air-photograph Interpretation for Road Engineers in Britain,* Transport and Road Research Laboratory, Crowthorne, Berks.
DUMBLETON, M.J. and WEST, G. (1976) 'Preliminary sources of information for site investigations in Britain', *Transport and Road Research Laboratory Report LR 403* (revised edition), Crowthorne, Berks.
DUNCAN, N. (1969) *Engineering Geology and Rock Mechanics,* Volume 1, Leonard Hill, London.
DUNCAN, N. and HANCOCK, K.E. (1966) 'The concept of contact stress in the assessment of the behaviour of rock masses as structural foundations', *Proc. 1st Congr. Int. Soc. Rock Mech.,* Lisbon, Volume 2, PP. 487—492.
DUNNICLIFF, J. with GREEN, G.E. (1988) Geotechnical Instrumentation for Monitoring Field Performance, Wiley, New York.
DUNNICLIFFE, C.J. (1971) 'Equipment for field deformation measurements', *Proc. 4th PanAmerican Conf on Soil Mech. Found. Eng.,* Puerto Rico, Volume 2, pp. 3 19—332.
DURGUNOGLU, H.T. and MITCHELL, J.K. (1975) 'Static penetration resistance of soils: I Analysis, II Evaluation of Theory and Implications for Practice', *Proc. ASCE Symp. on In-situ Measurements of Soil properties,* Rayleigh, Volume 1, Pp. 151—171.
EARLY, KR. and DYER, K.R. (1964) 'The use of a resistivity survey on a foundation site underlain by karst dolomite', *Géotechnique,* 14 (4), 341—348.
EHRENBERG, J. (1933) 'Geräte zur Entnahme von Bodenproben für bodenphysikalischer Untersuchungen', *Bautechnik,* 11, 303—306.
ENGLAND, B. (1976) 'Low velocity foam flush drilling', *Ground Engineering, 9* (3), 24—25.
EVANS, R. (1972) 'Air photographs for soil survey in lowland England: Soil patterns', *Photogrammetric Record,* 7 (39), 302—22.
EWAN, V.J. and WEST, G. (1981) 'Reproducibility of joint orientation measurements in rock', *TRRL Report SR 702,* Transport and Road Research Laboratory, Crowthorne, Berks.
EWAN, V.J., WEST, G. and TEMPORAL, J. (1981) 'Reproducibility of joint spacing measurements in rock', *TRRL Report LR 1013,* Transport and Road Research Laboratory, Crowthorne, Berks.
FAHLQUIST, F.E. (1941) 'New methods and technique in subsurface explorations', *J. Boston Soc. Civ. Eng.,* 28, 144—160.
FELD (1948) 'Early history and bibliography of soil mechanics'. *Proc. 2nd Int. Conf. Soil Mech. and Found. Eng.,* Rotterdam, Volume 1, pp. 1—7.
FISCHER, W.A. (1958) 'Colour aerial photography in photogeologic interpretation', *Photogrm. Engng,* 24, 545—49.
FISCHER, W.A. (1962) 'Colour aerial photography in geologic investigations', *Photogrm. Engng.,* 28

(1), 133—139.
FLETCHER, G.F.A. (1965) 'Standard penetration test: Its uses and abuses', Proc. ASCE, J. Soil Mech. and Found. Engng. Div., 91 (SM4), pp. 67—75.
FOOKE5, P.G., DEARMAN, W.R. and FRANKLIN, J.A. (1971) 'Some engineering aspects of rock weathering with field examples from Dartmoor and elsewhere', Q. *J. Eng. Geol., 4,* 139—185.
FOSTER, C.J. (1980) 'A study of coastal landslips at Sandgate, Kent', MSc dissertation, University of Surrey.
FOSTER, CR. (1962) 'Field problems: Compaction', in G.A. Leonards (ed.) *Foundation Engineering,* McGraw-Hill, New York, pp. 1000—1024.
FRANKLIN, J.A., BROCH, E. and WALTON, G. (1971) 'Logging the mechanical character of rock', *Trans. Inst. Mm. Met.,* 80 A, 1—9.
FRY, N. (1984) *The field description of metamorphic rocks, Geological Society of London Handbook,* Open University Press, Milton Keynes, 110 pp.
GALLE, E.M. and WILHOIT, J.C. (1962) 'Stresses around a wellbore due to internal pressure and unequal principal geostatic stresses', *Soc. Petroleum Engnrs J.,* 2 (2), 145—155.
Geological Society of America (1963) *Rock Colour Chart,* Geological Society of America. Geological Society of London (1970) 'The logging of rock cores for engineering purposes: Engineering Group Working Party Report', Q. *J. Eng. Geol.,* 3 (1), 1—24.
Geological Society of London (1972) 'The preparation of maps and plans in terms of engineering geology: Engineering Group Working Party Report', Q. *J. Eng. Geol.,* 5(4), 295—382.
Geological Society of London (1977) 'The description of rock masses for engineering purposes: Engineering Group Working Party Report', Q. *J. Eng. Geol.,* 10 (4), 355—388.
GEORGIANNOU, V.N. and HIGHT, D.W. (1994) 'The effects of centreline tube sampling strains on the undrained behavior of two stiff overconsolidated clays', *ASTM Geotech. Testing J.,* 17 (4), 475—487.
GIBBS, H.J. and HOLTZ, W.G. (1957) 'Research on determining the density of sands by spoon penetration testing', *Proc. 4th Int. Conf Soil Mech. and Found. Eng.,* London, Volume 1, pp. 35—39.
GIBSON, R.E. (1963) 'An analysis of system flexibility and its effects on time-lag in pore- water pressure measurements', *Géotechnique,* 13 (1), 1—11.
GIBSON, R.E. (1966) 'A note on the constant head test to measure soil permeability in situ', *Géotechnique,* 16, 256—259.
GIBSON, R.E. (1970) 'An extension to the theory of the constant head in situ permeability test', *Géotechnique,* 20 (2), 193—197.
GIBSON, R.E. and ANDERSON, W.F. (1961) 'In situ measurement of soil properties with the pressuremeter', *Civil Engineering and Public Works Review,* 56 (658), 615—618.
GIBSON, R.E. and HENKEL, D.J. (1954) 'Influence of duration of tests at constant rate of strain on measured "drained" strength', *Géotechnique, 4* (1), 6—15.
GILBERT, P.F.C. (1972) 'The reconstruction of a three-dimensional structure from projections and its application to electron microscopy: II direct methods', *Proc. R. Soc. London,* B-182, 89—102.
GOODMAN, R.E. (1976) *Methods of Geological Engineering in Discontinuous Rocks,* West, New York.
GOODMAN, R.E. (1993) *Engineering Geology,* J. Wiley & Sons, New York. GORDON, R. and HERMAN, G.T. (1974) 'Three-dimensional reconstruction from projections: a review of algorithms', *Int. Rev. Cytology,* 5 (38), 111—151.
GRAINGER, P., MCCANN, D.M. and GALLOIS, R.W. (1973) 'The application of the seismic refraction technique to the study of the fracturing of the Middle Chalk at Mundford, Norfolk', *Géotechnique,* 23 (2), 219—32.
GREEN, P.A. (1968) 'Ground and materials investigations for road schemes: Needs and methods: Informal discussion,' *Proc. Inst. Civ. Eng.,* 41, 635—38.
GREENMAN, A. (1992) 'Motorway pavement checked in 40km radar survey', *Ground Engineering,* 25 (5), 8.
GREENSMITH, J.T. (1978) *Petrology of the Sedimentary Rocks,* Allen and Unwin, London.
GRIFFITHS, J.C. (1946) 'Clay research and oil developing problems', *J. Inst. Pet.,* 32, 18—31.

GRIFFITHS, D.H. and KING, R.F. (1965) *Applied Geophysics for Engineers and Geologists,* Pergamon Press, Oxford.
GUTT W.H. and HARRISON, W.H. (1977) 'Chemical resistance of concrete', *Concrete,* 11 *(5), 35—37.*
GUYOD, H. (1964) 'Use of geophysical logs in soil engineering', *American Society for Testing and Materials Symposium on Soil Exploration, Special Technical Publication No. 351,* pp. 74—85.
HABERFIELD, C.M. and JOHNSTON, I.W. (1993) 'Factors influencing the interpretation of pressuremeter tests in soft rock', *Proc. Symp. on Geotechnical Engineering of Hard Soils — Soft Rocks,* Athens, Volume 1, Balkema, Rotterdam, pp. 525—532.
HACKMAN, R.J. (1967) 'Time shadows, terrain and photo-interpretation', *Professional Paper 575-B,* US Geological Survey, pp. B 155—60.
HAJJ, A.R. (1990) 'The simulation of sampling disturbance and its effects on the deformation behaviour of clays', PhD thesis, University of Sheffield.
HANNA, T.H. (1973) Foundation Instrumentation, Trans Tech Publications, Cleveland, Ohio.
HANZAWA, H. and MATSUDA, E. (1977) 'Density of alluvial sand deposits obtained from sand sampling', *Proc. Speciality Session 2 on Soil Sampling, 9th Int. Conf. Soil Mech. and Found. Eng.,* Tokyo, pp. 7—14.
HARDING, H.J.B. (1949) 'Site investigations including boring and other methods of subsurface explorations', *J. Inst. Civ. Eng.,* 32, 111—137.
HARLEY, J.B. (1975) *Ordnance Survey Maps: A Descriptive Manual,* Ordnance Survey, Southampton.
HARLEY, J.B. and PHILLIPS, C.W. (1964) *The Historian's Guide to Ordnance Survey Maps,* The Standing Conference for Local History, London.
HARPER, H.J. (1931) 'An improved soil sampling tube', *Soil Science,* p. *65.*
HARPER, T.R. and Ross-BROWN, D.M. (1972) 'An inexpensive durable borehole packer', *Imperial College Rock Mechanics Research Report No. D24,* Imperial College, London.
HARRISON, I. R. (1991) 'A pushed thinwall tube sampling system for stiff clays', *Ground Engineering,* Apr., 30—34.
HAWKINS, A.B. (1986) 'Rock descriptions', in A.B. *Hawkins (ed.) Site investigation Practice: Assessing BS 5930, Eng. Geol. Special Pub. No. 2,* Geological Society, pp. *59—65.*
HEAD, K.H. (1980) *Manual of Soil Laboratory Testing, Volume 1, Soil Classification and Compaction Tests,* Pentech Press, London.
HEAD, K.H. (1982) *Manual of Soil Laboratory Testing,* Volume 2, Pentech Press, London. HEAD, K.H. (1986) *Manual of Soil Laboratory Testing: Effective Stress Tests,* Volume 3, Pentech Press, London.
HEALY, P.R. and HEAD, J.M. (1984) *Construction over abandoned mine workings,* Construction Industry Research and Information Association Special Publication 32, 94pp.
HEINZ, W.F. (1989) *Diamond drilling handbook,* 2nd edn., W.F. Heinz, Halfway House, South Africa, S2Spp.
HENCHER, SR. and RICHARDS, L.R. (1989) 'Laboratory direct shear testing of rock discontinuities', *Ground Engineering,* 24—31.
HENDERSON, G., SMITH, P.D.K. and ST JOHN, H.D. (1979) 'The development of the push-in pressuremeter for offshore site investigation', *Proc. Conf On Offshore Site Investigation,* Society for Underwater Technology, London, March, pp. 157—167.
HENDRON, A.J. (1968) 'Mechanical properties of rock', in K.K. Stagg and O.C. Zienkiewicz (eds) *Rock Mechanics in Engineering Practice,* Wiley, London.
HEYMAN, J. (1972) *Coulomb's Memoir on Statics — An Essay on the History of Civil Engineering,* Cambridge University Press.
HIGc1INB0Tr0M, I.E. (1971) 'Superficial structures in reconnaissance and feasibility studies', Q. *J. Eng. Geol., 4,* 307—10.
HIGGINBOTTOM, I.E. (1976) 'The use of geophysical methods in engineering geology', *Ground Engineering, 9* (2), 34—38.
HIGGINB0Tr0M, I.E. and FOOKES, P.G. (1970) 'Engineering aspects of periglacial features in Britain', Q. *J. Eng. Geol.,* 3(2), 85—117.
HIGHT, D.W. (1982) 'A simple piezometer probe for the routine measurement of pore pressure in

