
Code of practice for

Site investigations

(Formerly CP 2001)

Code de bonne pratique pour les inspections de terrain

Richtlinie für Baugrunduntersuchungen

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Foreword

The original Civil Engineering code of practice No. 1, 'Site investigations', was prepared by a committee convened in 1943 by the Institution of Civil Engineers on behalf of the Codes of Practice Committee for Civil Engineering, Public Works, Building and Construction Work, under the aegis of the former Ministry of Works, for publication in the Civil Engineering and Public Works Series. The code was published in 1950.

In 1949, the responsibility for the preparation and issue of the codes of practice in the Civil Engineering and Public Works Series was handed over to the Institution of Civil Engineers, the Institution of Municipal Engineers, the Institution of Structural Engineers and the Institution of Water Engineers. Arrangements for the preparation and publication of these codes were made by the Civil Engineering Codes of Practice Joint Committee constituted by those four Institutions.

In 1954, agreement was reached between the professional Institutions concerned, the British Standards Institution

and the Council for Codes of Practice for Buildings (under the aegis of the former Ministry of Works) to transfer responsibility for the issue of codes of practice to the British Standards Institution. At the request of the British Standards Institution, a committee was convened by the Institution of Civil Engineers to amend the 1950 code. A revised code was published in 1957 by the British Standards Institution, under authority of its Council for Codes of Practice, as CP 2001.

The Institution of Civil Engineers similarly convened a further committee in 1965 to prepare a revised code. In 1976, the convening responsibility was transferred to the British Standards Institution.

NOTE. The numbers in square brackets used throughout the text of this code relate to the bibliographic references in appendix H.

This code of practice represents a standard of good practice and therefore takes the form of recommendations. Compliance with it does not confer immunity from relevant statutory and legal requirements.

British Standard Code of practice for Site investigations

Section one. General

1. Scope

This British Standard deals with the investigation of sites for the purposes of assessing their suitability for the construction of civil engineering and building works and of acquiring knowledge of the characteristics of a site that affect the design and construction of such work and the security of neighbouring land and property. It has been assumed that in the selection of construction sites due regard has been paid to the wider environmental and economic considerations affecting the community generally. More than one site may require detailed investigation before the final choice is made.

In this code, the expression 'site investigation' has been used in its wider sense. It is often used elsewhere in a narrow sense to describe the exploration of the ground, which in this code has been termed 'ground investigation'. It is to be noted, however, that although the treatment of ground investigation is detailed, the treatment of other aspects of site investigation is less detailed.

The use of soil and rock as construction materials is treated only briefly; further information is given in BS 6031.

From section two onwards, the code is divided as follows:

Section two. Section two deals with those matters of a technical, legal or environmental character that have usually to be taken into account in selecting the site (or in determining whether a selected site is suitable) and in preparing the design of the works.

Section three. Section three discusses general aspects and planning of ground investigations, including the influence of general conditions and ground conditions on the selection of methods of investigation.

Sections four, five and six. Sections four, five and six discuss methods of ground investigation, sub-divided as follows: section four: excavation, boring, sampling, probing and tests in boreholes; section five: field tests; section six: tests on samples. The division between sections four and five is somewhat arbitrary and has been based mainly on convenience in arranging the subject matter.

Section seven. Section seven deals with the preparation of field reports and final borehole logs, the interpretation of the data obtained from the investigation and the preparation of the final report.

Section eight. Section eight deals with the terminology and systems recommended for use in describing and

classifying soil and rock materials and rock masses.

Appendix G refers to the application of geology to engineering.

Users of this code of practice, particularly those of limited experience, are advised against referring to the methods of ground investigation in sections four, five and six without studying the general considerations in sections two and three. It should be appreciated that the description of a method is only to be considered as a guide and should not be taken as mandatory. In this respect, it should be realized that development will continue to take place involving changes in some of the methods.

It may be noted that there is an imbalance of treatment between tests; in some cases more comprehensive treatment has been given to tests less frequently used. This is because many of the common tests are described extensively elsewhere, whereas there is a paucity of reference to other tests.

The code has been drawn up in relation to conditions existing in the United Kingdom, but occasional reference is made to conditions overseas where this has appeared desirable.

2. References

The titles of the standards publications referred to in this code are listed on the inside back cover.

Section two. General considerations

3. Primary objects of site investigation

Investigation of the site is an essential preliminary to the construction of all civil engineering and building works and the objects in making such investigations are as follows.

- Suitability.** To assess the general suitability of the site and environs for the proposed works.
- Design.** To enable an adequate and economic design to be prepared, including the design of temporary works.
- Construction.** To plan the best method of construction; to foresee and provide against difficulties and delays that may arise during construction; to identify ground and other local conditions; in appropriate cases, to explore sources of indigenous materials for use in construction (see 8.4); and to select sites for the disposal of waste or surplus materials.

(d) *Effect of changes.* To determine the changes that may arise in the ground and environmental conditions, either naturally or as a result of the works, and the effect of such changes on the works, on adjacent works, and on the environment in general.

(e) *Choice of site.* Where alternatives exist, to advise on the relative suitability of different sites, or different parts of the same site.

In addition, site investigations may be necessary in reporting upon the safety of existing works (see 8.3), for the design of extensions, vertical or horizontal, to existing works, and for investigating cases where failure has occurred (see 8.2).

4. General procedure

4.1 Extent and sequence of investigation

4.1.1 General. The extent of the investigation depends primarily upon the magnitude and nature of the proposed works and the nature of the site.

A site investigation will normally proceed in stages, as follows: desk study; site reconnaissance; detailed examination for design, including ground investigation, topographic and hydrographic survey and special studies; follow-up investigations during construction. This may be followed by appraisal of performance. Some of the stages may overlap, or be taken out of sequence; for example, the site reconnaissance may well take place before completion of the desk study.

The costs of a site investigation are low in relation to the overall cost of a project and may be further reduced by intelligent forward planning. Discussion at an early stage with a specialist contractor will help to formulate an efficient and economic plan. The technical requirements of the investigation should be the overriding factor in the selection of investigatory methods, rather than their cost.

As far as possible, assembly of the desk study information should be complete, at least in respect of those aspects related to ground conditions, before ground investigation begins. A preliminary ground investigation may often be desirable to determine the extent and nature of the main ground investigation. The extent of the ground investigation is discussed in clause 10.

4.1.2 Adjacent property. In view of the possibility that the construction of new works may affect adjacent property or interests, it is desirable that investigations for new works should cover all factors that may affect adjacent land or existing works and that, where possible and expedient, records of ground levels, ground water levels and relevant particulars of adjacent properties should be made before, during and after the construction of the new works. Where damage of existing structures is a possibility, adequate photographic records should be obtained. This is particularly important where existing structures are defective.

4.2 Desk study. As a first stage in a site investigation, a desk study is necessary and appendix A indicates the kinds of information that may be required. Where there is a choice of site, information obtained from this study may well influence such choice. Much information may already be available about a site in existing records. A summary of the most important sources of information is given in appendix B, and a more detailed catalogue is given elsewhere [1].

Information based on local experience, including earlier

and local or regional statutory undertakings (see clause 5). Occasionally, it is possible to obtain the results of previous ground investigations carried out on or near the site.

Excavations on or near the site may have been made in the recent past by such authorities or with their knowledge. Information may also be available from local industry and others. Copies of old maps are often available in public libraries and local museums. Papers on local projects presented to professional bodies and local societies and technical journals may be other sources of information. Local oral tradition, although of variable reliability, may sometimes give a lead. Helpful information may be obtained from aerial photographs, see clause 6 and B.6. For sources of information about mine workings, see E.5.

4.3 Site reconnaissance. At an early stage, a thorough visual examination should be made of the site. The extent to which ground adjacent to the site should also be examined is, in general, a matter of judgement, see 4.1.2. Appendix C gives a summary of the procedure for site reconnaissance and the main points to be considered but, should not, however, be regarded as necessarily covering all requirements.

Nearby railway or road cuttings can reveal soil and rock types and their stability characteristics as can old pits and quarries. Similarly, in the vicinity there may be embankments or buildings and other structures having a settlement history because of the presence of compressible or unstable soils. Other important evidence that might be obtained from an inspection is the presence of current or old mine workings (see appendix E) or other underground excavations, such as old cellars, tunnels, sewers. The behaviour of structures similar to those intended should also provide useful information, and the absence of such structures may be significant, as may be also the presence of a vacant site in the midst of otherwise intensive development.

For instances of earlier uses of the site that may affect new construction works, see clause 5.

4.4 Detailed examination and special studies (see also appendix D). For most projects, the design and planning of construction will require a detailed examination of the site and its surroundings. Such requirements may necessitate a detailed land survey, see D.1, or an investigation of liability to flooding. The investigation of ground conditions is dealt with in other sections in this code, e.g. section three. Other requirements may entail studies of special subjects such as hydrography, see D.4; climate, see D.5; sources of materials, see D.7; disposal of waste materials, see D.8; or other environmental considerations.

Where mining and quarrying, whether past, present or prospective, is likely to be a factor affecting the site, reference should be made to appendix E. Effects of tunnelling, where this is a possibility, should be considered.

4.5 Construction and performance appraisal. Construction and performance appraisal are discussed in clause 16.

5. Earlier uses and state of site

5.1 General. If a site has been used for other purposes in the past, this can have much effect on present intended use. A careful visual inspection of a site and the vegetation it sustains may reveal clues suggesting interference with the natural subsoil conditions at some time in the past. Examples are given in 5.2 to 5.7.

The former uses of the site can sometimes be discovered by a study of old Ordnance and earlier maps. The fore-runners of local authorities in some cases commissioned the making of maps of their areas long before Ordnance maps were produced, and these maps were often made in the most meticulous detail.

In the marine environment where topographic changes may occur more rapidly, reference to early Admiralty charts and other charts may indicate earlier configurations, see B.3.1.

5.2 Underground mining. Appendix E gives details of the procedure that should be followed for enquiries and subsequent site investigation in areas where underground mining has been carried out, is at present being carried out, or is envisaged.

5.3 Opencast mining. The extraction of coal by opencast methods has been carried on extensively since the late 1930s and details of the availability of records are given in appendix E. Other minerals have also been extracted by opencast methods.

5.4 Quarry operations. The quarrying of materials such as stone, chalk, sand, gravel and clay has been carried on since ancient times and many excavations have, over the years, been back-filled and put to some other use.

5.5 Waste tips. The site may have been used for the tipping of mining waste, industrial waste, domestic refuse, chemical waste and miscellaneous refuse. Frequently, harmful industrial waste may also be encountered.

5.6 Industrial complexes. In many parts of the country there are sites where heavy industries once existed. Often no visible signs remain of the buildings and other structures, but below ground level, for example, there may be foundations, engine beds, pits and chambers, often of massive construction, which can be severe obstacles to redevelopment. The ground may still be affected by extremes of temperature from installations such as cold-storage plant, high-temperature kilns and furnaces. These effects may be revealed by aerial photography, see clause 6.

5.7 Other earlier use. Redevelopment of urban areas may encounter obstacles in the form of cesspits, sewers, wells, cellars, drains and communication passages. Disused tunnels may also be met.

5.8 Ancient monuments. Should it become apparent during the desk study that any ancient monument or site of archaeological interest is likely to be affected by the investigation or the subsequent works, the matter should be referred to the Chief Inspector of Ancient Monuments*.

6. Aerial photographs

6.1 General. Aerial photographs can be used in the preparation and revision of maps and plans. Although of use for small and large sites, they are particularly useful for large undeveloped sites, notably for such projects as dams, power stations, and highways. They can also assist in the identification of geological and geomorphological features on or in relation to a site, and in the interpretation of earlier uses of the site [2, 3]. Trained interpreters are necessary if full information is to be obtained from aerial photographs, although much can be done by a non-specialist [4]. Interpreters should be carefully briefed on the requirements of the interpretation.

Before special photography is commissioned, available sources should be checked, see B.6.

6.2 Topographical mapping. Accurate contoured maps can be produced from aerial photographs by a competent survey organization, which will normally carry out ground control. This is done by placing markers on the ground that can be identified from the air and also measured in plan and level on the ground.

The scale of photography should be properly related to the project. Generally, photography is available at the various scales considered appropriate for conventional map or plan making. The scales 1:500, 1:1000, and 1:2500 are most appropriate for investigations of limited areas, whereas scales of 1:5000 to 1:20 000 are more appropriate to regional studies. Although much map revision of urban areas is made by the Ordnance Survey using aerial survey, specially commissioned surveys are not likely to be justifiable for small urban sites.

A conventional vertical photograph is not true to scale. Orthophotomaps, which consist of rectified (true to scale) photographs overprinted with contours and grid, are also available and these provide for some purposes an alternative to a conventional map or plan, and may be of particular value to major site investigations.

6.3 Identification of features. Aerial photographs can often be used to identify such features of engineering significance as geological lineaments, e.g. strata boundaries, faults; soil and rock types; landforms; and drainage patterns and unstable ground, including areas of mining disturbance and swallow holes. They are particularly useful in the study of extended sites where ground visibility is limited by obstructions or access is difficult. Significant features should be checked on the ground whenever possible. Although photographs are best studied stereoscopically, much data can be obtained from single photographs.

Additional information may be gained by the study of aerial photographs taken at different times when the direction or nature of light differs, or when soil and vegetation has undergone seasonal changes.

6.4 Sophisticated techniques. Natural colour does have particular advantages. Multispectral techniques providing simultaneous images in selective wave bands, including the infra-red, are also available. The use of false colour to enhance such features as vegetation is possible when selective wave bands have been used [2]. It is stressed that the more sophisticated, and costly, systems require more highly trained interpreters if the full value of such systems is to be realized. It should be appreciated that rapid advances are being made in the use of sensor systems, in both the conventional airborne field and using earth satellite imagery.

Section three. Ground investigation

7. Introduction

7.1 Objects. For new works, the objects of ground investigation are to obtain reliable information to produce an economic and safe design and to meet the design and construction requirements. The investigation should be designed to verify and expand information previously collected.

The objects of ground investigation related to defects or failures of existing works, see 8.2, or to safety of existing works, see 8.3, will be directly related to the particular problems involved. The requirements for investigation of materials for construction purposes are discussed in 8.4.

An understanding of the geology of the site is a fundamental requirement in the planning and interpretation of the ground investigation. In some cases where the geology is relatively straightforward and the engineering problems are not complex, sufficient geological information may have been provided by the desk study, subject to confirmation by trial pits or boreholes or both. In other cases, it may be necessary to undertake geological mapping, which is discussed in clause 9.