triaxial tests on saturated soils', *Géotechnique,* 33 (4), 355—383.
HIGHT, D.W. (1986) 'Laboratory testing: assessing BS5930', in A.B. Hawkins (ed.) *Site Investigation Practice: Assessing BS5930,* Eng. Geol,. Special Pub. No. 2, Geological Society, pp. 43—51.
HIGHT, D.W. and BURLAND, J.B. (1990) *Review of soil sampling and laboratory testing for the Science and Engineering Research Council,* Summary Report, SERC, Swindon, England.
HILL, R. *(1950) The Mathematical Theory of Plasticity,* Clarendon Press, Oxford. HILLIER, R.P. (1992) 'The plate test on clay — A finite element study', PhD thesis, Department of Civil Engineering, University of Surrey.
HIMUS, G.W. and SWEETING (1968) *The Elements of Field Geology,* 2nd edn, University Tutorial Press, London.
HINDS, D. (1974) 'A method of taking an impression of a borehole wall', *Imperial College Rock Mechanics Research Report No. 28,* Imperial College, London.
HOBBS, N.B. (1975) 'Factors affecting the prediction of settlement of structures on rock: with particular reference to the Chalk and Trias: General report and state-of-the-art review for session *4', Proc. Conf on Settlement of Structures,* BGS Cambridge, Pentech Press, London, pp. 579—610.
HOBBS, N.B. (1986) Mire morphology and the properties and behaviour of some British and foreign peats', Q. *J. Eng. Geol.,* 19 (1), 7—80.
HOEK, E. and BRAY, J.W. (1974) *Rock Slope Engineering,* Institution of Mining and Metallurgy, London.
HOEK, E. and BRAY, J.W. (1981) *Rock Slope Engineering,* 3rd edn, Institution of Mining and Metallurgy, London, 358pp.
HOEK, E. and BROWN, E.T. (1980) *Underground Excavations in Rock,* Institution of Mining and Metallurgy, London.
HOLM, G. and HOLTZ, R.D. (1977) 'A study of large diameter piston samplers', *Proc. Speciality Session 2 on Soil Sampling, 9th Int. Conf. Soil Mech. and Found. Eng.,* Tokyo, pp. 73—78.
HONG, R.C. (1961) 'The Hong Piston Sampler', *Proc. 5th Int. Conf. Soil Mech. and Found. Eng.,* Paris, Volume 1, Part 2, pp. 471—474.
HOOPER, W. and MCDOWELL, P.W. (1977) 'Magnetic surveying for buried mine shafts and wells', *Ground Engineering,* 10 (2), 21—23.
HOPPER, R.J. (1992) 'The effects and implications of sampling clay soils', PhD thesis, University of Surrey.
HUCKA, V. (1965) 'A rapid method of determining the strength of rocks in situ', *Int. J. Rock Mech. Mi Sci.,* 2, 127—134.
HUIZINGA, T.K. (1944) 'Tien Jare Grondmechanica in Nederland', *Weg en Waterbouw,No.* 1—2, August 1944.
HVORSLEV, M.J. (1940) 'Preliminary draft report on the present state-of-the-art of obtaining undisturbed samples of soils', *Supplement to Proc. Purdue Conf. On Soil Mechanics and its Applications,* Purdue University, Lafayette, Indiana, USA.
HVORSLEY, M .J. (1949) *Subsurface Exploration and Sapling of Soils for Civil Engineering Purposes,* Waterways Experimental Station, Vicksburg, USA.
HVORSLEV M.J. (1951) 'Time lag and soil permeability in ground water observations', *Bull. No. 36,* Waterways Experimental station, Vicksburg, USA.
IDEL, K.H., MUHS, H. and VON SOOS, P. (1969) 'Proposal for "Quality Classes" in soil sampling in relation to boring methods and sampling equipment', *Proc. Speciality Session 1 on Soil Sampling, 7th Int. Conf. Soil Mech. and Found. Eng.,* Mexico, pp. 11—14.
International Society for Soil Mechanics and Foundation Engineering (1965) 'Report of the subcommittee on problems and practices of soil sampling', *Proc. 6th Int. Conf. Mech. and Found. Eng.,* Montreal, Volume 3, Appendix II, pp. 64—71.
International Society for Soil Mechanics and Foundation Engineering (1977) 'Report of the subcommittee on the penetration test for use in Europe', available from the Secretary General, ISSMFE.
IRELAND, H.O., MORETTO, 0. and VAROAS M. (1970) 'The dynamic penetration test: A standard that is not standardized', *Géotechnique,* 20 (2), 185—192.
IRWIN, M.J. (1967) 'A mercury-filled gauge for measuring the settlement of foundations', *Road*

Research Laboratory Report LR62, Road Research Laboratory, Crowthorne, Berks.
I5IHARA, K. and SILVER, ML. (1977) 'Large diameter and sampling to provide specimens for liquefaction testing', *Proc. Specialty Session 2 on Soil Sampling, 9th Int. Conf. Soil Mech. and Found. Eng.,* Tokyo, pp. 1—6.
IWASAKI, Y.T., HA5HIM0T0, T., HONGO, T., HIRAYAMA, H. and MURIKAMI, S. (1977) 'On the
undisturbed sampling for stiff clay', *Proc. Speciality Session 2 on Soil Sampling, 9th Int. Conf. Soil. Mech. and Found. Eng.,* Tokyo, pp. 57—62.
JACKSON, D.D. (1972) 'Interpretation of inaccurate, insufficient and inconsistent data', *Geophys. J. R. Abstr. Soc.,* 28, 97—109.
JARDINE, R.J., SYMES, M.J. and BURLAND, J.B. (1984) 'The measurement of soil stiffness in the triaxial apparatus', *Géotechnique,* 34 (3), 323—340.
JAMIOLKOWSKI, M., LADD, C.C., GERMAINE, J.T. and LANCELLOTTA, R. (1985) 'New developments in field and laboratory testing of soils', *Proc. 11th Int. Conf. Soil Mech. and Found. Eng.,* San Francisco, Volume 1, Balkema, Rotterdam, pp. 57—153.
JENSEN, M. (1969) *Civil Engineering around 1700,* Danish Technical Press, Copenhagen.
JOHNSON, H.L. (1940) 'Improved sampler and sampling technique for cohesionless soils', *Civil Engineering,* 10, pp. 346—348.
JOHNSON, R.B. (1954) 'Use of the refraction method for differentiating pleistocene deposits in Arcola and Tuscola Quadrangles, Illinois', *Illinois Geol. Surv. Report Investigation 176,* Illinois Geological Survey.
KACZMARZ, M.S. (1934) Angenaherte Auflosung von Systemen linearer Gleichungen, *Bulletin International de l'Academie Polonaise des Sciences et des Lettres,* Classe des Sciences Mathematiques et Naturelles, serie A Science Mathematiques, pp. 335—337.
KALLSTENIUS, T. (1958) 'Mechanical disturbances in clay samples taken with piston samplers', *Proc. Roy. Swed. Geot. Inst.,* No. 16.
KALLSTENIUS, T. (1961) 'A standard piston sampler prototype', *Proc. Roy. Swed. Geot. Inst.,* No. 19, pp. 38—44.
KAROL, R.H. (1970) 'Use of chemical grouts to sample sands', *Proc. ASTM Symp. on Sampling Soil and Rock', ASTM Spec. Tech. Pub. No. 483,* ASTM, pp. 5 1—59.
KEE, R. (1974) 'The behaviour and design of foundations in chalk', MPhil Thesis, Hatfield Polytechnic.
KENNEY, T.C. and WATSON, G.H. (1961) 'Multi-stage triaxial test for determining c' and 0 of saturated soils', *Proc. 5th.Int. Conf. Soil Mech. and Found. Eng.,* Paris, Volume 1, pp.
191—195.
KENNIE, T.J.M. and MATTHEWS, M.C. (1985) *Remote Sensing in Civil Engineering,* Surrey University Press, Glasgow, 3S'lpp.
KILFORD, W.K. (1973) *Elementary Air Survey,* 3rd edn, Pitman, London.
KIMBALL, W.P. (1936) 'Settlement records of the Mississippi River Bridge at New Orleans', *Proc. 1st Int. Conf. Soil Mech. and Found. Eng.,* Harvard, Volume 1, pp.
85—92.
KJELLMAN, W. (1938) 'Report and drawings on a new piston type sampler, to the Committee on Sampling and Testing, Soil Mechanics and Foundations Division, American Society of Civil Engineers', see Hvorslev (1949).
KJELLMAN, W., KALLSTENIUS, T. and WAGER, 0. (1950) 'Soil sampler with metal foils', *Proc. Roy. Swed. Geot. Inst.,* No. 1.
KNILL, J.L. (1971) *Collecting and Processing of Geological Data for Purposes of Rock Engineering: The Analysis and Design of Rock Slopes,* University of Alberta, Edmonton, Canada.
KOVACS, W.D., EVANS, J.C. and GRIFFITH, A.H. (1977) 'Towards a more standardised SPT', *Proc. 9th Int. Conf. Soil Mech. and Found. Eng.,* Tokyo, Volume 2, 269—276.
KRAWCZYK, J.V. (1969) 'An examination of methods of determining the specific gravity of soils', *Road Research Laboratory Report LR272,* Crowthorne, Berks.
KUHL, D.E. and EDWARDS, R.Q. (1963) 'Image separation radioisotope scanning', *Radiology,* 8, 653—662.
LACASSE, S. and LUNNE, T. (1986) 'Dilatometer tests in sands', *Proc. ASCE Symp. on Use of In-*

Situ Tests in Geotech. Eng., pp. 686—699.
La Compagnie Générale de Géophysique (1963) *Aba que de sondage electrique,* European Association of Exploration Geophysicists, The Hague.
LADANYI, B. (1972) 'In situ determination of undrained stress-strain behaviour of sensitive clays with the pressuremeter', *Canadian Geot. J., 9,* 313—319.
LAKE, L.M. and SIM0N5, N.E. (1970) 'Investigations into the engineering properties of chalk at Welford Theale, Berkshire', *Proc. Conf. on In situ Investigations into Soils and Rocks,* British Geotechnical Society, London, pp. 23—29.
LAKE, L.M. and SIMONS, N.E. (1975) 'Some observations on the settlement of a four storey building founded on chalk at Basingstoke, Hampshire', *Proc. Conf. on Settlement of Structures,* BGS Cambridge, Pentech Press, London, pp. 283—291.
LAMBE, T.W. (1951) *Soil Testing for Engineers,* John Wiley, New York.
LAMBE, T.W. (1967) 'Stress path method', *J. Soil Mech. and Found. Eng. Div. Am. Soc. Civ. Engrs.,* 93 (SM9), 309—33 1.
LANCASTER-JONES, P.F.F. (1975) 'The interpretation of the luge on water test', Q. *J. Eng. Geol., 8,* 151—1 54.
LA ROCHELLE, P. (1960) 'The short term stability of slopes in London clay', PhD thesis, University of London.
LA ROCHELLE, P., Roy, M. and TAVERNAS, F. (1973) 'Field measurements of cohesion in Champlain clays', *Proc. 8th Int. Conf. Soil Mech. and Found Eng.,* Moscow, Volume 1.1, pp. 229—236.
LA ROCHELLE, P., LEROUEIL, S. and TAVENAS, F. (1987) 'Discussion on "Strain Path Method"', *Proc. ASCE, J. Geot. Eng. Div.,* 113 (9), 1088—1090.
LA ROCHELLE, P., SARRAILH J., TAVENAS, F., Roy, M. and LEROUEIL, S. (1981) Causes of sampling disturbance and design of a new sampler for sensitive soils. Can. Geotech. J. 18, No. 1, 52—66.
LEFEBVRE, G. and P0uLIN, C. (1979) 'A new method of sampling in sensitive clay', *Can. Geotech. J.,* 16 (1), 226—233.
LEGGO, P.J. and LEECH, C. (1983) 'Sub-surface investigation for shallow mine workings and cavities by the ground impulse radar technique', *Ground Engineering,* 16 (1), 20—23.
LE GRAND, SUTCLIFFE and GELL (1934) 'Tool for producing true cores of clay', *Engineering (London),* 138, 23.
LEONARDS, G.A. and FROST, J.D. (1988) 'Settlement of shallow foundations on granular soils', *Proc. ASCE, J. Geot. Eng.,* 114 (GT7), 79 1—809.
LEVY, J.F. and MORTON, K. (1975) 'Loading tests and settlement observations on granular soils', *Proc. Conf on Settlement of Structures,* BGS Cambridge, Pentech Press, London, *pp. 43—52.*
LIA0, S. and WHITMAN, R.V. (1985) 'Overburden correction factors for SPT in sand', *Proc. ASCE, J. Geot, Eng. Div.,* 112 (3), 373 377.
LILLESAND, TM. and KIEFER, R.W. (1979) *Remote Sensing and Image Interpretation,* John Wiley, New York.
LINEHAM, D. and MURPHY, V.J. (1962) 'Engineering seismology applications in metropolitan areas', *Geophysics,* 27 (2), 213—20.
LITTLE, AL., STEWART, J.C. and FOOKES, P.J. (1963) 'Bedrock grouting tests at Mangla Dam, West Pakistan', *Proc. Symp. on Grouts and Drilling Muds in Engineering Practice,* Butterworths, London, pp. 9 1—97.
LONGSDON, A.E.C. *(1945)* 'An improved clay sampler', *The Engineer (London),* 18 (3 August), 97.
LOWE, J. (1960) 'Current practice in soil sampling in the United States', *Highway Res. Board, Special Report 60,* Highway Research Board, Washington, DC.
LUGEON, M. (1933) *Barrages et Géologie,* Dunod, Paris.
LUNNE, T., LACA5SE, S., RAD, N.S. and DECOURT, L. (1989) *SPT, CPT, Pressuremeter Testing and Recent Developments on In-Situ Testing,* Norwegian Geotechnics Institute Report No. 591390-1.
LUNNE, T., POWELL, J.J.M., HAUGE, E.A., UGLOW, TM. and MOKKELBO5T, K.H. (1990) 'Correlation of Dilatometer Readings to Lateral Stress', *69th Annual Meeting of the Transportation Research Board,* Washington, USA.

LUPINI, L.F., SKINNER, A.E. and VAUGHAN, P.R. (1981) 'The drained residual strength of cohesive soils', *Géotechnique,* 31(2), 181—213.
LUTENEGGER, A.J. (1988) 'Current status of the Marchetti Dilatometer Test', *Proc. 1st Int. Symp. on Penetration Testing, ISOPT-1,* Orlando, Florida, Volume 1, pp. 137—*155.*
LUTENEGGER, A.J. and KABIR, M. (1988) 'Dilatometer C-Reading to help determine stratigraphy', *Proc. 1st. Int. Symp. on Penetration Testing, ISOPT-1,* Orlando, Florida, Volume 1, pp. *549—554.*
MCCANN, D.N. and TAYLOR-SMIT, D. (1973) 'Geotechnical mapping of sea floor sediments by the use of geophysical techniques', *Trans. Inst. Mm. Met.,* 81B, 155 67.
MCDOWELL, P.W. *(1975)* 'Detection of clay filled sink-holes in chalk by geophysical methods', Q. *J. Eng. Geol.,* 8, 303—310.
MCGOWN, A. and DERBYSHIRE, F. (1977) 'Genetic influence on the properties of tills', Q. *J. Eng. Geol,* 10, 389—410.
MCGOWN, A. and ILEY, P. (1973) 'A comparison of data from agricultural soil surveys with engineering investigations for roadworks in Ayrshire', *J. Soil Sci.,* 24 (2), *145* 56.
MCGOWN, A., SALDIVAR-SALI, A. and RADWAN, AM. (1974) 'Patterns and slope failures in till at Hurlford, Ayrshire', Q. *J. Eng. Geol., 7,* 1—26.
MCLEAN, A.C. and GRIBBLE, C.D. *(1985) Geology for Civil Engineers,* 2nd edn, Unwin Hyman, London, 314 pp.
MAEDA, K. *(1955)* 'Apparent resistivity for dipping beds', *Geophysics,* 20 (1), 123—47.
MAIR, R.J. and WOOD, D.M. (1987) *In-Situ Pressuremeter Testing: Methods Testing and Interpretation,* CIRIA Ground Engineering Report, Butterworths, London.
MALLARD, D.J. (1977) 'Discussion: Session 1 — Chalk', *Proc. ICE Conf. on Piles in Weak Rock, pp.* 177—180.
MALLARD, D.J. (1983) 'Testing for Liquefaction Potential', *Proc. NATO Workshop on Seismicity and Seismic Risk in the Offshore North Sea Area,* Utrecht, Reidel, Dordrecht, *pp.* 289—302.
MARCHETTI, S. (1980) 'In-situ tests by flat dilatometer', *Proc. ASCE, J. of Geot. Eng.,* Volume 106, pp. 299—321.
MARCHETTI, S. (1985) 'On the field termination of *k0* in sand, Panel Presentation: In- situ testing techniques, Proc. 11th Int. Conf. on Soil Mech. and Found. Eng., San Francisco.
MARCHETTI, S. (1991) Written communication, cited in BRIAND and MIRAN (1992).
MARCHETTI, S. and CRAPPS, D.K. (1981) *Flat Dilatometer Manual,* GPE Inc., USA.
MARGA50N, G., IRWIN, M.J. and HUXLEY, M.A. (1968) 'The effect of tides on subsoil pore-water pressures at a site on the proposed Shoreham by pass', *Road Research Laboratory Report No. LR195,* Road Research Laboratory, Crowthorne, Berks.
MARKLAND, J. (1972) A useful technique for estimating the stability of a slope when the rigid wedge slide type of failure is expected', *Imperial College Research Report No. 19,* Imperial College, London.
MARSLAND, A. (1972) 'Clays subjected to in situ plate tests', *Ground Engineering,* 5 (6), 24—31.
MARSLAND, A. (1986) 'The choice of test methods in site investigation', in A.B. Hawkins (ed.) *Site Investigation Practice: Assessing BS 5930,* Eng. Geol. Special Pub. No. 2, Geological Society, pp. 289—298.
MARSLAND, A. and EA50N, B.J. (1973) 'Measurements of the displacements in the ground below loaded plates in deep boreholes', *British Geotech. Soc. Symp. on Field Instrumentation in Geot. Eng.,* Volume 1, pp. 304—317.
MARSLAND, A. and QUARTERMAIN, R. (1974) 'Further developments of multipoint magnetic extensometers for use in highly compressible ground', *Géotechnique,* 24, 429—433.
MARSLAND, A. and RANDOLPH, M.F. (1977) 'Comparison of the results from pressuremeter tests and large in situ plate tests in London clay', *Géotechnique,* 27, 217—243.
MASON, R. (1978) *Petrology of the Metamorphic Rocks,* George Allen & Unwin, London.
MATHERSON, GD. (1983) 'Rock stability assessment in preliminary site investigations — graphical methods', *TRRL Report LR 1039,* Transport Research Laboratory, Crowthorne, Berks.
MATHESON, J.L. (1939) 'An aerial survey of the estuary of the River Dee, employing a simple method of rectifying oblique photographs', *J. Inst. Civ. Eng.,* 10 (Paper 5157),