Of primary importance will be the establishment of the soil profile or soil and rock profile, and the ground water conditions. The profile should be obtained by close visual inspection and systematic description of the ground using the methods and terminology given in clause 4.1. In many cases, this supplemented by limited in situ or laboratory testing will suffice. In others, it will be necessary to determine in detail the engineering properties of the soils and rocks. The extent of the ground investigation is discussed in clause 10. Where appropriate, the geometry and nature of discontinuities should be established, see 12.10.

The investigation should embrace all ground in which temporary or permanent changes may occur as a result of the works. These changes include: changes in stress and associated strain, changes in moisture content and associated volume changes, changes in ground water level and flow pattern, and changes in soil properties such as strength and compressibility. Materials placed in the ground may deteriorate. It is therefore necessary to provide information from which an estimate of the corrosivity of the ground can be made, see clause 13.

In certain districts, special measures need to be taken to locate underground cavities, such as old mine workings, swallow holes and dene holes, which may collapse resulting in damage to structures. Other hazards may arise from earlier uses of the site (see clause 5).

7.2 Planning and control. Before commencing ground investigation, all relevant information collected from the sources discussed in section two should be considered together to obtain a preliminary conception of the ground conditions and the engineering problems that may be involved.

This will assist in planning the amount and types of ground investigation required.

A ground investigation should be conducted as an operation of discovery. Planning should be flexible so that the work can be varied as necessary in the light of fresh information. On many occasions, especially on large or extended sites, a preliminary investigation is necessary in order that the main investigation may be planned to best advantage.

The ground investigation should be completed before the works are finally designed. It is therefore important that sufficient time for ground investigation, including reporting and interpretation, is allowed in the overall programme for any scheme. Should changes in the project occur after completion of the main investigation, additional ground investigation may be required. If so, the programme should be adjusted to allow for the additional time required.

Sometimes, conditions necessitate additional investigation after the works commence. In tunnelling, for example, probing ahead of the face may be required to give warning of hazards or changes in ground conditions. The properties of the ground and also the ground water levels may vary with the seasons. In planning the investigation, consideration should be given to predicting the ground conditions at other times of the year.

The imposition of limitations on the amount of ground investigation to be undertaken, on the grounds of cost and time, may result in insufficient information being obtained to enable the works to be designed, tendered for and constructed adequately, economically and on time. Additional investigations carried out at a later stage may prove more costly and result in delays.

To obtain the greatest benefit from a ground investigation, it is essential that there be adequate direction and supervision of the work by competent personnel who have appropriate knowledge and experience and the authority to decide on variations to the ground investigation when required (see clause 15).

8. Types of ground investigation

8.1 Sites for new works. Investigations for new works differ from the other types of investigation mentioned in clause 7, in that they are usually wider in scope because they are required to yield information to assist in selecting the most suitable location for the works, and the design and construction of the works. For instance, when excavation has to be carried out, a knowledge of the subsurface strata and ground water conditions should indicate, for example:

- (a) whether removal of the material will be difficult;
- (b) whether the sides of the excavation will be stable if unsupported or will require support (see BS 6031);
- (c) whether ground water conditions will necessitate special precautions such as ground water lowering or other geotechnical processes;
- (d) whether the nature of the excavated material will change.

On the design side, it is necessary to assess such considerations as bearing capacity and settlement of foundations, stability of slopes in embankments and cuttings, and earth pressures on supporting structures, and the effect of any chemically aggressive ground conditions. For the design of new works, it is important that the range of conditions, including least favourable conditions, should be known. This entails not only a study of the degree of variability in the strata over the area of the site, but also an appreciation of the possible injurious effects of ground water variations and weather conditions on the properties of the various strata. Where works require excavations into or within rock, the orientation and nature of discontinuities in the rock may be the most important factors.

8.2 Defects or failures of existing works. The investigation of a site where a failure has occurred is often necessary to establish the cause of the failure and to obtain the information required for the design of remedial measures.

Observations and measurements of the structure to determine the mode or mechanism of failure are first needed and these will often suggest the origin of the trouble or at least indicate whether the ground conditions were partly or wholly responsible. If this is the case, ground investigation will be required to ascertain the condition of the strata and the ground water conditions as they exist in situ and as they existed before the works were constructed. Each problem will need to be considered on its merits. Indications of the probable cause of a failure will often result in detailed attention being directed to a particular aspect or to a particular stratum of soil.

In the case of slope failure, or where such failure is considered imminent, it is common practice to monitor movements both of the surface and underground. The former is conducted by conventional survey methods and the latter by means of slip indicators or inclinometer measurements. These techniques are fully described in BS 6031.

Therefore, an investigation to determine the causes of a failure may be much more detailed in a particular respect than would normally be the case in an investigation for new works.

8.3 Safety of existing works

8.3.1 Effect of new works upon existing works. It may be necessary to investigate existing works in the immediate vicinity or even remote from the site of the proposed new works, to decide whether the existing works are likely to be affected by changes in the ground and ground water conditions brought about by the new works.

8.3.2 Kinds of effects. Existing structures may be affected by changed conditions such as the following:

- (a) excavations or demolitions in the immediate vicinity, which may cause a reduction in support to the structure either by general ground deformation or by slope instability;
- (b) mining or tunnelling operations in the neighbourhood, which may cause deformations and subsidence; the effect of tension and compression on drainage should not be overlooked;
- (c) stresses that the new structure may impose on the foundation strata below adjacent structures or upon earthworks and supporting structures;
- (d) vibrations and ground movement resulting from traffic, vibratory compaction, piling or blasting in the immediate vicinity, or from earthquakes;
- (e) lowering the ground water level by pumping from wells will cause an increase in the effective stress in the subsoil affected, which can lead to excessive settlement of adjacent structures. Also, if pumps do not have an adequate filter, the leaching of fines from the subsoil can easily result in excessive settlement of structures at considerable distance from the pump;
- (f) shrinkage or swelling of clay soils due to weather, transpiration of trees and shrubs, and heat from furnaces;
- (g) soil movements associated with freezing, which may occur naturally or artificially;
- (h) impeded drainage, which may result in a rise in the ground water level. This can cause softening of cohesive strata and reduction of bearing capacity of permeable strata, and give rise to increased pore pressures affecting the stability of slopes and retaining

(i) alteration in stream flow of a waterway, which may cause undercutting of banks or scouring of foundations of walls, bridges and piers, and may be due to works carried out some distance away;

(j) siltation of the approaches of harbour works or the changing of navigation channel alignments.

8.3.3 Procedure. In problems of this kind, the first requirement is an appreciation of the changes that are likely to occur. The ground investigation will need to provide knowledge of the subsurface strata, together with the examination and testing of samples to assess the effect that the changed conditions are likely to have on these strata. In some cases, it may be necessary to carry out a detailed analysis to estimate the effect of the changed conditions on the safety of the existing works.

8.4 Material for constructional purposes. Investigations of sites are sometimes required:

- (a) to assess the suitability, and quantities, for construction work of material that becomes available from excavations or dredgings, e.g. whether spoil from cuts in road and railway works will be suitable for fills in other places;
- (b) to find suitable material for specific purposes, e.g. borrow pits for earthworks, aggregate for concrete and road construction (see BS 882); to assess the suitability of material in waste tips that may need to be removed for environmental reasons;
- (c) to locate suitable disposal sites for waste and dredged materials.

In works such as railways and roads, a prior knowledge of the strata on the line of route may influence design, e.g. by suggesting deeper cuts at places where the material is particularly suitable for fill construction, or a reduction of cuts in unsuitable material. It may be possible to assess the suitability of the material by inspection alone, and in such cases the main task is to explore the extent of the material and to estimate the amount available. In other cases, the suitability of the material may not be evident, and then it is necessary to carry out tests to determine what steps are required to make the best use of it, see table 4, BS 882 and [5]. In investigations of this kind, published and unpublished records of the Institute of Geological Sciences may be found useful as a guide (see B.2).

9. Geological mapping for ground investigation

Details of the geological survey maps are given in B.2. Where these are insufficiently detailed for any investigation in question, it may be necessary to carry out additional geological mapping. Nevertheless, when the maps are considered to be sufficiently detailed, a walkover survey should still be undertaken where appropriate. Methods used for geological mapping [6, 7] are equally suitable for ground investigation purposes, and may be supplemented by geophysical investigations, see clause 33, and interpretation of aerial photographs, see clause 6. The object of mapping is the elucidation of the character, distribution, sequence, and structure of the soils and rocks underlying the area. Interpretation of the geological conditions at the site may not be possible without mapping a larger area. An understanding of geomorphological features may be valuable in interpreting the nature and distribution of soils and rocks. Hydrogeological conditions and man-made features should be recorded.

Natural exposures and artificial exposures, such as quarries and cuttings, beyond the site may provide data on the material and mass characteristics of soils and rocks, including, for example, the orientation, frequency and character of bedding and jointing discontinuities, weathering profiles, and the nature of the junction between superficial and solid formations. Such information should be used as a guide only to conditions likely to be present at the site. Caution is needed in extrapolating data; geological deposits may vary laterally, and very important geological structures, such as faults and other major discontinuities, may have only a restricted extent.

It may be expedient to investigate local conditions at an early stage of the mapping, using mechanically excavated shallow pits and trenches. The walls of excavations and, where appropriate, the floor should be mapped at a suitably large scale and sampled before backfilling takes place.

Recording of geological information should be undertaken at all stages of the works, revising early maps as ground conditions are exposed by excavation, and recording the existing ground conditions.

Further information and examples of engineering geological mapping are given elsewhere [8, 9].

10. Extent of the ground investigation

10.1 General. The extent of the ground investigation is determined by the character and variability of the ground and ground water, and the type of project, and the amount of existing information. It is important that the general character and variability of the ground be established before deciding on the basic principles of the design of the works.

Investigations include a range of 'methods', e.g. excavations, boreholes, probing, see section four. The factors determining the selection of a particular method are discussed in clauses 9 and 12 and section four. In general, the recommendations in 10.2 to 10.7 apply irrespective of the method adopted, and the term 'exploration point' is used to describe a position where the ground is to be explored by any particular 'method'.

Each combination of project and site is likely to be unique, and the following general points should therefore be considered as guidance in planning the ground investigation and not as a set of rules to be applied rigidly in every case.

10.2 Character and variability of the ground. The greater the natural variability of the ground, the greater will be the extent of the ground investigation required to obtain an indication of the character of the ground. The depth of exploration is generally determined by the nature of the works projected, but it may be necessary to explore to greater depths at a limited number of points to establish the overall geological structure. The technical development of the project should be kept under continuous review since decisions on the design will influence the extent of the investigation.

It is important to realize that the detailed geology of a site can be no more than inferred from aerial photography, surface outcrops and subsurface information at the positions of the exploration points. The possibility remains that significant undetected variations or discontinuities can exist, including lateral or vertical variations within a given stratum. The uncertainties can be reduced but, except by

complete excavation, can never be wholly eliminated by a more intensive investigation. The use of angled boreholes can in certain cases greatly assist interpreting variations between vertical boreholes. In some circumstances, additional information between investigation points can be obtained by geophysical methods, see clause 33.

10.3 Nature of the project. The investigation should yield sufficient data on which to base an adequate and economical design of the project. It should in addition be sufficient to cover possible methods of construction and, where appropriate, to indicate sources of constructional materials. The lateral and vertical extent of the investigation should cover all ground that may be significantly affected by the new works or their construction. Two typical examples are the zone of stressed ground beneath the bottom of a group of piles; and an adjacent slope, the stability of which may be decreased by the works.

10.4 Trial pits. Before deciding on a programme, it may be advisable to put down trial pits, which should be carefully examined, logged and sampled (see, for example, [10]).

10.5 Location. The points of exploration, e.g. boreholes, soundings, pits, should be so located that a general geological view of the whole site can be obtained with adequate details of the engineering properties of the soils and rocks and of ground water conditions. More detailed information should be obtained at positions of important structures and earthworks, at points of special engineering difficulty or importance and where ground conditions are complicated, e.g. suspected buried valleys and old slipped areas. Rigid, preconceived patterns of pits, boreholes or soundings should be avoided. In some cases, it will not be possible to locate structures until much of the ground investigation data has been obtained. In such cases, the programme of investigations should be modified accordingly.

For tunnels and inclined shafts, boreholes should be offset so as not to interfere with subsequent construction. For other structures, the need to offset boreholes and trial excavations from critical points should be considered. In most cases, boreholes should be carefully backfilled, concreted or grouted up. Trial excavations should be located outside proposed foundation areas.

For highways, it is important that some exploration points are arranged at offsets to the centre line of the proposed road, so that lateral variation in ground conditions can be revealed.

It is essential that the locations and ground levels for all exploration points be established, if necessary by survey.

10.6 Spacing. Although no hard and fast rules can be laid down, a relatively close spacing between points of exploration, e.g. 10 m to 30 m, will often be appropriate for structures. For structures small in plan area, exploration should be made at a minimum of three points if possible. Where a structure consists of a number of adjacent units, one exploration point per unit may suffice. Certain engineering works, such as dams, tunnels and major excavations, are particularly sensitive to geological conditions, and the spacing and location of exploration points should be more closely related to the detailed geology of the area than is usual for other work.

10.7 Depth of exploration

10.7.1 General. The depth of exploration is governed by the depth to which the new project will affect the ground and ground water or be affected by them. Normally,

exploration should be taken below all deposits that may be unsuitable for foundation purposes, e.g. made ground and weak compressible soils, including the case where weak strata are overlain by a layer of higher bearing capacity. The exploration should be taken through compressible cohesive soils likely to contribute significantly to the settlement of the proposed works, generally to a depth where stress increases cease to be significant, or deeper. If rock is found, a penetration of at least 3 m in more than one borehole may be required to establish whether bedrock or a boulder has been encountered, unless prior knowledge of the local geology obviates this. Three metres may be an insufficient penetration into rock in areas where the rocks have a massive structure (see table 9) or where large, apparently undisturbed sedimentary masses underlain by till may occur as rafts in till deposits. No guidance can be given in such cases, but where doubt arises, consideration should be given to drilling a deeper hole, or making an *in situ* examination in an excavation.

More specifically, the recommendations given in 10.7.2 to 10.7.8 may be considered for certain types of work. It is not always necessary that every exploration should be taken to depths recommended in 10.7.2 to 10.7.8. In many instances, it will be adequate if one or more boreholes are taken to those depths in the early stages of the field work to establish the general ground profile, and then the remainder sunk to some lesser depths to explore more thoroughly the zone near the surface which the initial exploration had shown to be most relevant to the problem in hand.