47—54.
MATTHEWS, MC. (1993) ‘The mass compressibility of fractured chalk’, PhD thesis, University of Surrey.
MAXWELL, G.M. (1976) ‘Old mineshafts and their location by geophysical surveying’, Q. *J. Eng. Geol.*, 9 (4), 283—90.
MAYNIEL, K. (1808) *Traité Experimental, Analytique et Pratique de Ia Poussee des Terres et des Murs de Revetment,* D. Colas, Paris.
MAZIER, G. (1974) ‘Methodes de prélêment des sols meubles’, *Annales de l’Institut Technique du Batiment et des Travaux Publics,* July—August, pp. 75—85.
MEIGH, A.C. (1987) ‘Cone penetration testing: methods and interpretation’, *CIRIA Ground Engineering Report: In-situ testing,* p. 141, Construction Industries Research and Information Association, London.
MEIGH, A.C. and GREENLAND, S.W. *(1965)* ‘In situ testing of soft rocks’, *Proc. 6th Int. Conf Soil Mech. and Found. Eng.,* Montreal, Volume 1, pp. 73—76.
MENARD, L. (1957) ‘Mésures in situ des proprietés physique des sols’, *Annales des Fonts et Chaussées,* 127, 357—377.
MENZIES, B.K. and MAILEY, L.K. (1976) ‘Some measurements of strength anisotropy in soft clays using diamond shaped shear vanes’, *Géotechnique,* 26 (3), 535—538.
MERRIFIELD, C.M. (1980) ‘Factors affecting the interpretation of the in-situ shear vane test’, PhD thesis, University of Surrey.
MILITITSKY, J., CLAYTON, CRI., TALBOT, J.C.S. and DIKRAN, S. (1982) ‘Previsao de recalques em solas granulares ulitizando resultados de SPT: revisao critica’, *Proc. 7th Brazilian Conf Soil Mech. and Found. Eng.,* pp. 133—150.
MILLER, V.C. (1961) *Photogeology,* McGraw-Hill, New York.
MITCHELL, J.K., GUZIKOWSKI, F. and VILLET, W.C.B. (1978) *The measurement of soil properties in situ, present methods — their applicability and potential,* U.S. Dept. of Energy Report, Dept. of Civil Engineering, Univ. of California, Berkeley.
MOHR, HA. (1937) ‘Exploration of soil conditions and sampling operations’, *Harvard Soil Mechanics Series No. 21,* pp. 63.
MOHR, HA. (1943) ‘Exploration of soil conditions and sampling operations’, *Soil. Mech. Series No. 21,* Graduate School of Engineering, Harvard.
MOHR, HA. (1966) ‘Discussion on Fletcher (1965)’, *Proc. ASCE, J. Soil Mech. Found. Div.,* Volume 92, SMI, pp. 196—199.
MOLLARD, J.D. (1962) ‘Photo analysis and interpretation in engineering geology investigations: A Review’, in T. Fluhr and R.F. Legget (eds), *Reviews in Engineering Geology,* The Geological Society of America, New York, pp. 105—127.
MOONEY, H.M. and ORELLANA, E. (1966) *Master Tables and Curves for Vertical Electrical Sounding over Layered Structures,* Interciencia, Madrid.
MOONEY, H.M. and WETZEL, W.W. (1956) *The Potentials about a Point Electrode and Apparent Resistivity Curves for a Two-, Three- and Four-layer Earth,* University of Minnesota Press, Minneapolis.
MOORE, J.R,, ECHARD, J.D. and NEIL, G.G. (1980) ‘Radar detection of voids under concrete highways’, *IEEE Int. Radar Conf., CH 1493-6/80/0000-0131.*
MORGENSTERN, N.R. and TCFJALENKO, J.S. (1967) ‘Microscopic structures in Kaolin subjected to direct shear’, *Géotechnique,* 17, 309—328.
MORGENSTERN, N.R. and VAUGHAN, P.R. (1963) ‘Some observations on allowable grouting pressures’, *Proc. Symp. on Grouts and Drilling Muds in Engineering Practice,* Butterworths, London, pp. 36—42.
M0RI, H. (1977) ‘Method to take undisturbed samples using thin wall sampler with stationary piston: draft standard of the Japanese Society of Soil Mechanics and Foundation Engineering’, *Proc. Speciality Session in Soil Sampling, 9th Int. Conf. Soil Mech. and Found. Eng.,* Tokyo, Appendix to Discussion Volume, pp. 49—52.
MUIR WOOD, A.M. and CASTE, G. (1970) ‘In situ testing for the Channel Tunnel’, *Proc. Conf on In-situ Investigations in Soils and Rocks,* BGS, London, pp. 109—116.
MUKABE, J.N., TATSUOKA, F, and HIROSE, K. (1991) ‘Effect of Strain rate on small strain

stiffness of kaolin', *Proc. 26th Japanese National Conf. on Soil Mech. and Found. Eng.,* Nagano, pp. 659—662.
MULLER, L. (1959) 'The European approach to slope stability problems in open pit mines', Proc. 3rd Symp. Rock Mech., *Colorado School Mines Quarterly,* 54, 114—133.
NEUMANN-DENZAU, G. and BEHRENS, J. (1984) Inversion of seismic data using tomographical reconstruction techniques for investigations of laterally inhomogeneous media', *Geophys. J. R. Abstr. Soc., 79, 305—315.*
NIXON, I.K. (1954) 'Some investigations on granular soils with particular reference to the compressed-air sand sampler', *Géo technique, 4,* 16—31.
NORBURY, D.R., CHILD, G.H. & SPINK, T.W. (1986) 'A critical review of Section 8 (BS *5930)* — soil and rock description', in A.B. Hawkins (ed.) *Site Investigation Practice: Assessing BS 5930,* Eng. Geol. Special Pub. No. 2, Geological Society, pp. 33 1—342.
NORMAN, J.W. (1968a) 'Photogeology of linear features in areas covered with superficial deposits', *Trans. Inst. Mm. Met, 77* B, 60—77.
NORMAN, J.W. (1968b) 'The air photograph requirements of geologists', *Photogrammetric Record, 6* (32), 133—49.
NORMAN, J.W. (1969) 'Photo-interpretation of boulder clay areas as an aid to engineering geological studies', *Q.J. Eng. Geol.,* 2, 149—57.
NORMAN, J.W. (1970) 'The photogeological detection of unstable ground', *J. Inst. Highways Engnrs.,* 17 (2), 19—22.
NORMAN, J.W., LEISOWITZ, T.H. and FOOKE5, PG. (1975) 'Factors affecting the detection of slope instability with air photographs in an area near Sevenoaks, Kent', *Q. J. Eng. Geol,* 8 (3), 159—76.
OLSSON, J. (1925) 'Kolvborr, ny borrtyp for upptagning ar lerprov', *Teknisk Tidskrfl,* 55, 13—16.
OLSSON, J. (1936) 'Method for taking earth samples with the most natural consistency', *2nd Congress on Large Dams,* Washington DC, Volume 4, pp. 157—161.
OSTERBERG, J.O. (1952) 'New piston type soil sampler', *Eng. News Record,* 148 (17), 77—78.
OSTERBERG, J.O. (1973) 'An improved hydraulic piston sampler', *Proc. 8th Int. Conf Soil Mech. and Found. Eng.,* Moscow, Volume 1.2, pp. 317—321.
PACHER, F. (1959) 'Kennziffern des flachengefuges', *Geologies and Bauwesen,* 24, 223—227.
PAHL, P.J. (1981) 'Estimating the mean length of discontinuity traces', *Int. J. Rock Mech. Mi Sci. and Geomech. Abstr.,* 18, 221—228.
PALMER, A.C. (1972) 'Undrained plane-strain expansion of a cylindrical cavity in clay; a simple interpretation of the pressuremeter test', Géotechnique, 22 (3), 451—457.
PALMER, D.J. and STUART, J.G. (1957) 'Some observations on the standard penetration test and a correlation of the test with a new penetrometer', *Proc. 4th Int. Conf. Soil Mech. and Found. Eng.,* London, Volume 1, pp. 231—236.
PARRY, R.H.G. (1963) 'Testing small undisturbed samples', *4th Australian—New Zealand Conf on Soil Mech. and Found. Eng.,* pp. 6 1—68.
PARRY, R.H.G. (1968) 'Field and laboratory behaviour of a lightly overconsidered clay', *Géotechnique,* 18 (2), 151—171.
PARRY, R.H.G. and NADARAJAH, V. (1973) 'Multistage triaxial testing of lightly over-consolidated clays', *J. Testing and Evaluation,* 1 (5), 374—381.
PEARSON, R. and MONEY, MS. (1977) 'Improvements in the Lugeon or Packer permeability test', *Q. J. Eng. Geol.,* 10, 222—239.
PECK, RB. (1943) 'Earth pressure measurements in open cuts, Chicago subway', *Trans. ASCE,* 108, 1008—36.
PECK, R.B. (1969) 'Advantages and limitations of the observational method in applied soil mechanics: 9th Rankine lecture', *Géotechnique,* 19 (2), 171—87.
PECK, R.B., HANSON, WE, and THORBURN, T.H. (1974) *Foundation Engineering,* John Wiley, New York.
PENMAN, A.D.M. (1956) 'A field piezometer apparatus', *Géotechnique, 4* (2), *57—65.* PENMAN, A.D.M. (1961) 'A study of the response times of various types of piezometers', *Proc. Conf. on Pore*