10.7.2 Foundations for structures. For foundations for structures, the depth of exploration should be at least one and a half times the width of the loaded area. Commonsense will indicate exceptions to this guideline; for example, it would not usually be necessary to continue drilling for long distances in strong rock. For foundations near the surface, the loaded area is considered as either:

- (a) the area of an individual footing; or
- (b) where the spacing of foundation footings is less than about three times the breadth, or where the floor loading is significant, the loaded area should be taken as the plan area of the structures; or
- (c) the area of a foundation raft.

In each case, the depth should be measured below the base of the footing or raft.

Where piled foundations are considered a possibility, the length of pile usually cannot be decided until an advanced stage of the project. No explicit rules can be given for the depth of exploration, but the following offer some guidance.

- (1) Made ground, and weak compressible soils seldom contribute to the shaft resistance of a pile and may add down drag to the load on it. The whole pile load, possibly with the addition of down drag, will have to be borne by the stronger strata lying below the weak materials.
- (2) The length of driven piles is often determined initially by the driving resistance, and subsequently checked by load tests. Hence, in such cases, the length of the pile is not known with any accuracy until the piling contract begins.
- (3) In the case of end bearing piles in strong rock, boreholes should be of sufficient depth to establish

conclusively the presence of rock head. The rock should then be further explored, usually by means of rotary drilling, to such a depth that the engineer directing the investigation is satisfied that there is no possibility of weaker strata occurring lower down that could affect the performance of the piles.

(4) Pile-supported rafts on clays are often used solely to reduce settlement. In these cases, the depth of exploration is governed by the need to examine all strata that could contribute significantly to the settlement. A commonly used approximation in settlement calculations for piled rafts is to assume that the whole of the load is carried on an imaginary raft situated at a depth below the underside of the real raft equal to two-thirds of the pile length. The size of this imaginary raft is determined by assuming a spread of load of 4:1 (vertical : horizontal) outwards from the edge of the rectangle containing the piles. In theory, the depth of exploration should be one-and-a-half times the width of this imaginary raft below its base. In practice, on many occasions, this would lead to an excessive and unnecessary depth of exploration, when the engineer directing the investigation should terminate the exploration at that point once relatively incompressible strata have been reached. Similar considerations would apply to pile groups in these soils, except that it would be necessary to ensure that the depth of exploration is sufficient to prove the strata well below the toes of the piles.

(5) In chalk and other weak rocks, the exploration should be taken to the base of the weathered materials and to a sufficient depth into the unweathered rock to prove its continuity.

Based on the information of the probable ground profile obtained from the desk study, the general guidance given in (1) to (5) and his assessment of the types of pile likely to be considered, the engineer directing the investigation should determine the depth of exploration and be ready during the course of the field work to modify this depth as appears to him to be necessary. In any event, exploration should at some points be taken below the depth to which it is considered likely that the longest piles will penetrate.

It should be noted that if any structure is likely to be affected by subsidence due to mining or any other causes, greater exploration depths than those recommended in this clause may be required so that basic geological data can be obtained.

10.7.3 Embankments. For embankments, the depth of the exploration should be sufficient to check possible shear failure through the foundation strata and to assess the likely amount of any settlement due to compressible strata. In the case of water-retaining embankments, investigation should explore all strata through which piping could be initiated or significant seepage occur.

10.7.4 Cuttings, quarries and opencast mines. For cuttings, quarries and opencast mines, the depth of exploration should be sufficient to permit assessment of the stability of the future slopes; this may necessitate proving the full depth of any relatively weak strata. Groundwater conditions, including the possibility of artesian water, should also be determined.

10.7.5 Roads and airfields. For roads and airfields, the depth of exploration should be sufficient to determine the strength and frost susceptibility of possible subgrades and the drainage conditions. Exploration to a depth 2 m to 3 m below the proposed formation level will probably be sufficient in most cases.

10.7.6 Pipelines. For shallow small pipelines, it will frequently be sufficient to take exploration to 1 m below the invert level. For deeper pipelines the depth of exploration should be sufficient to enable any likely difficulties in excavating trenches and supporting the pipelines to be discovered; a depth at least 1 m to 2 m below the invert level may be advisable. Large pipelines, especially those in ground of low bearing capacity, require special consideration.

10.7.7 Maritime works. For maritime works, the effects described in 10.7.2 and 10.7.3 may apply and, in addition, consideration should be given to the effects of tidal variations.

10.7.8 Tunnels. For tunnels, it is important to take the exploration to a generous depth below the proposed invert level because changes in design may result in the lowering of the level of the tunnel, and because the zone of influence of the tunnel may be extended by the nature of the ground at a greater depth.

11. General considerations in the selection of methods of ground investigation

11.1 Introduction. Although the character of the ground and the technical requirements are the most important aspect in the selection of methods of ground investigation, selection may also be influenced by the character of the site, the availability of equipment and personnel, and the cost of the methods.

11.2 Site considerations. The topography, nature of the ground surface, surface water, and the existence of buildings or other structures, may cause problems of access to the locations for boreholes, or interfere with geophysical methods. For example, if the ground surface is soft, it can be traversed only by very light equipment. Where this would not be effective, access roads for heavier equipment will be required. Alternatively, the use of helicopters or hovercraft may be appropriate. On very steeply sloping open sites, it may be necessary to construct an access road or lower the equipment down the slope or haul it up. Where the working position is on steeply sloping ground, it will be necessary to form a horizontal working area by excavation or the use of a staging. On sites that are obstructed by buildings and other structures, it may be necessary to demolish walls to gain access. Alternatively, it may be possible to lift the equipment over obstructions using a crane or to use special equipment that can be dismantled and manhandled through the building and used in a confined space. Gaining access to sites covered by water presents special problems (see clause 14).

Certain methods of boring* and of field testing require a supply of water. There may be legal restrictions on obtaining water for drilling purposes. In the United Kingdom this may involve the prior publication of notices in order to obtain a licence to abstract. The Water Resources Act 1963 controls permissions to take water

from any natural source. On a site where water is not available, it will be necessary to arrange for a temporary supply to be provided. This generally involves pumping through pipelines or transporting the water in bowsters. On sites where the provision of water presents a major problem, it may be necessary to use alternative methods of investigation; for example, with rotary drilling an air flush could be used instead of water flush, see 18.7.

Other site considerations that may restrict the methods used are:

- (a) in built-up areas, seismic surveys with explosives may be prohibited, and surface obstructions and buried services may interfere with other methods;
- (b) the electrical resistivity method is more difficult to apply in saline ground water conditions, or where there are buried metals;
- (c) difficulty in storage of spoil may restrict the use of pits and trenches.

12. The effect of ground conditions on the selection of methods of ground investigation

12.1 Introduction. 12.2 to 12.10 consider the factors involved in the choice of the most suitable procedures for boring (including drilling), sampling, probing, tests in boreholes, and field tests, as determined by the character of the ground. Frequent reference is made to classes of sample quality; these are defined in 19.2.

The determination of the ground water conditions is a most important part of a ground investigation. It involves the installation of piezometers and borehole or field testing, see clauses 20, 21 and 25, and is not, in general, a major consideration in the choice of a procedure for boring and sampling. Geophysical methods are often a useful means of interpolating between boreholes in a variety of ground conditions, see clause 33.

12.2 Non-cohesive soils containing boulders, cobbles or gravel. Within the limits of cost, the best method for investigating this type of ground is by means of a dry excavation, see 18.1 and 18.2. If it is necessary to investigate the ground below the ground water table, dewatering will be required to obtain a dry excavation. The excavation permits the structure of the ground to be inspected, class 4 samples to be obtained, and field tests to be used to measure the in situ density, strength and deformation characteristics, see clauses 29 and 30.

When boring, there may be difficulty in advancing the boreholes and, consequently, in obtaining samples of adequate quality. Generally, the most suitable method of boring is light cable percussion, see 18.5, using the shell, and supporting the sides of the borehole with casing. Pieces that are too large to enter the shell need to be broken up by the chisel, and this is often a slow process. To reduce the use of the chisel, it is advantageous to use the largest size of boring equipment that is available. Generally, the 200 mm to 250 mm sizes are suitable for gravel, and larger sizes in the range 300 mm to 450 mm may give faster progress in cobbles. Boring in soils containing boulders calls for the alternate use of the chisel and the shell, and can be a very slow operation. Other methods of boring, such as the high-powered machines may sometimes be used to obtain a

faster rate of progress than is obtained by light cable percussion. With each method, it is necessary to employ some technique to support the sides of the borehole, and there may be an appreciable cost from wear of the tools.

None of the sampling methods described in 19.4 to 19.7 is suitable for this type of ground. Disturbed samples taken from the shell are only class 5 (grading incomplete) because the fine fraction has been washed out and the coarse fragments may have been broken up by the use of the chisel.

Borehole tests can be used to obtain an indication of the properties of the ground. The standard penetration test, see 21.2, with the 60° core replaced by the cutting shoe, will give some indication of the relative density in gravel. Occasional high values that are unrepresentative of the true relative density will be obtained when the penetrometer encounters coarse gravel. In ground containing cobbles or boulders, the standard penetration test gives an increasing proportion of unrealistically high results. The borehole permeability test, see 21.4, may give a reasonable indication of permeability, and the results can also be used to give a guide to the proportion of fine particles in the soil. A more reliable assessment of permeability will be obtained from a pumping test, see clause 25. The Dutch cone test, see 23.3, has limitations where there is a significant content of boulders or cobbles. It is also limited because of the inability of the cone to penetrate dense gravel. The static-dynamic test, see 23.4, is useful for this purpose, although its results will also be affected where cobbles or boulders are encountered. The pressuremeter is useful in coarse granular soils, used within a slotted casing, see 21.7.

12.3 Sand. Boreholes in sand are usually sunk by the light cable percussion method using the shell. This requires sufficient water in the borehole to cover the shell, it being necessary to add water in dry ground. The addition of water to the borehole, so that the shell can be used, renders the moisture content of samples unrepresentative of the stratum. Below the water table, some sand tends to 'blow' up the borehole, therefore loosening the ground below the bottom of it. Keeping the borehole full of water, and careful use of an undersized shell can reduce the tendency to blow, but may not completely eliminate it. Disturbed samples taken with the shell are likely to be deficient in fines owing to the action of the water, and therefore only class 5. Samples suitable for particle size distribution, class 4, are usually obtained using the split barrel standard penetration test sampler, see 19.4.5. Larger class 4 samples can generally be obtained using the 100 mm open-tube sampling equipment with a core catcher fitted above the cutting shoe, see 19.4.4.

The action of forcing a sampler into sand tends to cause a change in volume, even if the area ratio is small, see (c) of 19.4.1.2, and hence the density of the sample may not be representative of the stratum. In some cases, a piston sampler will be effective, see 19.5; this should produce class 2 samples or, where the sand is loose or very dense, class 3 samples. However, in both cases the moisture content of the samples may be unrepresentative of the stratum. With clean sand, normal sampling equipment may fail to recover a sample, and it will then be necessary to use the compressed air sand sampler, see 19.7; sample classes will be similar to those obtained with a piston sampler.

A guide to the relative density of sands is obtained by the standard penetration test. However, the results can easily be invalidated by loosening of sand below the water table, as previously discussed. Where it is important that the relative density should not be underestimated, for example when a driven pile project is being investigated, the relative density should be assessed by the Dutch cone test.

Approximate values of the strength and compressibility parameters can be estimated on an empirical basis from the results of the standard penetration test or, preferably, from the results of the Dutch cone test. Pressuremeter tests are also useful. A more direct determination requires the use of plate tests carried out in a dry excavation see 21.6 and clause 29.

Permeability may be assessed from borehole permeability tests, see 21.3, or by pumping tests, see clause 25.

12.4 Silt. Silt is a difficult material to sample and to test, and in considering methods it is sometimes necessary to distinguish between finer grained, cohesive silts and coarse silts, which approach fine sand in behaviour. Boreholes in silt are usually sunk by the light cable percussion method using the shell or clay cutter above the water table. The sample may be deficient in fines owing to the action of the water and therefore only class 5. Except where uniformly coarse, silt is usually sufficiently cohesive to permit the recovery of samples with 100 mm open-tube sampling equipment without requiring a core catcher. Because of the relatively low permeability, these samples usually permit a reliable determination of moisture content, even when water has been added to the borehole. Silts are often sensitive to disturbance during sampling, and hence samples taken with the 100 mm open-tube sampler are generally only class 3. In low or medium density silt, class 2 samples can be obtained using a thin-walled piston sampler, see 19.5.

In preliminary investigations, the standard penetration test is used to obtain an indication of the relative density and generally is alternated with the recovery of 100 mm open-tube samples. Blowing and disturbance caused by the borehole tools can lead to the standard penetration test giving erroneously low results. As with sand, a more reliable result can be obtained using the Dutch cone test. The vane test, see 21.3, is useful for measurement of the undrained shear strength of clayey silts.

12.5 Normally consolidated and lightly overconsolidated clay. Boreholes in clay are usually sunk by the light cable percussion method using the clay cutter where boreholes are dry. Where the borehole contains water, it may be necessary to use the shell. Disturbed samples obtained with the clay cutter are class 4. Disturbed samples recovered with the shell will be class 4 provided that lumps of intact clay can be recovered. Samples taken with an open-tube sampler will suffer disturbance, the degree of disturbance increasing with the sensitivity of the clay to remoulding, and with the depth at which the sample is taken; such samples are therefore class 2. Class 1 samples can usually be obtained by using a thin-walled piston sampler. It should be noted that clays that exhibit permeable fabric may require a water head in the borehole as for sands.

The vane test is useful for the measurement of the undrained shear strength of normally consolidated or lightly overconsolidated clays. Conventional penetrometer tests have limitations in respect of determination of rate of settlement, see table 4. To overcome this difficulty, such tests can be used in conjunction with in situ constant head permeability

measurements. Alternatively, large diameter piston samples should be obtained, either 150 mm or 250 mm diameter, depending on the soil fabric, so that Rowe cell tests can be conducted in the laboratory. Probing may be used to determine the thickness of superficial soft clays and peats.

12.6 Overconsolidated clays. Boreholes in overconsolidated clays are usually sunk by the light cable percussion method. Alternatively, they can be sunk using continuous flight augers, with solid or hollow stems. However, this leads to difficulties if granular materials are also encountered. Rotary core drilling can also be used in overconsolidated clays, but considerable care is required to avoid change of moisture content and disturbance by the drilling fluid.