Pressures and Suction in Soils, Butterworths, London, pp. 53—58.
PENMAN, A.D.M. (1972) 'Instrumentation for embankment dams subjected to rapid drawdown', *Building Research Station, Current Paper CP1/72,* Building Research Station.
PENMAN, A.D.M. and MITCHELL, P.B. (1970) 'Initial behaviour of Scammonden Dam', *Trans 10th Int. Congress on Large Dams,* Montreal, Volume 1, pp. 723—747.
PERLOW, M. and RICHARDS, A.F. (1977) 'Influence of shear velocity on vane shear strength', *Proc. ASCE, J. Geotech. Eng. Div.,* 103 (GT1), pp. 19—32.
PERRIN, R,M.S. (1963) 'The use of air photographs in the study of patterned ground in East Anglia', *Photogrammetria,* 14, 183—88.
PERRIN, R.M.S. (1977) 'Air photography and soil science', in J.K.S. St Joseph (ed), *The Uses of Air Photography,* 2nd edn, John Baker, London, pp. 66—85.
PETERSON, J.E., PAULSSON, B.N.P. and MCEVILLY, T.V. *(1985)* 'Applications of ART to crosshole seismic data', *Geophysics,* 50 (10), 1566—1580.
PETLEY, D.J. (1966) 'The shear strength of soils at large strains', PhD thesis, University of London.
PETTERSON, K.E. (1933) 'Förbättrad apparat for upptagning av lerprov (jordprov)', *Tekn. Ts.,* 63 (H.8), 85—87.
PHILLIPS, F.C. (1971) *The Use of Stereographic Projection in Structural Geology,* Edward Arnold, London.
PINCHES, G.M. and THOMPSON, R.P. (1990) 'Crosshole and downhole seismic surveys in the UK Trias and Lias', in F.G. Bell, M.G. Culshaw, J.C. Cripps and J.R. Coffey (eds) *Field Testing in Engineering Geology,* Eng. Geol. Special Pub. No. 6, Geological Society, pp. 299—308.
PITEAU, DR. (1970) 'Geological factors significant to the stability of slopes cut in rock', *Proc. Symp. Planning Open Pit Mines,* Johannesburg. A.A. Balkema, Amsterdam, pp. 33—53.
PITEAU, DR. (1973) 'Characterizing and extrapolating rock joint properties in engineering practice', *Rock Mechanics Supplement,* 2, 5—31.
PORTER, O.J. (1936) 'New soil sampler for deep tests', *Eng. News Record,* 116, 804—805.
PORTER, O.J. (1937) *Drilling Cost Cut by New Rig for Foundation Investigations,* California Highways and Public Works.
PORTER, O.J. (1938) 'The preparation of subgrades', *Proc. Highway Res. Board,* 18 (2), 324—331.
PORTER, O.J. (1942) 'Foundations for flexible pavements', *Proc. Highway Res. Board,* 22, 100—136.
POULOS, H.G. (1989) 'Pile behaviour — theory and application: 29th Rankine Lecture', *Géotechnique,* 39 (3), 363—416.
POWELL, J.J.M. and UGLOW, I.M. (1988) 'Marchetti dilatometer testing in UK Soils', *Proc. 1st Int. Symp on Penetration Testing, ISOPT-1,* Orlando, Florida, Volume 1, pp. *555—* 562.
PRIEST, S.D. (1980) 'The use of inclined hemisphere projection methods for the determination of kinematic feasibility, slide direction and volume of rock blocks', *Int. J. Rock Mech. Mi Sci. and Geomech. Abstr.,* 17, 1—23.
PRIEST, S.D. (1985) *Hemispherical Projection Methods in Rock Mechanics,* George Allen and Unwin, London.
PRIEST, S.D. (1993) *Discontinuity Analysis for Rock Engineering,* Chapman and Hall, London, 473pp.
PRIEST, S.D. and HUDSON, J.A. (1976) 'Discontinuity spacings in rock', *Int. J. Rock Mech. Mm. Sci. and Geomech. Abstr.,* 13, 135—148.
PRIEST, S.D. and HUDSON, J.A. (1981) 'Estimation of discontinuity spacing and trace length using scanline surveys', *Int. J. Rock Mech. Mi Sci. and Geomech. Abstr.,* 18, 183—197.
PROCTOR, R.R. (1933) 'The design and construction of rolled earth dams', *Eng. News Record,* 3 (9), 245—248; (10), 216—219; (12), 348—351; (13), 372—376.
PUGH, R.S. (1978) 'The strength and deformation characteristics of a soft alluvial clay under full scale loading conditions', PhD thesis, University of London.

RADON, J. (1917) 'Uber die Bestimmung von Funktionen durch ihre Intergralwerte langs gewisser Mannigfaltigkeiten', *Berichte Sdchissche Akademie der Wissenschaften, Liepzig, Math.-Phys. K].*, 69, 262— 277. [English translation in S.R. Deans (1983) *The Radon transform and some of its applications*, John Wiley, New York.
RAY, R.G. (1960) *Aerial Photographs in Geologic Interpretation and Mapping*, US Geological Survey Professional Paper 373, Washington, DC.
RAY, R.G. and FISCHER, WA. (1960) 'Quantitative photogeology — a geologic research tool', *Photogrammetric Engineering*, 26, 146—47.
RAYBOULD, DR. and PRICE, D.G. (1966) 'The use of the proton magnetometer in engineering geological investigations', *Proc. 1st Congr. Int. Soc. Rock Mech.*, Lisbon, Volume 1, pp. 11—14.
RAYMOND, G.P. (1977) 'Effect on soil sampling on the drained properties of Leda Clay', *Proc. Speciality Session 2 on Soil Sampling, 9th Int. Conf. Soil Mech. and Found. Eng.*, Tokyo, pp. 47—55.
RAYMOND, G.P., TOWNSEND, D.L. and LOJKA5EK, M.J. (1971) 'Evaluation of sample quality on undrained soil properties', *Proc. Speciality Session on Quality in Soil Sampling, 4th Asian Regional Conf, Int. Soc. Soil Mech. and Found. Eng*, Bangkok, pp. 88—92.
REDFORD, P.R. (1981) 'Site investigation practices for hard rock tunnel design', MSc dissertation, University of Surrey.
RIGGS, C.O. (1986) 'North American Standard Penetration Test Practice — an essay', *Proc. ASCE Conf on use of in situ tests in geot. eng.*, Blacksburg, VA, pp. 949—965.
Road Research Laboratory (1952) *Soil Mechanics for Road Engineers*, HMSO, London. Road Research Laboratory (1954) *Technical Paper No. 15: Soil Survey Procedures*, HMSO, London.
Road Research Laboratory (1970) *Road Note 29: A Guide to the Structural Design of Pavements for New Roads*, Road Research Laboratory, Dept. of the Environment, HMSO, London.
ROBERTSON, A. MACG. (1970) 'The interpretation of geological factors for use in slope theory', *Proc. Symp. Planning Open Pit Mines*, Johannesburg, A.A. Balkema, Amsterdam, pp. 57—77.
ROBERTSON, P.K., CAMPANELLA, R.G., and GILLESPIE, D. (1988) 'Excess pore pressure and the flat dilatometer test', *Proc. 1st Int. Symp. on Penetration Testing, ISOPT-1*, Orlando, Florida, Volume 1, pp. 567—576.
RODIN, 5. (1949) 'Discussion on Harding (1949)', *J. Inst. Civ. Eng.*, 32, 111—137.
ROGUE, E., JANBU, N. and SENNESET, K. (1988) 'Basic interpretation procedures of flat dilatometer tests', *Proc. 1st. Int. Symp. on Penetration Testing, ISOPT-1*, Orlando, Florida, Volume 1, pp. 577—587.
ROWE, P.W. (l968a) 'The influence of geological features in clay deposits on the design and performance of sand drains', *Proc. Inst. Civ. Engnrs., London, Suppl. Paper 7058 S.*
ROWE, P.W. (1 968b) 'Failure of foundations and slopes on layered deposits in relation to site investigation practice', *Proc. Inst. Civ. Eng., Paper 7057 S.*
ROWE, P.W. (1971) 'Representative sampling in location, quality and size', *Proc. Symp. on Sampling of Soil and Rock*, Toronto, *ASTM Spec. Tech. Pub. No. 483*, pp. 77—106.
ROWE, P.W. (1972) 'The relevance of soil fabric to site investigation practice: 12th Rankine Lecture', *Géotechnique*, 22 (2), 195—300.
ROWE, P.W. and BARDEN, L. (1964) 'The importance of free ends in triaxial testing', *Proc. ASCE, J. Soil Mech. and Found. Engng. Div.*, 90 (SM 1), 1—27.
ROWE, P.W. and BARDEN, L. (1966) 'A new consolidation cell' *Géotechnique*, 16 (2), 162—170.
ST JOSEPH, J.K.S. (1957) 'A survey of pioneering in air photography', in W.F. Grimes (ed.), *Aspects of Archaeology*, pp. 305—3 15.
ST JOSEPH, J.K.S. (1977) *The Uses of Air Photography*, 2nd edn, John Baker, London.
SANGERAT, G. (1972) *The Penetrometer and Soil Exploration*, Elsevier-North Holland.
SCARROW, J.A. and GOSLING, R.C. (1986) 'An example of rotary core drilling in soils', in A.B. Hawkins (ed.) *Site Investigation Practice: Assessing BS 5930*, Eng. Geol. Special Pub. No. 2, Geological Society.
SCHJETNE, K. (1971) 'The measurement of pore pressure during sampling', *Proc. Speciality Session on Quality in Soil Sampling, 4th Asian Regional Conf., Int. Soc. Soil Mech. and Found. Eng.*, Bangkok, pp. 12—16.
SCHMERTMANN, J.H. (1955) 'The undisturbed consolidation behaviour of clay', *Trans. ASCE*, 120,