Samples of clays in the firm to stiff strength range taken with a 100 mm open-tube sampler will usually suffer only a small amount of disturbance, and are generally accepted as class 1. With very stiff and hard clays, however, there will be significant disturbance, particularly if the clays are closely fissured or stony, and samples should be regarded as class 2 or 3. Where the nature of the problem warrants it, more accurate assessment of strength and undrained deformation characteristics requires plate tests in large diameter boreholes, although these can give erroneous results below the water table, see 21.5.

There may be a significant amount of disturbance in laminated clays, particularly if there is no water head in the boreholes. Also, samples of laminated clays soften through migration of porewater from the more permeable laminations into the cohesive laminations.

Piston samples, including large diameter samples up to 250 mm diameter can also be obtained in overconsolidated clays, in the lower part of their strength range.

12.7 Clay containing gravel, cobbles or boulders.

Within the limits of cost, the best method of investigating clay containing gravel, cobbles or boulders is by a dry excavation. The excavation will enable the structure of the ground to be inspected, class 3 samples to be obtained, and field tests for the determination of the in situ density, strength and undrained deformation characteristics to be carried out.

When using borehole methods of investigation, this type of ground presents difficulties in both advancing the boreholes and recovering samples of adequate quality. Using the light cable percussion method, it is often necessary to resort to alternate use of the chisel and shell. Disturbed samples recovered by the shell are class 5 and give a very poor guide to the character of the ground because the coarse particles will have been broken up by the chisel, and much of the fine matrix will have been washed out. It may be possible to recover samples of the clay matrix using the 100 mm open-tube sampler fitted with a strengthened cutting shoe, but the sampler frequently cannot be driven past the coarser particles. Therefore, quite often, the samples recovered are of limited length, have been disturbed by coarse particles, and are only class 4 or 5. It is to be noted, however, that in some boulder clays the gravel content is relatively low, and small in particle size, so that sample disturbance will be less severe.

An alternative borehole method, for use when the clay matrix is stiff or hard, is rotary core drilling, see 18.7.

The core recovery increases with core size. In general, the core size should not be less than about 70 mm diameter (H size) and better results will be obtained with cores of about 100 mm diameter or larger. Considerable care and expertise is required in the operation of the drill. The core samples will give an indication of the structure of the ground, but the moisture content is likely to have been altered by the use of the drill flush. The samples are therefore only likely to be class 4.

The standard penetration test, see 21.2, is useful in tills to give a rough indication of strength, which may be adequate in some problems. Pressuremeter tests can also be useful, see 21.7.

12.8 Made ground. Made ground can consist of replaced natural ground, or waste materials of various origins. The uniformity of made ground will depend on the degree of control which has been imposed, on the quality of the incoming material and on the method of placing and compaction. In most made ground, there has been little or no control of the fill, and the major problem in planning the ground investigation will be to assess the variation in character and quality across the site. Often, the variation will be random.

Conventional methods of boring, sampling and in situ testing, as appropriate to the character of the ground, can give information on the thickness and properties of the made ground at the particular locations of the boreholes or in situ tests. It is essential to ensure that the borehole is always fully cased through made ground to avoid contamination of the natural ground from falling material. Pits and trenches are particularly useful for investigating the nature and variability of made ground, see clause 18.

In combustible fills, temperature measurements may be necessary. It should be noted that on waste tips, burning materials below ground may give rise to toxic or flammable fumes from the borehole. Tip fires may also create voids, which may collapse under the weight of an investigation rig. Lagoons within waste tips may be areas of very soft ground.

12.9 Rock. Boreholes can be sunk in some weaker and weathered rocks by the light cable percussion method, using the clay cutter, shell or chisel, as appropriate to the character of the rock. Disturbed samples from the drill tools will be class 5 if recovered by the shell, and class 4 or 5 if recovered by the clay cutter. In many weak rocks, driven samples can be recovered using the 100 mm diameter sampler fitted with a strengthened cutting shoe. Driving of the sampler generally causes severe disturbance; often the sample is shattered, making it very difficult, if not impossible, to identify the natural structure of the rock. The samples are rarely much better than class 3 or 4. Light cable percussion boring in rock gives results of very limited value, and at best provides a general lithological profile. In many cases, as for example when rock is overlain by boulders, the method cannot identify the interface between soil and rock with any certainty. In such cases, the identity and character of the rock should be confirmed by core drilling or excavations. Rotary core drilling with diamond or tungsten carbide bits is normally used in rock strata, see 18.7. In weathered weak rock and in harder rocks that are closely bedded or jointed, or affected by

faulting, there may be difficulty in recovering cores of satisfactory quality. The use of larger-sized equipment producing cores of about 100 mm diameter or more, and the proper selection of barrel and bit type, will help to improve the core recovery. Where weathered rock occurs at shallow depth, pits are probably the most effective means of investigation; block samples can be obtained for laboratory testing.

Soft infilling of rock discontinuities can sometimes be lost by erosion by the flush water, and air flush drilling may be desirable. In general, rotary core drilling will produce samples which may permit an investigation of the character and engineering properties of the intact rock material and which may give some indication of the frequency of discontinuities but not their orientation. The use of borehole periscopes, impression packers, cameras, and television cameras may be useful in this connexion. In many cases, rotary core samples give no indication of the character of any infilling of the discontinuities. There are techniques available for determining the orientation of cores, and hence of the discontinuities within them.

There are many possible combinations of drilling methods, tools, and core barrel and bit designs in rotary core drilling. The brevity of most site investigations does not allow much experiment to achieve the best results, and attention should be paid to the best method of sampling rock in the design of the investigation.

In a ground investigation using light cable percussion boring or rotary core drilling, an indication of the properties of the rock mass can best be obtained from tests in boreholes. For certain of such tests, however, it will be necessary to take into account the probable effects of disturbance of the ground by the boring process. The standard penetration test, see 21.2, can give a rough indication of the variation of strength and compressibility in weak rock. The permeability test, see 21.4, or the packer or Lugeon test, see 21.5, may give a measure of the mass permeability, which in turn can give an indication of the presence of open joints and other water bearing discontinuities. Where applicable, the plate test, see 21.5, and dilatometers such as the pressuremeter, see 21.7, can be used to investigate deformation properties and possibly also the strength.

The best method for determining the properties of a rock mass, including the orientation of discontinuities, is visual inspection, and field tests carried out in excavations, shafts, or headings, see clause 18.

12.10 Discontinuities. In most rocks, the mass properties depend largely on the geometry and nature of the discontinuities. This can require the engineering properties to be measured in the plane of the discontinuities along specific orientations determined by the anticipated directions of the stresses to be applied. The control by discontinuities over the strength and deformation characteristics of a ground mass is less obvious in soils than in rock, but may be equally important.

No satisfactory drilling or boring techniques are available that ensure the core recovered can be orientated over the full depth penetrated. In soils, the discontinuities are often destroyed by the drilling and therefore overlooked. Where discontinuities are important to the engineering problems involved, in situ exposures of discontinuities are necessary to obtain data on their orientation and

aerial photographs, surface outcrop logging and the drilling of vertical and inclined orientated holes, it may be necessary to undertake full surface exposure, large diameter boreholes, trenches, pits or adits to allow visual inspection around and within the undisturbed ground mass, and measurement of the relevant discontinuity data. In some projects, suitable exposures may be provided in excavations necessitated by the permanent works. The extraction or in situ preparation of orientated test samples can be carried out in these excavations together with orientated large scale tests. The orientation of the excavations controls their intersection with the discontinuities and, consequently, the discontinuity data that can be obtained. Generally, three orthogonal exposures are required to define fully the spatial distribution of the discontinuities. The extent of the excavations is governed by the spacing between discontinuities and the size of the works, see 41.2.3.

13. Aggressive ground and ground water

13.1 Introduction. In certain localities, ground water, soil and rock may contain constituents in amounts sufficient to cause damage to Portland cement concrete, particularly thin members. The constituents principally concerned are sulphates, which are most common in clay soils, and acidic waters, which are found in or derived from peaty soils.

Some types of ground have a corrosive action on metals, particularly on cast iron, owing to electrolytic or other chemical or bacteriological agencies. In industrial areas, corrosive action may arise from industrial waste products that have been dumped on the site. In river and maritime works, the possible corrosive action of soft water, sea water and other saline waters and of trade effluents may also require investigation. In a marine environment, the most severe corrosion is found in the zone that is occasionally wetted. This effect is increased in waters with a high tidal range. In estuarine situations, there may be an adverse condition because of alternation of water of different salinities. Generally, disturbed soils are more aggressive than undisturbed soils.

13.2 Investigation of potential deterioration of concrete.

Laboratory tests to assess the aggressiveness of the ground and ground water against Portland cement concrete include determination of pH value and sulphate content, see soil chemical tests in table 4.

13.3 Investigation of potential corrosion of metals.

Local enquiry is desirable in order to ascertain whether corrosion of metals has previously occurred. The likelihood of corrosion can be assessed from geophysical testing, see 33.5, and from chemical tests on undisturbed specimens, which should be placed in air-sealed containers immediately after sampling, see soil corrosivity tests in table 4. For specimens for bacteriological tests, the containers should be sterilized.

13.4 Investigation of made ground containing industrial waste products.

Industrial waste products can contain a wide range of chemicals, depending on the industrial processes from which the waste products are derived. Some chemicals are highly aggressive to concrete or metal in underground structures, and some can be highly obnoxious or even poisonous. The last two characteristics can present major constructional problems, e.g. the disturbance or disposal of contaminated ground, or the

disposal of contaminated ground water. Local enquiries may give some indication of the origins of the waste materials, and the pH value and sulphate content for the made ground and the ground water will generally give some indication of the magnitude of the contamination. It may then be necessary to carry out a detailed chemical study of the ground conditions.

14. Ground investigations over water

14.1 Introduction. Sinking boreholes below water presents special difficulties in comparison with working on dry land. A reasonably stable working platform may be provided, such as a staging, barge or ship, and the borehole sunk through a conductor pipe spanning between the working platform and the water bottom. Increasing use is being made of a variety of penetration testing techniques. In some cases, it may be feasible to lower specially designed boring, drilling or penetration testing equipment to the water bottom to be operated by remote control or a diver. With remote control, operation is restricted to a single continuous operation. Penetration depths vary from less than 5 m for some devices, to 20 m or more for others. Geophysical techniques are also used to augment the information obtained from borings, or as a preliminary investigation before putting down borings.

The scope of the work, including the methods of boring, sampling and in situ testing, requires careful consideration depending on the particular difficulties of the site. When working over water it is essential that due consideration be given to safety requirements, navigational warnings, and the regulations of governmental departments and other authorities.

Ground investigations conducted from above water are often more expensive and time consuming than comparable investigations conducted on dry land, and there may be a temptation to economize by reducing the scope of the investigation. The extent of the requirement for ground investigations should be realistically assessed, since economies in this direction can turn out to be false.

14.2 Stages and platforms. Where stable working platforms are available or can be provided, such as oil drilling platforms and jetties or purpose-built scaffold stages and drilling towers, it is generally possible to use conventional dry-land site investigation boring equipment and conventional methods of sampling and in situ testing. When working from existing structures, it may be necessary to construct a cantilever platform over the water on which to mount the boring rig. When boring close to the shore in relatively shallow water, it may be most convenient to construct a scaffold staging from land to the borehole location. Alternatively, it may be more economical to construct a scaffold or other tower at the borehole location, in which case some means of transporting the boring equipment to the tower will need to be provided. Some towers are so constructed that they can be moved from one borehole location to another without having to be dismantled.

Jack-up platforms, and special craft fitted with spud legs, can be floated into position and then jacked out of the water to stand on their legs. They can combine manoeuvrability with fulfilment of the requirement for a fixed working platform.

The design of all staging, towers and platforms should take into account the fluctuating water levels due to tides,

waves and swell conditions and it is essential that such constructions be sufficiently strong for the boring operations to resist waves, tidal flow and other currents and floating debris.

14.3 Floating craft. The type of floating craft suitable as a boring vessel depends on whether the water is sheltered or open; the anchor holding properties of the sea bed; whether accommodation for the personnel is required on board; the likely weather conditions; the depth of water; and the strength of currents. In inland waters, a small anchored barge may suffice, but in less sheltered waters a barge should be of substantial size, and anchors will require to be correspondingly heavy. In off shore conditions, a ship is often employed, and it may then be possible to accommodate the personnel on board, with a saving in auxiliary supply vessels.

An auxiliary vessel will be required to handle the moorings if a barge is used for the work, but in certain cases a ship may be able to lay and pick up its own moorings. Generally, four or six point moorings will be required, and anchors should have the best holding capacity feasible. In water deeper than 80 m, conventional moorings become difficult, and the use could be considered of vessels maintained in position by computer-controlled thrusting devices.

Special techniques are required to deal with fluctuating water levels due to tides, waves and swell conditions, particularly with rotary drilling, where a constant pressure between the drill bit and the bottom of the borehole is required. Because of the special equipment employed when boring from floating craft, and also the movement of the working platform, the types of sample to be recovered and in situ tests to be performed require particular consideration.

14.4 Working between high and low tide levels. Sinking boreholes between high and low tide levels may be achieved using scaffold stagings or platforms, see 14.2, by using flat-bottomed pontoons or by moving boring rigs to the location during periods permitted by the tides.

14.5 Setting-out and locating borehole positions. Close in shore, it is often possible to set-out boreholes satisfactorily by using sextant observations to features on shore or by lining in from previously placed shore markers. The accurate location of the boreholes can then be determined using theodolite observations. Further off shore, it is necessary to use electronic methods of position fixing. Electronic methods are also advantageous in bad visibility conditions.

14.6 Determination of reduced level of bed and strata boundaries. Reduced levels are generally transferred to a boring vessel from shore by setting up a tide gauge close in shore which is read at frequent intervals throughout the tidal cycles at the same time as readings of water depth are taken on the boring vessel. Corrections may be necessary to allow for tidal variations when the distance between the tide gauge and the vessel is significant.

15. Personnel for ground investigation

15.1 Introduction. In view of the importance of ground investigation as a fundamental preliminary to the proper design and efficient and economical construction of all civil engineering and building works, it is essential that personnel involved in the investigation should have appropriate specialized knowledge and experience, and be familiar with the purpose of the work.

15.2 Direction, planning and execution. The person directing and planning the ground investigation should be a suitably qualified senior engineer who has considerable experience and competence in geotechnical engineering. He should determine the extent and adequacy of the investigation and direct the investigation both in the field and in the laboratory, and finally assess the results in relation to the design of the proposed works. He may delegate part of these duties to geotechnical specialists who are on his staff or who act as consultants or contractors.