1201—1233.
SCHMERTMANN, J.H. (1970) ‘Static cone to compute static settlement over sand’, *Proc. ASCE, J. Soil Mech. and Found. Engng. Div.,* 98 (SM3), 1011—1043.
SCHMERTMANN, J.H. (1972) ‘Effects of in situ lateral stress on friction cone penetrometer data in sand’, *Fugro Sonder Symposium,* Fugro-Cesco, Holland, pp. 37—39.
SCHMERTMANN, J.H. (1982) ‘A method for determining the friction angle in sands from the Marchetti Dilatometer Tests’, *Proc. 2nd Eur. Symp. on Penetration Testing, ESOPT-Il,* Amsterdam, Volume 2, pp. 853—861.
SCHMERTMANN, J.H. (1983) ‘Revised procedures for circulation *K0* and OCR from DMT’s with ID> 1.2 and which incorporates the penetration measurement to permit calculating the plane strain angle of friction’, *DMT Digest No. 1,* GPE Inc., USA.
SCHMERTMANN, J.H. (1986) ‘Suggested method for performing the flat dilatometer test’, *ASTM Geot. Testing J., 9* (2), 93—101.
SCHMERTMANN, J.H. and PALACI05, A. (1979) ‘Energy dynamics of SPT’, *Proc. ASCE, J. Geot. Eng. Div.,* GTB, 105, 909—926.
SCHULTZE, E. and SHERIF, G. (1973) ‘Prediction of settlements from evaluated settlement observations for sands’, *Proc. 8th Int. Conf Soil Mech. Found. Eng.,* Moscow, Volume 1, 3, pp. 225—230.
SEED, H.B., TOKIMATSU, K., HARDER, L.F. and CHUNG, R.M. (1985) ‘Influence of SPT procedures in soil liquefaction resistance evaluations’, *Proc. ASCE, J. Geot. Eng.,* Dec., 1425—1445.
SEKO, R. and TOBE, K. (1977) ‘Research of stiff clay sampling’, *Proc. Speciality Session 2 on Soil Sampling, 9th Int. Conf. Soil Mech. and Found. Eng.,* Tokyo, pp. 44—45.
SEROTA, S. and JENNINGS, R.A. (1957) ‘Undisturbed sampling techniques for sands and very soft clays’, *Proc. 4th Int. Conf. Soil Mech. and Found. Eng.,* London, Volume 1, pp. 245—248.
SHERWOOD, P.T. (1970) ‘The reproducibility of the results of soil classification and compaction tests’, *Road Research Laboratory Report No. LR 339,* Road Research Laboratory, Crowthorne, Berks.
SHERWOOD, P.T. and RYLEY, D.M. (1968) ‘An examination of cone-penetration methods for determining the liquid limit of soils’, *Road Research Laboratory Report No. LR339,* Road Research Laboratory, Crowthorne, Berks.
SIDDIQUE, A. (1990) ‘A numerical and experimental study of sampling disturbance’, PhD thesis, University of Surrey.
SIM0NS, N.E. (1971) ‘The stress path method of settlement analysis applied to London Clay’, *Proc. Roscoe Mem. Symp. on Stress—Strain Behaviour of Soils,* Cambridge, pp. 241—252.
SIM0NS, N.E. and MENZIE5, B.K. (1977) *A short course in foundation engineering,* IPC Science and Technology Press, London.
SIM0NS, N.E. and SOM, N.N. (1969) ‘The influence of lateral stress on the stress deformation characteristics of London Clay’, *Proc. 7th Int. Conf. Soil Mech. and Found. Eng.,* Mexico, Volume 1, pp. 369—377.
SIMPSON, B. (1992) ‘Retaining structures: displacement and design: 32nd Rankine Lecture’, *Géotechnique,* 42 (4), 541—76.
SIMPSON, B., O’RIORDAN, N.J. and CROFT, D.D. (1979) ‘A computer model for the analysis of ground movements in London clay; *Géotechnique,* 29(2), 149—175.
SKEMPTON, A.W. (1948) ‘Vane tests in the Alluvial Plain of the River Forth, near Grangemouth’, *Géotechnique,* 1, 111—124.
SKEMPTON, A.W. (1951) ‘The bearing capacity of clays’, *Build. Res. Cong.,* London, Volume 1, pp. 180—189.
SKEMPTON, A.W. (1954) ‘The pore pressure coefficients A and B’, *Geotechnique, 4(4),* 153 173.
SKEMPTON, A.W. (1959) ‘Cast in-situ bored piles in London clay’, *Géotechnique, 9* (4), pp. 153—173.
SKEMPTON, A.W. (1960a) ‘The pore pressure coefficients in saturated soils’, *Géotechnique,* 10, 186—187.

SKEMPTON, A.W. (1960b) ‘Significance of Terzaghi’s concept of effective stress’, in *From Theory to Practice in Soil Mechanics,* John Wiley, New York.
SKEMPTON, A.W. (1964) ‘Long term stability of clay slopes’, 4th Rankine Lecture, *Géotechnique,* 14, pp. 77—101.
SKEMPTON, A.W. (1986) ‘Standard penetration test procedures and the effects in sands of overburden pressure, relative density, particle size, ageing and overconsolidation’, *Géotechnique,* 36 (3), 425—447.
SKEMPTON, A.W. and BISHOP, A.W. (1954) ‘Soils’, in M. Reiner (ed.), *Building Materials, their Elasticity and Plasticity,* Chapter X, North Holland, Amsterdam.
SKEMPTON, A.W. and BJERRUM, L. (1957) ‘A contribution to the settlement analysis of foundations on clay’, *Géotechnique, 7* (4), pp. 168—178.
SKEMPTON, A.W. and LA ROCHELLE, P. (1965) ‘The Bradwell Slip: A short term failure in London Clay’, *Géotechnique,* 15 (3), pp. 22 1—242.
SKEMPTON, A.W. and PETLEY, D.J. (1967) ‘The strength along structural discontinuities in stiff clays’, *Proc. Geot. Conf., Oslo,* 2, pp. 3—20.
TELFORD, W.M., GELDART, L.P. and SHERIFF, RE. (1990) *Applied Geophysics,* 2nd edn, Cambridge University Press, Cambridge, 770pp.
TERZAGHI, K. (1923) ‘Die berechnung der Durchlassigkeitsziffer des Tones aus dem Verlauf der hydrodynamischen Spannungserscheinungen’, *Sitzungberichte (Abt. ha) Akademit der Wissenschaften,* Vienna, Part 20, 32 (3/4), 125—138.
TERZAGHI, K. (1936) ‘Relation between soil mechanics and foundation engineering’, Presidential address, *Proc. 1st Int. Conf Soil Mech. and Found. Eng.,* Harvard, Volume 3, pp. 13—18.
TERZAGHI, K. (1939) ‘Soil mechanics — a new chapter in engineering science’, James Forrest Lecture, *J. Inst. Civ. Eng.,* 12, 106—142.
TERZAGHI, K. (1943) *Theoretical Soil Mechanics,* John Wiley, New York.
TERZAGHI, K. (1951) ‘The influence of modern soil studies on the design and construction of foundations’, *Building Research Congress,* London, pp. 139—145.
TERZAGHI, K. (1953) ‘Fifty years of subsoil exploration’, *Proc. 3rd Int. Conf. Soil Mech. and Found. Eng,* Zurich, Volume 3, pp. 227—237.
TERZAGHI, RD. (1965) ‘Sources of error in joint surveys’, *Géotechnique,* 15, 287—304.
TERZAGHI, K. and PECK, RB. (1948) *Soil Mechanics in Engineering Practice,* 1st edn, John Wiley, New York.
THORPE, R. and BROWN, G. (1985) ‘The field description of igneous rocks’, *Geological Society of London Handbook,* Open University Press, Milton Keynes, lS4pp.
TOHNO, I. (1977) ‘Methods to evaluate quality of undisturbed samples of sand’, *Proc. Speciality Session 2 on Soil Sampling, 9th Int. Conf. Soil Mech. and Found. Eng.,* Tokyo, pp. 29—36.
TOMLINSON, M.J. (1980) *Foundation Design and Construction,* Pitman Publishing.
TOMLINSON, M.J. and MEIGH, A.C. (1971) ‘Site investigations for piling in soil and rocks’, *Proc. Inst. Civ. Eng.,* 48, 355—57.
TORNAGHI, R. and CESTARI, F. (1977) ‘A new Italian piston sampler’, *Discussion at Speciality Session 2 on Soil Sampling, 9th Int. Conf. Soil Mech. and Found. Eng.,* Tokyo, Volume 2, pp. 45—47.
TORSTENS5ON, B. (1977) ‘Time dependent effects in the field vane test’, *Proc. Int. Symp. on Soft Clay,* Asian Inst. Tech., Bangkok, pp. 387—397.
TRANTINA, J.A. and CLUFF, L.S. (1964) ‘“NX” bore-hole camera’, *Proc. Symp. on Soil Exploration,* Philadelphia, *American Society Testing and Materials, Spec. Tech. Pub. No. 351,* pp. 108—17.
TUCKER, M.E. (1982) ‘The field description of sedimentary rocks’, *Geological Society of London Handbook,* Open University Press, Milton Keynes, 1 l2pp.
TYRREL, A.P., LAKE, L.M. and PARSONS, A.W. (1983) ‘An investigation of the extra costs arising on highway contracts’, *TRRL Supplementary Report SR814,* Transport and Road Research Laboratory, Crowthorne, Berks.
UFF, J.F. and CLAYTON, CR.!. (1986) *Recommendations for the procurement of ground investigation,* Construction Industry Research and Information Association Special Publication 45, ‘Wpp.
UFF, J.F. and CLAYTON, CR.!. (1991) *Role and responsibility in site investigation,* Construction

Industry Research and Information Association Special Publication 73, 42pp.
UGLOW, 1.M. (1989) 'The measurement of in-situ horizontal earth pressures in a range of cohesive soils', MSc dissertation, University of Surrey.
ULRIKSEN, C.P.F. (1983) *Application of impulse radar to civil engineering,* Doctoral Thesis, Lund University of Technology, Sweden, published by Geophysical Survey systems, Inc., U.S.A.
URSIN, B. (1983) 'Review of elastic and electromagnetic wave propagation in horizontally layered media', *Geophysics,* 48, 1063—1081.
US Bureau of Reclamation (1963) *Earth Manual,* US Dept. of the Interior, Denver, Colorado, USA.
VAN BRUGGEN, J.P. (1936) 'Sampling and testing undisturbed sands from boreholes', *Proc. 1st Int. Conf. Soil Mech. and Found. Eng.,* Harvard, Volume 1, pp. 3—6.
VAN DER VEEN, C. and BOERSMA, L. (1957) 'The bearing capacity of a pile predetermined by a cone penetration test', *Proc. 4th Int. Conf. Soil Mech. and Found. Eng.,* London, Volume 2, pp. *72—75.*
VAN ZIJL, J.S.V. (1978) 'On the uses and abuses of the electrical resistivity method', *Bullet in of the Association of Engineering Geologists,* 15 (1), 85—111.
VAUGHAN, P.R. (1969) 'A note on sealing piezometers in boreholes', *Géotechnique,* 19,405—413.
VERMEIDEN, J. (1948) 'Improved sounding apparatus as developed in Holland since 1936', *Proc. 2nd Int. Conf. on Soil Mech. and Found. Eng.,* Rotterdam, Volume 1, pp. 280—287.
VICKERS, B. (1978) *Laboratory Work in Civil Engineering — Soil Mechanics,* Granada Publishing, London.
VOLD, R.C. (1956) 'Opptogning av uforstyrredt jardprover', *Norwegian Geot. Inst. Bull.,* No. 17.
VON POST, L. (1922) 'Sveriges Geologiska Undersøknings torvinentering och nogra av dess hittils vunna resultat (SGU peat inventory and some preliminary results)', *Svenska Mosskulturføreningens Tidskrft,* Jønkøping, Sweden, 36, 1—37.
WARD, W.H. (1953) 'Soil movement and weather', *Proc. 3rd Int. Conf. Soil. Mech. and Found. Eng.,* Zurich, Volume 1, pp. 477—82.
WAKELING, T.R.M. (1970) 'A comparison of the results of standard site investigations methods against the results of a detailed geotechnical investigation in Middle Chalk at Mundford, Norfolk', *Proc. Conf. on In situ Investigations in Soils and Rocks,* British Geotechnical Society, London, pp. 17—22.
WARD, W.H. (1967) 'Discussion on shear strength of stiff clay', *Proc. Geot. Conf.,* Oslo, Volume 2, pp. 140—141.
WARD, W.H., BURLAND, J.B. and GALLOIS, R.W. (1968) 'Geotechnical assessment of a site at Mundford, Norfolk, for a large proton accelerator', *Géotechnique,* 18, 399—431.
WATT, AS. *(1955)* 'Stone stripes in Breckland' *Geol. Mag.,* 92, 173—74.
WATT, AS., PERRIN, R.M.S. and WEST, R.G. (1966) 'Patterned ground in Breckland: Structure and composition', *Journal of Ecology,* 54, 239—58.
WEBSTER, R. (1965a) 'The soils, physiography and cultural pattern of part of the English Cotswolds', *Photo Interpretation,* 65 (1/4), pp. 22—28.
WEBSTER, R. (l965b) 'Landscape and soils in the Vale of the White Horse, Berkshire (England)', *Photo Interpretation, 65* (4/2), 8—14.
WEBSTER, R. (l965c) 'Physiography and soils in the Upper Thames valley', *Photo Interprétation,* 65 (6/3), 15—21.
WEBSTER, R. (1968) 'Discussion on the air photograph requirements of geologists' (see Norman l968b), *Photogrammetric Record, 6* (32), 146—47.
WEBSTER, R. and WONG, I.F.T. (1969) 'A numerical procedure for testing soil boundaries interpreted from air photographs', *Photogrammetria,* 24, 54—72.
WEEKS, A.G. (1969) 'The stability of natural slopes in South-East England as affected by periglacial activity', Q. *J. Eng. Geol.,* 2 (1), 49—61.
WEEKS, A.G. (1970) 'The stability of the Lower Greensand escarpment in Kent', PhD thesis, University of Surrey.
WEST, G. (1979) 'A preliminary study of the reproducibility of joint measurements in rock', *TRRL*

Report SR 488, Transport and Road Research Laboratory, Crowthorne, Berks.
WHITMAN, R.V., RICHARDSON, A. and HEALY, K.A. (1961) 'Time lag in pore pressure measurements', *Proc. 5th Int. Conf Soil Mech. Found. Eng.*, Paris, Volume 1, pp. 407—411.
WHYTE, IL. (1976) 'The development of site investigations', *Ground Engineering*, 9(7), 35—38.
WIESEL, C-E. (1973) 'Some factors influencing in situ vane test results', *Proc. 8th Int. Conf Soil Mech. and Found. Eng.*, Moscow, Volume 1, pp. 475—479.
WILKINSON, W.B. (1968) 'Constant head in-situ permeability tests in clay strata', *Géotechnique*, 18, 172—194.
WILLIAMS, R.B.G. (1964) 'Fossil patterned ground in Eastern England', *Biuletyn Peryglajalny Societas Scientarum Lodziensis*, 14, 337—349.
WILLIAMS, J.T. and METTAM, *3D.* (1971) 'National Ports Council — port structures report', *Proc. Inst. Civ. Eng., Suppl. No. xiii.*
WILLIAMS, W.H. (1938) 'Foundation work in London, Terminal Building for Imperial Airways Limited', *Civil Engineering*, April.
WILSON, N.E. (1964) 'Laboratory shear testing of soils', *ASTM Symp. on Laboratory Shear Testing of Soils*, STP 361, pp. 377—389.
WISSA, A.E.Z., CHRISTIAN, J.T., DAVIS, E.H. and HEIBURG, S. (1971) 'Consolidation at constant rate of strain', *Proc. ASCE, J. Soil Mech. and Found. Engng. Div.*, 97 (SM 10), 1393—1413.
WITHERS, N.J., SCHAAP, L.H.J. and DALTON, C.P. (1986) 'The development of a full-displacement pressuremeter', *Proc. 2nd Symp. on the Pressuremeter and its Marine Applications*, ASTM, Philadelphia, pp. *38—56.*
WRENCH, B.P. (1984) 'Plate tests for the measurements of modulus and bearing capacity of gravels', *The Civil Engineer in South Africa*, Sept., 429—437.
WROTH, C.P. and HUGHES, J.M.O. (1973) 'An instrument for the in situ measurement of the properties of soft clays', *Proc. 8th Int. Conf. Soil Mech. and Found. Eng.*, Moscow, Volume 1.2, pp. 487—494.
YAMADA, G. and UEZAWA, J. (1969) 'Sampling of loose saturated sands', *Proc. Speciality Session 1 on Soil Sampling, 7th Int. Conf. Soil Mech. and Found. Eng.*, Mexico, pp. 39—42.
YENN, R.J. (1992) 'The verification of rock foundations — A case study', MSc dissertation, Department of Civil Engineering, University of Surrey.
Y0SHIMI, Y., HATANAKA, M. and OH-OKA, H. (1977) 'A simple method for undisturbed sand sampling by freezing', *Proc. Speciality Session 2 on Soil Sampling, 9th Int. Conf. Soil Mech. and Found. Eng.*, Tokyo, pp. 23—28.

Standards

American Society for Testing and Materials

ASTM D420—87 (1987) *Standard guide for investigating and sampling soil and rock,* American Society for Testing and Materials, Philadelphia, USA.
ASTM D422—63 (Re-approved 1990) *Standard test method for particle-size analysis,* American Society for Testing and Materials, Philadelphia, USA.
ASTM D698—9l (1991) *Test method for laboratory compaction characteristics of soil using standard effort (12,400 ft-lbs/ft3 (600 kN-m/m3),* American Society for Testing and Materials, Philadelphia, USA.
ASTM D854—92 (1992) *Standard test method for specific gravity of soils,* American Society for Testing and Materials, Philadelphia, USA.
ASTM Dl 194—72 (Re-approved 1987) *Standard test method for bearing capacity of soil for static load and spread footings,* American Society for Testing and Materials, Philadelphia, USA.
ASTM Dl452—80 (Re-approved 1990) *Standard practice for soil investigation by auger boring,* American Society for Testing and Materials, Philadelphia, USA.
ASTM Dl 556—90 (1990) *Standard test method for density and unit weight of soil in place by the sand-core method,* American Society for Testing and Materials, Philadelphia, USA.
ASTM Dl 557—91(1991) *Test method for laboratory compaction characteristics of soil using modified effort (56,000 ft-lbs/ft3 (2,700 kN-m/m3)),* American Society for Testing and Materials, Philadelphia, USA.
ASTM D1586—84 (Re-approved 1992) *Standard test method for penetration test and split- barrel sampling of soils,* American Society for Testing and Materials, Philadelphia, USA.
ASTM Dl 587—83 (1983) *Standard practice for thin-walled tube sampling of soils,* American Society for Testing and Materials, Philadelphia, USA.
ASTM Dl883—92 (1992) *Standard test method for CBR (California Bearing Ratio) at laboratory-compacted soils,* American Society for Testing and Materials, Philadelphia, USA.
ASTM D2 113—83 (Re-approved 1987) *Standard practice for diamond core drilling for site investigation,* American Society for Testing and Materials, Philadelphia, USA.
ASTM D2 167—84 (Re-approved 1990) *Standard test method for density and unit weight of soil in place by a rubber balloon method,* American Society for Testing and Materials, Philadelphia. USA.
ASTM D22l6—92 (1992) *Standard test method for laboratory determination of water (moisture) content of soil and rock,* American Society for Testing and Materials, Philadelphia, USA.
ASTM D22 17—85 (1985) *Standard practice for wet preparation of soil samples for particle size analysis and determination of soil constants,* American Society for Testing and Materials, Philadelphia, USA.
ASTM D2434—68 (Re-approved 1974) *Standard test method for permeability of granular soils (constant head),* American Society for Testing and Materials, Philadelphia, USA.
ASTM D2435—90 (1990) *Standard test method for one-dimensional consolidation properties of soils,* American Society for Testing and Materials, Philadelphia, USA.
ASTM D2487—92 (1992) *Standard classification of soils for engineering purposes (unified soil classification system),* American Society for Testing and Materials, Philadelphia, USA.
ASTM D2488 90 (1990) *Standard practice for description and identification of soils (visual—manual procedure),* American Society for Testing and Materials, Philadelphia, USA.
ASTM D2573—72 (Re-approved 1978) *Standard test method for field vane shear test in cohesive soil,* American Society for Testing and Materials, Philadelphia, USA.