The person responsible for the execution of the ground investigation may be on the staff of the directing engineer, or may be a geotechnical consultant or contractor. He should be a suitably qualified senior engineer or engineering geologist who has considerable experience and competence in geotechnical engineering and particularly in carrying out ground investigation and in the interpretation of the results.

15.3 Supervision in the field. The supervision of the work in the field should be the full time or part-time responsibility, depending on the size of the investigation, of a suitably qualified and experienced civil engineer or engineering geologist. He may be assisted by:

- (a) assistant civil engineers or engineering geologists;
- (b) field engineers and senior field technicians who are skilled in the work described in 15.7 and who are competent to supervise such work by others, and in addition may be required to supervise work described in 15.8.1 and 15.8.2;
- (c) drilling supervisors who are skilled in the work described in 15.8.1 and who are competent to supervise such work by others, and to supervise the work described in 15.8.2.

15.4 The logging of excavations and boreholes and the describing of soils and rocks. The driller should normally be responsible for recording, as it arises, the information obtained from the borehole, see 15.8.1 and clause 39; this should include a measured record of strata, with simple soil and rock descriptions. Subsequently, detailed engineering descriptions of all the soil and rock samples obtained, see section eight, should be made by a suitably qualified and experienced civil engineer or engineering geologist. On more extensive investigations, it is preferable to do this on site, and proper facilities should be provided. For smaller investigations, the description may be made in the laboratory. (The incorporation of these engineering descriptions in the borehole logs will form part of the report preparation.)

For rotary cored boreholes, it may sometimes be necessary to have the full-time attendance on site of an engineering geologist so that the rock cores may be logged and described in their fresh condition, as the work proceeds. Where trial pits and other exploratory excavations are required, and where existing natural or man-made exposures of the ground are concerned, the detailed recording of the soils and rocks, their stratification, structure and fabric should be made by a suitably qualified and experienced civil engineer or engineering geologist.

15.5 Laboratory work. The testing of soil and rock samples should be carried out in a laboratory approved by the engineer, under the control of a suitably qualified and experienced supervisor, see section six. Laboratory

and experience in the type of test they are conducting.

15.6 Interpretation. The preparation of the report, including the factual information obtained and the engineering interpretation of it, is described in section seven. This work should be prepared by suitably qualified and experienced civil engineers or engineering geologists; the engineering interpretation should be made by the senior civil engineer directing the investigation or be delegated by him to a suitably qualified civil engineer. Except where the geology is simple, the geological aspects should be reviewed by a suitably qualified and experienced engineering geologist.

15.7 Field technicians. Field technicians who carry out special sampling and testing in boreholes, piezometer installations and probing (described in section four) and field tests (described in section five) should have received training and have skill and experience in the work.

15.8 Operatives

15.8.1 Drillers. The driller in charge of an individual drilling rig should be skilled in the practice of exploration of the ground by means of boreholes, simple sampling and testing, and in making ground water observations in boreholes, and properly recording the information obtained.

15.8.2 Operators of excavating plant. Operators of excavating plant should be skilled and experienced in its safe use for digging trial pits and trenches. Any timbering or other support required in excavations should be installed by skilled operatives.

16. Review during construction

16.1 Introduction. There is an inherent difficulty in forecasting ground conditions from ground investigations carried out before the works are started since, however intensive the investigation and whatever methods are used, only a small proportion of the ground is examined.

16.2 Purpose. The primary purpose of the review during construction is to determine to what extent, if any, in the light of the conditions newly revealed, conclusions drawn from the ground investigation are required to be revised.

In some cases, additional information is found which may necessitate amendment of the design or the construction procedures. In certain cases, it may therefore be appropriate to initiate a site procedure in the early stages of the contract, so that correct and agreed records are kept during the duration of the contract by both the engineer and the contractor. The purposes of the records are:

- (a) to check the adequacy of the design;
- (b) to check the safety of the works during construction and to assess the adequacy of temporary works;
- (c) to check the findings of the ground investigation and to provide a feed-back so that these findings may be reassessed;
- (d) to check initial assumptions about ground conditions, including ground water, related to construction methods;
- (e) to provide agreed information about ground conditions in the event of dispute;
- (f) to check the suitability of proposed instrument installations;
- (g) to enable the fresh use to be made of excavated materials;
- (h) to reassess the initial choice of construction plant and equipment.

16.3 Information required

16.3.1 Soil and rock. Accurate engineering descriptions of all strata encountered below ground level should be made in accordance with section eight. The soil profile revealed on site should be recorded and compared with that anticipated from the ground investigation. The descriptions should be made by a civil engineer or engineering geologist competent in geotechnics. It may be advantageous to arrange for the site to be inspected by the organization that carried out the site investigations, particularly if conditions appear to differ significantly from those described in the ground investigation.

16.3.2 Water. It is most important to record accurately all information about the ground water obtained during construction, to compare with information recorded during the investigation. The information should include the flow and static conditions in all excavations, any seepage from slopes, any seasonal variations, any tidal variations in excavations or tunnels near the sea or estuaries, suspect or known artesian conditions, the effect of weather conditions on ground water, and any unforeseen seepage under or from water-retaining structures. The effect of ground water lowering should also be recorded in observation holes to determine the extent of the cone of depression.

16.4 Instrumentation (see also 31.2). On many types of structures, such as earth dams, embankments on soft ground, some large buildings with underground construction, excavations and tunnels, it is prudent to consider regular observations by means of instrumentation in order to check that construction works can proceed safely. Such observations include measurement of pore pressure, seepage, earth pressure, settlement and lateral movements. The instrumentation may be usefully continued after construction in order to observe the performance of the project. This is particularly necessary in the case of earth dams for maintaining a safe structure under varying conditions, and in other cases for gaining valuable data for future design.

Section four. Excavation, boring, sampling, probing and tests in boreholes

17. Introduction

There is a considerable variety of methods of ground investigation, and normally a combination of methods is employed to cover the technical requirements and the range of ground conditions that are encountered. The factors involved in the selection of methods are discussed in section three. The collection and recording of data to be obtained is discussed in section seven. In situ tests that are carried out in boreholes as part of a borehole investigation are described in clause 21. Other in situ tests, for which a borehole either is unnecessary or is only an incidental part of the test procedure, are described in section five.

In planning ground investigations, particular attention should be paid to the safety of personnel. Certain methods present special safety problems, and recommendations are given in the relevant clauses. Other methods involve normal safety precautions appropriate to site or laboratory work.

A list of statutory regulations which may apply to ground investigations is given in appendix F; this list is not necessarily complete, and if there is doubt over safety precautions, further advice should be obtained.

Since backfilled pits and boreholes might interfere with subsequent construction, they should be sited and backfilled with care. It is essential that the precise location of every excavation, boring or probing is properly recorded during the execution of the fieldwork. It is equally essential that the ground levels of these locations be established and recorded. The records should be such that the locations and levels can be readily incorporated into the report on the investigations, see 10.5 and 40.2.8.

Should it appear during the course of ground exploration that buried structures of archaeological interest have been encountered, the matter should be reported to the Chief Inspector of Ancient Monuments at the address given in 5.8.

18. Excavations and boreholes

18.1 Shallow trial pits. Shallow trial pits are usually dug using an hydraulic back-hoe excavator, preferably mounted on a tractor for ease of mobility. This expedient is used in ground that will temporarily stand unsupported, and in suitable conditions the maximum depth of excavation is about 5 m. Where personnel are required to enter pits, it is essential that the sides are safe, particularly against sudden collapse; this is usually done by supporting the sides. Ideally, the support system should consist of purpose-made steel frames that can quickly be inserted and extracted. Alternatively, it may be possible to excavate the sides to a safe profile by means of a series of benches. The safety aspect is discussed more fully in BS 6031.

Shallow trial pits permit the in situ condition of the ground to be examined in detail both laterally and vertically; they provide access for taking samples and for carrying out in situ tests; and they provide a means of determining the orientation of discontinuities in the ground. When a pit is used for the last-mentioned purpose, the field record should include a plan giving the location and orientation of the pit, and a dimensioned section of each side and the floor. Shallow trial pits can readily be extended into trenches in order to trace any particular feature, and in suitable ground this method is very efficient and economical.

Shallow pits without side support can be used for making a rapid check on the condition of the ground. It may be unsafe for personnel to enter a pit but, working from ground surface, the technician can prepare a visual log of the strata and take disturbed samples using the excavator bucket. Tube samplers can be driven into the floor of the pit using a jarring link and drill rods and extracted by the excavator; in situ testing, such as the vane shear strength test, can also be carried out. Pits that are unsupported may collapse soon after being dug; therefore, any logging, sampling and in situ testing should be carried out immediately after the pit has been dug.

It is advisable to backfill pits as soon as possible after logging, sampling and testing have been completed since open pits can be a hazard to the general public. Recommendations on backfilling are given in 18.9. Any pits that have to be left open and unattended should be securely fenced off.

18.2 Deep trial pits and shafts. Deep trial pits and shafts are normally constructed by hand excavation using traditional methods for supporting the sides. In suitable ground conditions, shafts approximately one metre in diameter can be bored using large power-driven augers. Temporary liners are used to support the sides in unstable ground, and to provide the necessary protection for personnel working in the shafts. However, when making inspections, it is necessary to expose as much of the ground as possible, and considerable judgement and experience is required, since the excessive use of liners can lead to delay and to additional danger as a result of the build-up of water pressure behind the liners. Suitable ground conditions are those that can be bored with a minimum use of temporary liners and, in these conditions, augered shafts provide a fast and economical expedient for inspection, sampling and testing in situ. For many types of ground, excavations below the water table present serious problems both in maintaining a dry investigation and in stabilizing the sides. For such circumstances, the water table may be the maximum depth for which this method is feasible.

Working in deep shafts will be dangerous unless the appropriate safety precautions are strictly followed. Attention should be given to the possible occurrence of injurious or combustible gases or of oxygen deficiencies, and the correct inspections and precautions established, see clause 17 and appendix F. Oxygen-consuming engines that emit toxic exhaust fumes, such as petrol-driven pump motors, should not be employed in shafts.

18.3 Headings (adits). Headings are driven from the bottom of shafts or laterally into sloping ground, and can be used for the in situ examination of the strata, existing foundations and other underground constructions, and also for carrying out special sampling and in situ testing. One important use of headings is in the inspection of abutments of dams. In soil and many types of rock, the sides and roof of the heading require support.

Driving a heading below the water table presents a construction problem in strata that are not naturally self-supporting during the period required to erect support timbers. Below the water table, it is probable that headings are an economical means of ground investigation expedient only in rock and stiff impermeable clay. In other cases, one of the methods described in section six of CP 2004 : 1972 may be suitable.

18.4 Hand auger boring. The hand auger boring method uses light hand-operated equipment. The auger and drill rods are usually lifted out of the borehole without the aid of a tripod, and no borehole casing is used. Boreholes up to 200 mm diameter may be made in suitable ground conditions to a depth of about 5 m. The method can be used in self-supporting strata without hard obstructions or gravel-sized to boulder-sized particles. Hand auger boreholes can be used to obtain disturbed samples and small open-tube samples and for ground water observations.

18.5 Light cable percussion boring. Light cable percussion boring is an adaptation of standard well-boring methods, and generally uses a mobile rig specially designed for ground investigation work. For most investigations, the rig has a winch of 1 t to 2 t capacity that is driven by a diesel engine, and a derrick of about 6 m in height. With many types of rig, the legs of the derrick fold down to form a simple trailer that can be towed by a light vehicle. The clay

cutter is used in a dry borehole. The shell can only be used when there is sufficient water in the bottom of the borehole to cover the lower part of the shell. Hence, it will be necessary to add water to a borehole in order to bore through dry cohesionless strata which require the use of the shell.

The above methods may conflict with the geotechnical objects of the investigation, which may require the borehole to be drilled either in a dry condition or with the water level in the borehole maintained at or above the natural ground water level. In such cases, it may be necessary to adopt a less efficient method of boring with the cable percussion rig, e.g. boring a stiff clay under water might require the use of the chisel and shell, or to adopt a different method of drilling, e.g. mechanical augering.

Light cable percussion boring is suitable for soil and weak rock. The sizes of borehole casings and tools are generally 150 mm, 200 mm, 250 mm, and 300 mm, giving a maximum borehole depth of about 60 m in suitable strata. The type of rig may have an hydraulic power take-off to drive a rotary drilling attachment for coring rock. The drill tools, worked on a wire rope using the clutch of the winch for the percussive action, consist of the clay cutter, for dry cohesive soils; the shell, or baler, for cohesionless soils; and the chisel, for breaking up rock and other hard layers. The clay cutter and shell bring up disturbed material which is generally sufficiently representative to permit identification of the strata.

18.6 Mechanical augers. Mechanical augers normally use a continuous-flight auger with a hollow stem; they are suitable for augering in cohesive soils. When augering, the hollow stem is closed at its lower end by a plug, which may be removed so that the sampler can be lowered down through the stem and driven into the soil below the auger bit. The use of hollow-stemmed augers in cohesionless soils often presents practical problems because it is difficult to prevent material from flowing into the hollow stem on removal of the plug. When rock is encountered, boring can be extended by core-drilling through the hollow stem. Typically, augers with hollow stems of approximately 75 mm and 125 mm diameter produce boreholes of about 150 mm and 250 mm diameter respectively, to a depth of 30 m to 50 m. Continuous-flight augering requires considerable mechanical power and weight so that the machine is generally mounted on a heavy vehicle. The debris from drilling is brought to the surface by auger flights, and gives only a very rough indication of the levels and character of the strata. A precise intermittent identification of the strata may be obtained from drive samples taken through the hollow stem of the auger.

In self-supporting strata, solid rods and a suitable auger tool can be used, the auger tool being drawn up to the ground surface each time it has to be emptied. Drive sampling and testing can be carried out in the borehole.

18.7 Rotary open hole drilling and rotary core drilling. Rotary open hole drilling and rotary core drilling are the traditional methods of drilling for rock exploration in which the drill bit is rotated on the bottom of the borehole. The drilling fluid, which is pumped down to the bit through hollow drill rods, lubricates the bit, and flushes the drill debris up the borehole. The drilling fluid is most commonly water, but air or drilling mud may be used.

There are two basic types of rotary drilling: open hole (or full hole) drilling, in which the drill bit cuts all the material within the diameter of the borehole; and core drilling, in which an annular bit fixed to the bottom of the outer rotating tube of a rotary core barrel cuts a core, which is returned within the inner stationary tube of the core barrel and brought to the surface for examination and testing. Drill casing is normally used to support unstable ground or to seal off open fissures which cause a loss of drilling fluid. Alternatively, drilling mud or cement grouting can be used. Rotary drilling for ground investigation is usually core drilling. Open hole drilling is sometimes used in soil or weak rock, and equipment for taking soil samples or for making in situ tests is used in between runs of the open hole bit. The value of open hole drilling is much enhanced if the borehole records are prepared by an engineer or engineering geologist who is continually in attendance on the drill.

For rotary core drilling, the choice of a drill and suite of compatible equipment and their condition for the work required are most important. The drill should be well maintained, be smooth running and mounted such that minimum vibration or movement is transmitted by the drill string to the core barrel. The rotational speed and axial load on the core bit has to be adjusted to suit a number of factors, including rock hardness and core size. The use of the larger core sizes in strong rock often demands a considerable bit loading, necessitating a heavy drill rig with ballast or holding down belts. The drill rods should be straight and of a large enough diameter to prevent excessive whip, to ensure stability of the drill string, and to maintain the uphole velocity of the flushing medium.

Careful selection of a flushing medium, to be compatible with the equipment employed and suitable for the ground to be drilled, is very important. Water is the simplest and most commonly used flushing medium. Drilling muds consisting of water with clay or bentonite; water with an additive such as sodium chloride; foams; and polymer mixtures are also used as flushing media. Drilling muds, and water with additives, have the advantage over water alone in that the cuttings may be removed at a lower flushing velocity, thereby reducing disturbance and washing out at the cutting bit and within the hole, in weak or uncemented formations. Air may be used as a flushing medium with advantage in some rock conditions or where limitations are made on the use of muds in respect of aquifer contamination. However, whereas water and mud may tend to increase the natural moisture content of the core, air may tend to reduce it.

Drilling is in part an art, and its success is dependent upon good practice and the skill of the operator, particularly when coring partially cemented, fractured, weathered and weak rocks, where considerable expertise is necessary to obtain full recovery of core of satisfactory quality. This is greatly influenced by the choice of core barrel and cutting bit type and configuration, see 19.8 and by the method of extruding, handling and preservation of the core, see 19.10.5.

18.8 Wash boring and other methods

18.8.1 Wash boring. Wash boring is best suited to sands, silts and clays and is normally carried out using 65 mm borehole tools and borehole casing. Wash boring has been widely used in North America, but not often in Britain.

The drill rig consists of a simple winch and tripod. The ground at the bottom of the borehole is broken up by the percussive action of a chisel bit, and washed up to the surface by water that is pumped down the hollow drill rods. In collapsing ground, casing is driven down to support the sides of the borehole, or drilling mud may be used. The fragments of soil brought to the surface by the wash water return are not representative of the character and consistency of the strata that are being penetrated. These properties are determined by carrying out standard penetration tests at the top of each stratum and at frequent intervals of depth. Open-tube samples or piston samples can be taken in cohesive strata.

18.8.2 Other methods of boring. There are many methods of boring, which have been developed generally to obtain maximum penetration speed, e.g. rotary percussive drilling for blast holes and grouting. When such boreholes are sunk for other purposes, limited information about ground conditions may be obtained, provided that the boreholes are drilled under controlled conditions, with measurement of rate of penetration, observation of drilling characteristics, and sampling of the drilling flushings [12].

18.9 Backfilling excavations and boreholes. Poorly-compacted backfill will cause settlement at the ground surface and can act as a path for ground water. The latter effect can cause very serious inconvenience if the backfilled excavations or boreholes are on the site of future deep excavations, tunnels or water-retaining structures. It could also lead to the future pollution of an aquifer. For boreholes in dry ground, it is possible to use compacted soil as a backfill, although the procedure is often unsuccessful in preventing the flow of water. The best procedure is to refill the borehole with a cement-based grout introduced at the lowest point by means of a tremie pipe. Cement alone will not necessarily seal a borehole, on account of shrinkage, and it is often preferable to use a cement: bentonite grout, e.g. about 4 : 1 mixed with no more water than is necessary to permit the grout to flow or to be pumped. The addition of an expanding agent may be necessary. It is possible to compact the backfill of excavations by means of the excavator bucket or other mechanical means. In special cases, weak concrete may be used.

19. Sampling the ground

19.1 General principles. The selection of sampling technique depends on the quality of the sample that is required, and the character of the ground, particularly the extent to which it will be disturbed by the sampling process.

It should also be borne in mind that the behaviour of the ground in the mass is often dictated by the presence of weaknesses and discontinuities. Therefore, it is possible to obtain a good sample of material which may be unrepresentative of the mass. In choosing a sampling method, it should be made clear whether it is the mass properties or the intact material properties of the ground that are to be determined, see 12.10.

There are four main techniques for obtaining samples [13]:

- (a) taking disturbed samples from the drill tools or from excavating equipment in the course of boring or excavation, see 19.3;

(b) drive sampling, in which a tube or split tube sampler having a sharp cutting edge at its lower end is forced into the ground either by a static thrust or by dynamic impact, see 19.4 to 19.7;

(c) rotary sampling, in which a tube with a cutter at its lower end is rotated into the ground, thereby producing a core sample, see 19.8;

(d) taking block samples specially cut by hand from a trial pit, shaft or heading, see 19.9.

Samples obtained by techniques (b), (c) and (d) will often be sufficiently intact to enable the ground structure within the sample to be examined. The quality of such samples can vary considerably, depending on the technique and the ground conditions, and most will exhibit some degree of disturbance. A method for classifying the quality of the sample is given in 19.2. 19.3 to 19.9 describe the various sampling techniques and give an indication of the sample qualities that can be expected. Intact samples obtained by techniques (b), (c) and (d) are usually taken in a vertical direction, but specially orientated samples may be required to investigate particular features.

19.2 Sample quality. The sampling procedure should be selected on the basis of the quality of the sample that is required, and is assessed largely by the suitability of the sample for appropriate laboratory tests. A classification for soil samples developed in Germany [14] provides a useful basis for classifying samples in terms of quality.

Quality	Properties that can be reliably determined
Class 1	Classification, moisture content, density, strength, deformation and consolidation characteristics
Class 2	Classification, moisture content, density
Class 3	Classification, moisture content
Class 4	Classification
Class 5	None (sequence of strata only)

Laboratory classification tests are relevant only to soils, but the quality classification may usefully be applied to rock samples in respect of other properties. Laboratory testing of samples is discussed in section six.

In some cases, whatever sampling methods are used, it will only be possible to obtain samples with some degree of disturbance, i.e. class 2 at best. The results of any strength or compressibility tests carried out on such samples should be treated with caution. Samples of classes 3, 4 and 5 are commonly regarded as 'disturbed samples'.

A further consideration in the selection of procedures for taking class 1 samples is the size of the sample. This is determined largely by the structure of the ground, which, for soil, is often referred to as 'the fabric' [15]. Where the ground contains discontinuities of random orientation, e.g. in a clay fill, the sample diameter, or width, should be as large as possible in relation to the spacing of discontinuities. Alternatively, where the ground contains strongly orientated discontinuities, e.g. in jointed rock, it may be necessary to take samples which have been specially orientated. For fine soils that are homogeneous and isotropic, samples as small as 35 mm in diameter may be used. However, for general use, samples 100 mm in diameter are preferred since the results of laboratory tests may be then more representative of the ground mass. In special cases, samples 150 mm and 250 mm in diameter are used [15].

BS 1377 and BS 1924 give precise details of the mass of sample required for each test specimen. Where the approximate number of tests is known, it is a simple matter to estimate the total amount of soil that has to be obtained. Where the programme of laboratory tests is uncertain, table 1 gives some guidance on the amount of soil that should be obtained for each series of tests. Where in situ materials are being considered, details of the size of sample required are given in BS 812 : Part 1.

Table 1. Mass of soil sample required for various laboratory tests

Purpose of sample	Soil type	Mass of sample required
Soil identification, including Atterberg limits; sieve analysis; moisture content and sulphate content tests	Clay, silt, sand	1 kg
	Fine and medium gravel	5 kg
	Coarse gravel	30 kg
Compaction tests	All	25 kg to 60 kg
Comprehensive examination of construction materials, including soil stabilization	Clay, silt, sand	100 kg
	Fine and medium gravel	130 kg
	Coarse gravel	160 kg

19.3 Disturbed samples from boring tools or from excavating equipment. The quality of the sample depends on the technique used for sinking the borehole or excavation and on whether the ground is dry or wet. When disturbed samples are taken from below water in a borehole or excavation, there is a danger that the samples obtained may not be truly representative of the deposit. This is particularly the case with non-cohesive soils containing fines, which tend to be washed out of the tool. This can be partly overcome by placing the whole contents of the tool into a tank and allowing the fines to settle before decanting the water.

The following classes of sample can generally be expected from the various methods of boring and sampling:

Class 3: disturbed samples from dry excavations and from dry boreholes sunk either by a clay cutter using cable percussion equipment or by an auger.

Class 4: disturbed samples obtained in cohesive soil from excavations, or from boreholes sunk either by a clay cutter using cable percussion equipment, or by an auger, in conditions where water is present.

Class 5: disturbed samples in non-cohesive soil from wet excavations or from any borehole sunk by a shell using cable percussion equipment or from any borehole sunk by a method in which the drill debris is flushed out of the borehole, e.g. rotary wash boring.

The mass of sample required for various purposes is determined by the character of the ground and the tests that are to be undertaken. A guide to the mass of soil sample required for various purposes is given in table 1. Care should

be taken that the sample is representative of only the stratum from which it comes, and has not been contaminated by other strata.

19.4 Open-tube samplers

19.4.1 Principles of design

19.4.1.1 General. Open-tube samplers consist essentially of a tube that is open at one end and fitted at the other end with means for attachment to the drill rods. A non-return valve permits the escape of air or water as the sample enters the tube, and assists in retaining the sample when the tool is withdrawn from the ground. Figure 1 shows the basic details of a sampler suitable for general use having a single sample tube and simple cutting shoe.

The use of sockets and core catcher is discussed in 19.4.4. An alternative sampler incorporates a detachable inner liner.

The fundamental requirement of a sampling tool is that it should cause as little remoulding and disturbance as possible on being forced into the ground. The degree of disturbance is controlled by three features of the design: the cutting shoe, the inside wall friction and the non-return valve.

19.4.1.2 The cutting shoe. The cutting shoe should normally be of a design similar to that shown in figure 1; it should embody the following features.

(a) *Inside clearance.* The internal diameter of the cutting shoe, D_C , should be slightly less than that of the sample tube, D_S , to give inside clearance, typically about 1 % of the diameter. This allows for slight elastic expansion of the sample as it enters the tube, reduces frictional drag from the inside wall of the tube and helps to retain the sample. A large inside clearance should be avoided since it would permit the sample to expand, thereby increasing the disturbance.

(b) *Outside clearance.* The outside diameter of the cutting shoe, D_W , should be slightly greater than the outside diameter of the tube, D_T , to give outside clearance and facilitate the withdrawal of the sampler from the ground. The outside clearance should not be much greater than the inside clearance.

(c) *Area ratio.* The area ratio represents the volume of soil displaced by the sampler in proportion to the volume of the sample, see figure 1. It should be kept as small as possible consistent with the strength requirements of the sample tube. The area ratio is about 30 % for the general purpose 100 mm diameter sampler, and about 10 % for a thin-walled sampler. Some special samples have a large outside diameter D_T , relative to the internal diameter D_C , e.g. in order to accommodate a loose inner liner. The sampling disturbance is reduced by using a cutting shoe which has a long outside taper, and is considerably less than that which would be expected from the calculated area ratio.

19.4.1.3 Wall friction. This can be reduced by a suitable inside clearance, and by a clean, smooth finish to the inside of the tube.

19.4.1.4 Non-return valve. The non-return valve should have a large orifice to allow air and water to escape quickly and easily when driving the sampler, and to assist in retention of the sample when removing the sampler from the borehole.

Typical designs of open-tube samplers which are used for various purposes are described in 19.4.3 to 19.4.5.

19.4.2 Sampling procedure. Before a sample is taken, the bottom of the borehole or surface of the excavation or heading should be cleared of loose or disturbed material as far as possible. Some or all of any such loose or disturbed material that is left will normally pass into the overdrive space.

Below the water table, certain types of laminated soils occurring below the bottom of the borehole or excavation may be disturbed if the natural water pressure in the laminations exceeds the pressure imposed by the water within the borehole or excavation. To prevent this effect, it is necessary to keep the level of the borehole water above the ground water level appropriate to the location of the sample.

The sampler can be driven into the ground by dynamic means, using a drop weight or sliding hammer, or by a continuous static thrust, using a hydraulic jack or pulley block and tackle. There is little published evidence to indicate whether dynamic or static driving produces less sample disturbance, and for most ground conditions it is probable that there is no significant difference. The driving effort for each sample may be recorded as an indication of the consistency of the ground.

The distance that the tool is driven should be checked and recorded as, if driven too far, the soil will be compressed in the sampler. A sampling head with an 'overdrive' space, see figure 1, will allow the sample tube to be completely filled without risk of damaging the sample. After driving, the sampler is steadily withdrawn. The length of sample that is recovered should be recorded, compared with the distance that the tool was driven, and any discrepancy investigated. For example, if the length of the sample is less than the distance driven, the sample may have experienced some compression or, alternatively, the sample tool may have permitted the sample to slip out as the tool was being withdrawn.

19.4.3 Thin-walled samplers. Thin-walled samplers are used for soils that are particularly sensitive to sampling disturbance, and consist of a thin-walled steel tube whose lower end is shaped to form a cutting edge with a small inside clearance. The area ratio is about 10 %. These samplers are suitable only for fine soils up to a firm consistency, and free from large particles. They generally give class 1 samples in all fine cohesive soils, including sensitive clays, provided that the soil has not been disturbed by sinking the borehole. Samples between 75 mm and 100 mm in diameter are normally obtained; samples up to 250 mm in diameter are often obtained for special purposes. It is to be noted that disturbance at the base of the borehole will occur in weak soil below a certain depth because of stress relief. Piston samples penetrating well below the base of the borehole are therefore preferable, see 19.5. A typical thin-walled sampler is illustrated in figure 2.