ASTM D2850—87 (1987) *Standard test method for unconsolidated undrained compressive strength of cohesive soils in triaxial compression,* American Society for Testing and Materials, Philadelphia, USA.
ASTM D2922—9 1 (1991) *Standard test method for density of soil and soil aggregate in place by nuclear methods (shallow depth),* American Society for Testing and Materials, Philadelphia, USA.
ASTM D2937—83 (Re-approved 1990) *Standard test method for density of soil in place by the drive-cylinder method,* American Society for Testing and Materials, Philadelphia, USA.
ASTM D2974—87 (1987) *Standard test method for moisture, ash, and organic matter of peat and other organic soils,* American Society for Testing and Materials, Philadelphia, USA.
ASTM D3080—90 (1990) *Standard test method for direct shear test of soils under consolidated drained conditions,* American Society for Testing and Materials, Philadelphia, USA.
ASTM D3441—86 (1986) *Standard test method for deep, quasi-static, cone and friction-cone penetration tests of soil,* American Society for Testing and Materials, Philadelphia, USA.
ASTM D3550—84 (Re-approved 1991) *Standard practice for ring-lined barrel sampling of soils,* American Society for Testing and Materials, Philadelphia, USA.
ASTM D4220—89 (1989) *Standard practice for preserving and transporting soil samples,* American Society for Testing and Materials, Philadelphia, USA.
ASTM D43l8—84 (1984) *Standard test method for liquid limit, plastic limit, and plasticity index of soils,* American Society for Testing and Materials, Philadelphia, USA.
ASTM D4373—84 (Re-approved 1990) *Standard test method for calcium carbonate content of soils,* American Society for Testing and Materials, Philadelphia, USA.
ASTM D4395—84 (Re-approved 1989) *Standard test method for determining the in situ modulus of deformation of rock mass using the flexible plate loading method,* American Society for Testing and Materials, Philadelphia, USA.
ASTM D4429—84 (1984) *Standard test method for bearing ratio of soils in place,* American Society for Testing and Materials, Philadelphia, USA.
ASTM D4633—86 (1986) *Standard test method for stress wave energy measurement for dynamic penetrometer testing systems,* American Society for Testing and Materials, Philadelphia, USA.
ASTM D4643—87 (1987) *Standard test method for determination of water (moisture) content of soil by the microwave oven method,* American Society for Testing and Materials, Philadelphia, USA.
ASTM D4647—87 (1987) *Standard test method for identification and classification of dispersive clays by the pinhole test,* American Society for Testing and Materials, Philadelphia, USA.
ASTM D4719—87 (1987) *Standard test method for pressuremeter testing in soils,* American Society for Testing and Materials, Philadelphia, USA.

British Standards

BS 1377: Part 1: (1990) *British Standard Methods of test for soils for civil engineering purposes, Part 1: General requirements and sample preparation,* British Standards Institution, London.
BS 1377: Part 2: (1990) *British Standard Methods of test for soils for civil engineering purposes, Part 2: Classification tests,* British Standards Institution, London.
BS 1377: Part 3: (1990) *British Standard Methods of test for soils for civil engineering purposes, Part 3: Chemical and electro-chemical tests,* British Standards Institution, London.
BS 1377: Part 4: (1990) *British Standard Methods of test for soils for civil engineering purposes, Part 4: Compaction-related tests,* British Standards Institution, London.
BS 1377: Part 5: (1990) *British Standard Methods of test for soils for civil engineering purposes, Part 5: Compressibility, permeability and durability tests,* British Standards Institution, London.
BS 1377: Part 6: (1990) *British Standard Methods of test for soils for civil engineering purposes, Part 6: Consolidation and permeability tests in hydraulic cells and with pore pressure measurement,*

British Standards Institution, London.
BS 1377: Part 7: (1990) *British Standard Methods of test for soils for civil engineering purposes, Part 7: Shear strength tests (total stress),* British Standards Institution, London.
BS 1377: Part 8: (1990) *British Standard Methods of test for soils for civil engineering purposes, Part 8: Shear strength tests (effective stress),* British Standards Institution, London.
BS 1377: Part 9: (1990) *British Standard Methods of test for soils for civil engineering purposes, Part 9: In-situ tests,* British Standards Institution, London.
BS 4019: (1974) *Specifications for rotary core drilling equipment,* British Standards Institution, London.
BS *5750:* Part 1 (1987) *Specification for design/development, production, installation and servicing,* British Standards Institution, London.
BS *5930:* (1981) *Code of Practice for Site Investigations* (formerly CP 2001), British Standards Institution, London.

Other Standards

DIN 4094 (1980) *Dynamic and standard penetrometers, Part 1: Dimensions of apparatus and method of operation; Part 2: Application and evaluation.* Deutsches Institüt für Normung, Berlin.
JIS A-1219 (1976) Method of penetration test for soils, Japanese Industry Standard.

International Society for Rock Mechanics Suggested Methods

The ISRM Commission on Testing Methods (formerly the Commission for the Standardization of Laboratory and Field Tests) Suggested Methods are published in the International Journal of Rock Mechanics, Mining Science and Geomechanics Abstracts. They are listed here by year of publication.

1978

Suggested method for petrographic description of rocks, (Co-ordinators D.K. Hallbauer, C. Nieble, J. Berard, F. Rummel, A Houghton, E. Broch and J. Szlavin, Vol. 15, No. 2,
41—46.
Suggested methods for determining the strength of rock materials in triaxial compression, (Co-ordinators U.W. Vogler and K. Kovari), Vol. *15,* No. 2, 47—52.
Suggested methods for determining sound velocity, (Co-ordinators F. Rummel and W.L. van Heerden), Vol. 15, No. 2, 53—58.
Suggested methods for determining hardness and abrasiveness of rocks, (Co-ordinators R.H. Atkinson, WE. Bamford, E. Broch, D.U. Deere, J.A. Franklin, C. Nieble, F. Rummel, P.J. Tarkoy and H. van Duyse), Vol. 15, No. 3, 89—98.
Suggested methods for determining tensile strength of rock materials, (Co-ordinators Z.T. Bieniawski and I. Hawkes), Vol. 15, No. 3, 99—104.
Suggested methods for monitoring rock movements using borehole extensometers, (Coordinator W.H. Hansmire), Vol. 15, No. 6, 305—3 18.
Suggested methods for the quantitative description of discontinuities in rock masses, (Coordinator N. Barton), Vol. 15, No. 6, 319—368.

1979

Suggested method for determining the uniaxial compressive strength and deformability of rock materials, (Co-ordinators Z.T. Bieniawski and M.J. Bernede), Vol. 16, No. 2, 135—

140.
Suggested methods for determining water content, porosity, density, absorption related properties and swelling and slake durability index properties, (Co-ordinator J.A. Franklin), Vol. 16, No. 2, 141—156.

1980
Suggested methods for pressure monitoring using hydraulic cells, (Co-ordinator J.A. Franklin), Vol. 17, No. 2, 117—128.

1981
Suggested methods for geophysical logging of boreholes, (Co-ordinator M.S. King), Vol. 18, No. 1, 67—84.
Basic geotechnical description of rock masses, (Co-ordinator M. Rocha), Vol. 18, No. 1, 85—110.

1983
Suggested methods for determining the strength of rock materials in triaxial compression: revised version, (Co-ordinator K. Kovari), Vol. 20, No. 6, 283—290.

1984
Suggested methods for surface monitoring of movements across discontinuities, (Coordinator J. Dunnicliff), Vol. 21, No. 5, 265—276.

1985
Suggested methods for determining point load strength, (Co-ordinator J.A. Franklin), Vol. 22, No. 2, 5 1—60.
Suggested methods for rock anchorage testing, (Co-ordinators W.E. Lardnes and G.S. Littlejohn), Vol. 22, No. 2, 71—84.

1986
Suggested method for deformability determination using a large flat jack technique, (Coordinator J. Loureiro-Pinto), Vol. 23. No. 2, 131—140.

1987
Suggested methods for rock stress determination, (Co-ordinators K. Kim and J.A. Franklin), Vol. 24, No. 1, *53—74.*
Suggested methods for deformability determination using a flexible dilatometer, (Coordinator B. Ladanyi), Vol. 24, No. 2, 123—134.

1988
Suggested methods for determining the fracture toughness of rock, (Co-ordinator F. Ouchterlony), Vol. *25,* No. 2, 71—96.
Suggested methods for seismic testing within and between boreholes, (Co-ordinator K. Sassa), Vol. *25,* No, 6, 449—472.

1989
Suggested methods for laboratory testing of argillaceous swelling rocks, (Co-ordinator H. Einstein),

Vol. 26, No. 5, 415—426.
Suggested method for large scale sampling and triaxial testing of jointed rock, (Co-ordinators OP. Natau and TO. Mutschler), Vol. 26, No. *5*, 427—434.