19.4.4 General purpose 100 mm diameter open-tube sampler. The 100 mm diameter open-tube sampler is the standard sampler for routine use with cable percussion boring, and can be used in all cohesive soils and in weak rock, such as Chalk and weathered Keuper Marl. In non-sensitive fine cohesive soils of stiff or lower consistency, the equipment may give class 1 samples or, more likely, class 2 samples. In sensitive clay, class 2 samples may be obtained. In brittle or closely fissured materials, such as certain weak rock and hard clay,

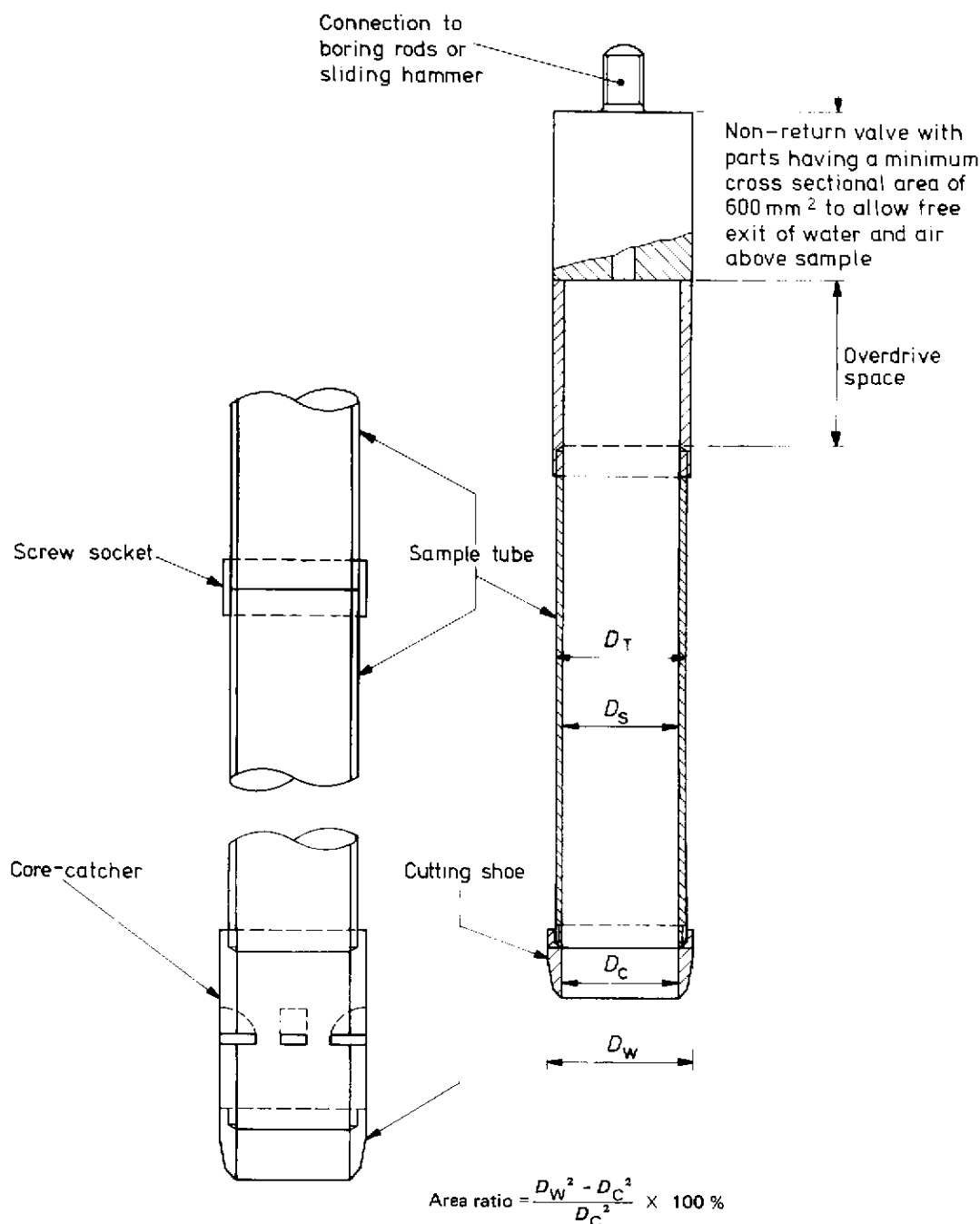


Figure 1. Basic details of open-tube sampler

and also in stoney materials, the sampler gives class 3 samples; this is because sampling disturbs these materials, therefore reducing sample quality to class 3 or class 4 if water has been added to the borehole.

The sampler is illustrated in figure 1 and consists of a sample barrel, about 450 mm in length, with a screw-on cutting shoe and drive head. The area ratio is about 30 %. Sample barrels can be coupled together with screw sockets to form a longer sampler. Two standard barrels, forming a sampler about 1.0 m in length, are often used for sampling soft clays [16], although the increased length of the sample tube may lead to some disturbance.

In soils of low cohesion, such as silt and silty fine sand, the sample may fall out when the tool is withdrawn from

a core-catcher between the cutting edge and the sample barrel, as shown in figure 1. When using a core-catcher, the sample quality is unlikely to be better than class 3.

Smaller samplers of about 50 mm or 75 mm diameter can be used if use of the 100 mm sampler is precluded by the borehole size. The smaller samplers are of similar design, except that the cutting edge may not be detachable.

19.4.5 Split-barrel standard penetration test sampler.

The split-barrel sampler is used in the standard penetration test and is described in test 19 of BS 1377 : 1975. It takes samples 35 mm in diameter and has an area ratio of about 100 %. It is used to recover small samples, particularly under conditions which prevent the use of the general purpose 100 mm sampler, and gives class 3 or class 4

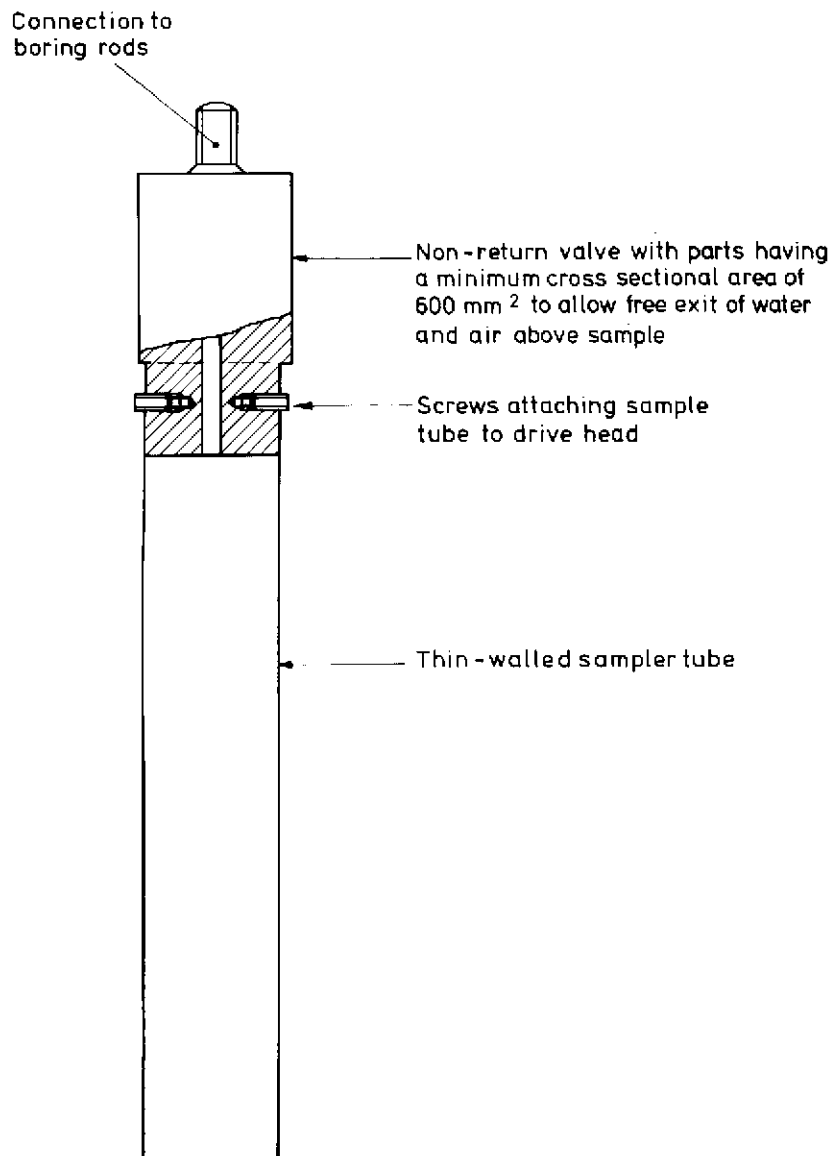


Figure 2. A typical thin-walled sampler

19.5 Thin-walled stationary piston sampler. The thin-walled stationary piston sampler consists of a thin-walled sample tube containing a close-fitting sliding piston, which is slightly coned at its lower face. The sample tube is fitted to the drive head, which is connected to hollow drill rods. The piston is fixed to separate rods which pass through a sliding joint in the drive head and up inside the hollow rods. Clamping devices, operated at ground surface, enable the piston and sample tube to be locked together or the piston to be held stationary while the sample tube is driven down. Figure 3 shows the basic details of a stationary piston sampler. The sample diameter is normally 75 mm or 100 mm, but samplers up to 250 mm diameter are used for special soil conditions.

Initially, the piston is locked to the lower end of the sample tube to prevent water or slurry from entering the sampler. In soft clay, with the piston in this position, the sampler can be pushed below the bottom of the borehole. When the sample depth is reached, the piston is held stationary and the sample tube is driven down by a static thrust until the drive head encounters the upper face of

the piston from dropping down and extruding the sample while the sampler is withdrawn.

The sampler is normally used in low strength fine soils and gives class 1 samples in silt and clay, including sensitive clay. Its ability to take samples below the disturbed zone and to hold them during recovery gives an advantage over the thin-walled sampler described in 19.4.3. Although normally used in soft clays, special piston samplers have been designed for use in stiff clays [15].

19.6 Continuous soil sampling

19.6.1 General. Continuous soil sampling can produce samples up to 30 m in length in soils such as recent fine alluvial deposits. This is of particular value for identifying the soil fabric [15] and gives results superior to those which can be obtained by consecutive fine sampling. The Swedish system [17] takes samples 68 mm in diameter using steel foils to eliminate inside friction between the sample and the tube wall. The Delft system, which uses lighter equipment and offers two sizes of

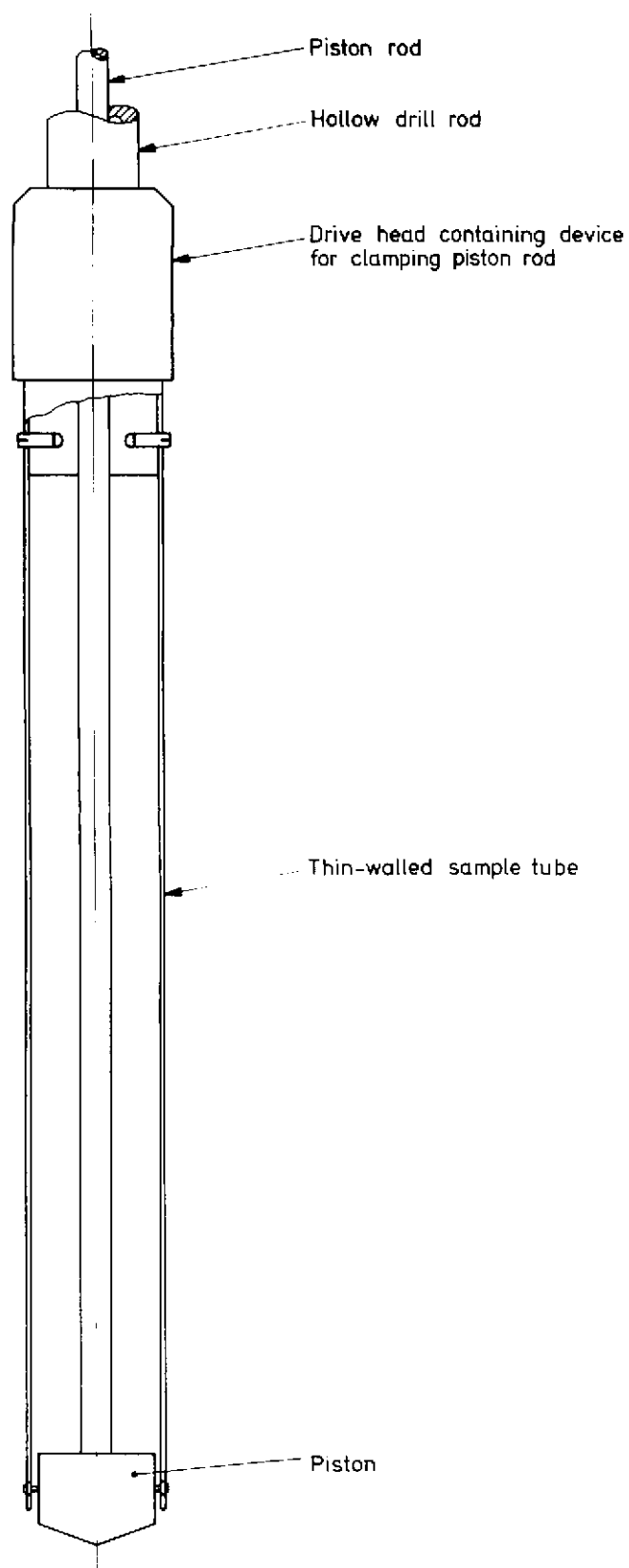


Figure 3. Basic details of piston sampler

19.6.2 The Delft continuous sampler. The Delft continuous sampler, developed by the Laboratorium voor Grondmechanica of Delft, Holland, is available in two sizes to take continuous samples 29 mm and 66 mm in diameter. The 66 mm sampler shown in figure 4 is pushed into the ground with a conventional Dutch deep sounding machine having a thrust of 200 kN. The sampler is advanced by pushing on the steel outer tubes, and the sample is fed automatically into a nylon stockinette sleeve which has been treated to make it impervious. The sample within the sleeve is fed into a thin-walled plastics inner tube filled to the appropriate level with a bentonite: barytes supporting fluid of similar density to the surrounding ground. The upper end of the nylon sleeve is fixed to the top cap of the sample, which is connected through a tension cable to a fixed point at ground surface. Extension tubes 1 m in length are added as the sampler is pushed into the ground. The sampler normally has a maximum penetration of 18 m, but in suitable strata, with a modified magazine and increased thrust, samples up to 30 m in length can be obtained. The 29 mm sampler is of similar design and requires less thrust to effect penetration.

The samples are cut into 1 m lengths and placed in purpose-made cases, samples taken with a 66 mm sampler being retained in the plastics tubes. The 66 mm samples are suitable for a range of laboratory tests. The 29 mm samples are used for visual examination and the determination of bulk density and index properties. After specimens have been removed for testing, the samples are split and are then described and photographed in a semi-dried state when the soil fabric is more readily identified. For 29 mm samples, only one half of the split material is used for testing, thus preserving a continuous record of the ground.

19.7 Sand samplers. The recovery of tube samples of sand from below the water table presents special problems because the sample tends to fall out of the sample tube. A compressed air sampler [19] enables the sample to be removed from the ground into an air chamber and then lifted to the surface without contact with the water in the borehole. The sampler is generally constructed to take samples 60 mm in diameter. If the sampler is driven by dynamic means, the change in volume of the sand caused by the driving gives a sample quality not better than class 3. However, if static thrust is used, generally class 2 and sometimes class 1 samples can be recovered. An alternative design [16] introduces a bubble of air at the base of the sampler before it is withdrawn from the ground.

19.8 Rotary core samples Samples are obtained by the core drilling procedures described in 18.7. The quality of sample may vary considerably depending on the character of the ground and the type of coring equipment used, see BS 4019 : Part 1.

Core barrel sizes and core sizes are listed in paragraph 2.2 of figure 29. In most rocks, satisfactory core will be obtained by using double-tube swivel type core barrels with core size not less than about 70 mm diameter (H size or equivalent). A reduction in core size to 55 mm diameter (NX size or equivalent) may be appropriate in massive rocks; and an increase in core size to between 100 mm and 150 mm diameter will usually give better results in weak, weathered or fractured rock. In weak, weathered or fractured rock, better core recovery can be obtained using triple-tube core barrels in which the inner tube

contains a removable sample tube or liner. The liner can be made of a variety of materials. Forms of liner include split tubes, seamless metal tubes, and plastics liners, both rigid and flexible. The satisfactory handling of core is also facilitated when triple-tube core barrels are used, see 19.10.5.

A further improvement can be obtained by using a retractor-type triple-tube core barrel, particularly where very weak material is interlayered with less weak material [20]. The retractor-type core barrel has a stationary inner tube, fitted with a cutting shoe, which is spring loaded so that the inner tube can advance beyond the cutting bit to a greater or lesser extent depending on the penetration resistance of the material being drilled. This greatly reduces the possibility of the drilling fluid coming into contact with the core at or just above the point of cutting. There is a variety of liners that can be used with the retractor-type triple-tube barrel. The retractor-type barrel is not effective where strong rocks are present.

A very wide range of coring bits is available, and the type which gives the best results for any given ground conditions may have to be determined by trial. It should be recognized that the factors affecting optimum bit selection may be different for rate of drilling and quality of core. Thin-walled bits produce fewer cuttings, which permits the use of a lower flushing rate, with diminished disturbance. On the other hand, thin-walled bits generally do not provide for face discharge of flushing fluid, and some disturbance may occur as the fluid passes between the inside of the bit and the core. This is obviated if retractor-type barrels are used. Stepped bits offer advantages, in some circumstances, in respect of drilling rates and quality, and also in that they produce straighter holes. For strong abrasive rocks, diamond quality and matrix selection are important considerations.

19.9 Block samples. Block samples are cut by hand from material exposed in excavations and are normally taken in rock and cohesive soil. The procedure is often used for obtaining specially orientated samples, and, in such cases, both the location and the orientation should be recorded before the sample is separated from the ground. The cutting of a block sample often takes an appreciable time during which there may be a tendency for the moisture content to change. The following precautions should be taken:

- (a) No extraneous water should be allowed to come into contact with the sample.
- (b) The sample should be protected from the wind and the direct rays of the sun.
- (c) Immediately after the sample has been cut, the orientation should be marked and then it should be coated with paraffin wax and packed as described in 19.10.6.

19.10 Handling and labelling of samples

19.10.1 General. Samples may have cost a considerable sum of money to obtain and should be treated with great care. The usefulness of the results of the laboratory tests depends on the quality of the samples at the time they are tested. It is therefore important to establish a satisfactory procedure for handling and labelling the samples, and also for their storage and transport so that they do not deteriorate, and can readily be identified and drawn from the sample store when required.

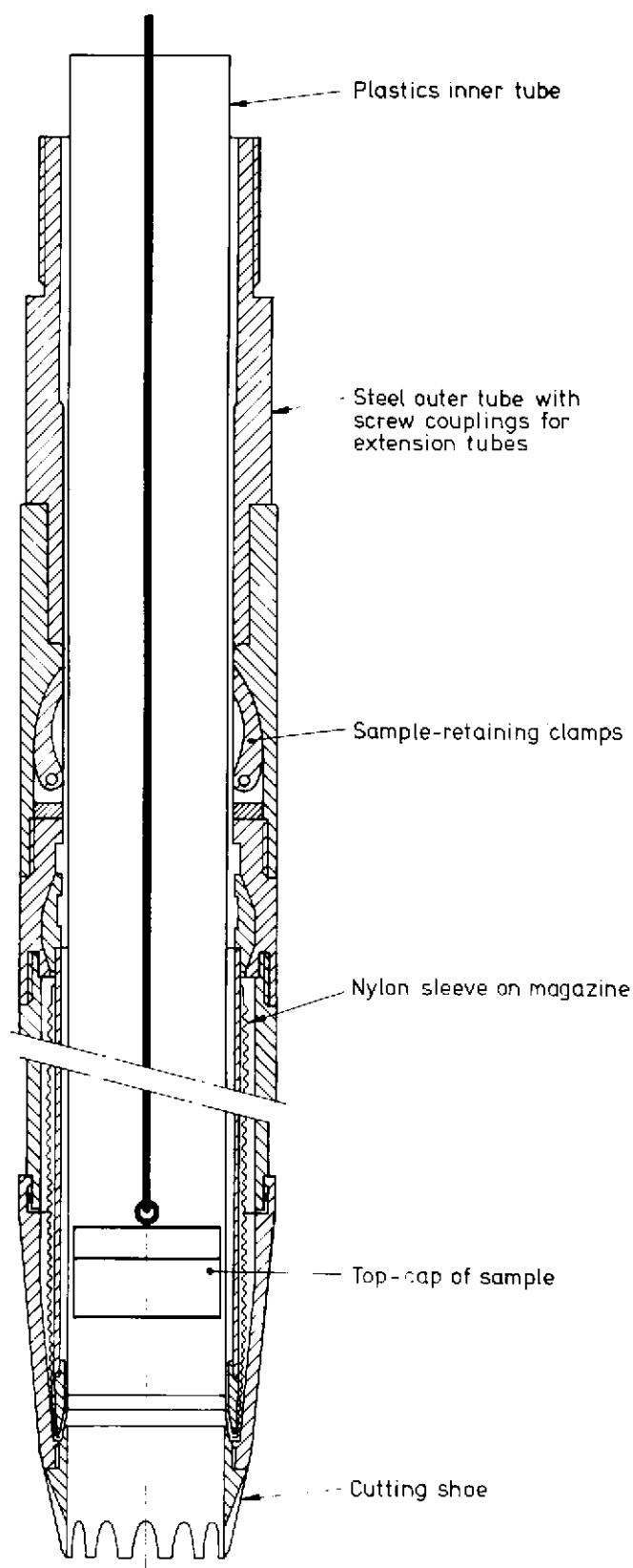


Figure 4. 66 mm Delft continuous sampler

The samples should be protected from frost, which will damage them, and from excessive heat and temperature variation, which may lead to deterioration in the sealing of the sample containers and subsequent damage to the samples. The temperature of the sample store will be influenced by the climate, but it is recommended that the samples be stored at the lowest temperature practicable within the range 2 °C to 45 °C. The daily temperature variation within the store should not exceed 20 °C.

19.10.2 Labelling. All samples should be labelled immediately after being taken from a borehole or excavation. If they are to be preserved at their natural moisture content, they will at the same time be sealed in an air-tight container or coated in wax. The label should show all necessary information about the sample, and an additional copy should be kept separately from the sample; this latter is normally recorded on the daily field report. The label should be marked with indelible ink and be sufficiently robust to withstand the effect of its environment and of the transport of the sample. The sample itself should carry more than one label or other means of identification so that the sample can still be identified if one label is damaged.

A typical sample label that is suitable for non-orientated

samples is shown in figure 5. The sheets are serially numbered and bound in book form in duplicate. Each sheet carries a portion which may be detached along a line of perforations and which is used as a label. A further portion, which can also be detached, carries the serial number of the label and can be fixed as a separate means of identification.

19.10.3 Disturbed samples of soil and hand specimens of rock. Where samples are required for testing, or where it is desirable to keep them in good condition over long periods, they should be treated as follows.

Immediately after being taken from a borehole or excavation, the sample should be placed in a non-corrodible and durable container of at least 0.5 kg capacity, which the sample should fill with the minimum of air space. The container should have an air-tight cover or seal so that the natural moisture content of the sample can be maintained until tested in the laboratory. For rock samples, an alternative procedure is to coat the sample in a layer of paraffin wax. A microcrystalline wax is preferred because it is less likely to shrink or crack. Larger disturbed samples that are required for certain laboratory tests may be packed in robust containers or plastics sacks.

Bound at
this edge

Perforated
here Tear
off Slip

No. 1234		
SAMPLE RECORD		
Location.....	Date	No. 1234
Borehole	O.D. of ground surface	
Position of sample, from		
to	below ground surface.	No. 1234
Container No.	Type of sample	
REMARKS		
Signed		No. 1234

Figure 5. Typical sample label

The sample containers should be numbered, and the tear-off slip or a label, as described in 19.10.2, should be placed in the container immediately under the cover. An identical label should also be securely fixed to the outside of the container under a waterproof seal (wax or plastics). The containers should be carefully crated to prevent damage during transit. During the interval while the samples are on site or in transit to the sample store, they should be protected from frost and from excessive heat.

For hand samples of rock, the reference number should be recorded by painting directly on the surface of the sample or attaching a label. Samples should then be wrapped in several thicknesses of paper and packed in a wooden box. It is advisable to include in the wrapping a label of the type described in 19.10.2.

19.10.4 Samples taken with a tube sampler. The following recommendations are applicable to all samples taken with tube samplers, except those taken with thick-wall samplers (see 19.4.5). The precautions for handling and protection of samples are to be regarded as a minimum requirement for samples taken by the usual methods. In special cases, it may be necessary to take more elaborate precautions. For samples that are retained in a tube or liner, procedure (a) should be followed; for other samples, procedure (b) should be followed.

(a) Immediately after the sample has been taken from the boring or excavation, the ends of the sample should be removed to a depth of about 25 mm and any obviously disturbed soil in the top of the sampler should also be removed. Several layers of molten wax, preferably microcrystalline wax, should then be applied to each end to give a plug about 25 mm in thickness. The molten wax should be as cool as possible. It is essential that the sides of the tube be clean and free from adhering soil. If the sample is very porous, a layer of waxed paper should first be placed over the end of the sample.

Any remaining space between the end of the tube or liner and the wax should be tightly packed with a material that is less compressible than the sample and not capable of extracting water from it, and a close-fitting lid or screw-cap placed on each end of the tube or liner. The lids should, if necessary, be held in position with adhesive tape.

(b) Samples that are not retained in a tube should be wholly covered with several layers of molten paraffin wax, preferably microcrystalline wax, immediately after being removed from the sampling tool, and then should be tightly packed with a suitable material into a metal or plastics container. The lid of the container should be held in position with adhesive tape. If the sample is very porous, it may be necessary to cover it with waxed paper before applying the molten wax.

A label bearing the number of the sample, preferably of the type shown in figure 5, should be placed inside the container just under the lid. The label should be placed at the top of the sample. In addition, the number of the sample should be painted on the outside of the container, and the top or bottom of the sample should be indicated. The liners or containers should be packed in a way that will minimize damage by vibration and shock during transit. For certain materials, testing in a site laboratory would be preferable.

19.10.5 Rotary core extrusion and preservation.

After recovery of the core barrel to the surface, every effort should be made in handling of the recovered core thereafter to ensure that the quality of the core is maintained in as near as is possible to its natural state until it is finally stored.

Except in relatively strong and massive rocks, core is almost inevitably disturbed if it is removed from the barrel held in a vertical position and then placed into the core box. The barrel should be held in a horizontal position, and the core extruded into a tray in such a manner that it is continuously supported. Rain-water guttering or other conveniently available rigid split tube can be used for this purpose. When it is required to preserve the core such that it does not dry out, a convenient method is to extrude it from the core barrel into sleeving formed of thin-gauge polyethylene, again supporting the core with rigid split tube. Where selected lengths of core are to be preserved at their natural moisture content for laboratory testing, any drilling mud contamination and softened material should first be removed, and the sample should then be wrapped in foil and coated with successive layers of waxed cheese cloth and labelled as described in 19.10.2.

In the extrusion process, the core should preferably be extruded in the same direction as it entered the barrel. Extruders should be of the piston type, preferably mechanically activated, since water-pressure type extruders can lead to water contact with the core, and to damage by impulsive stressing of the core. It should be noted that in weak, weathered or fractured rocks, extrusion can lead to core disturbance, however carefully it is done. The use of a low-friction transparent plastics liner in the inner tube of a modified conventional double-tube swivel core barrel overcomes the majority of the problems encountered in core extrusion, and facilitates preservation of the core in the condition in which it is recovered. The general practice is to tape the outside of the sleeved core every 200 mm, and lengthwise along the overlap in the plastics sheet, and then, with the aid of plastics guttering for extra support, the core can be boxed without too much disturbance to the fabric. However, the presence of abrasive and fractured rocks may preclude the use of such liners.

The difficulties of extrusion and preservation can be overcome by the use of triple-tube core barrels with low-friction liners, see 19.8. Split liner tubes are an ideal method of examining the recovered core without further damage after the drilling process. On the other hand, seamless metal liners and plastics liners are particularly useful where core is to be removed from site for logging or where confined undisturbed samples are required for sample preservation and subsequent laboratory testing.

It is usual to preserve all core obtained from the borehole for the period of the main work's contract to which the core drilling relates. This is conveniently achieved with wooden or plastics core boxes, usually between 1 m and 1.5 m in length and divided longitudinally to hold a number of rows of core. The box should be of such depth and the compartments of such width that there can be minimal movement of the cores when the box is closed [21]. The box should be fitted with a hinge, latched with a fastener, and should be so designed as not to be too heavy for two men to lift when the box is full of core.

In removing the core from the barrel and placing it in the box, great care should be taken to ensure that the core is not turned end for end, but lies in its correct natural sequence. Depths below ground surface should be indicated in indelible marker on small spacers of core diameter size that are inserted in the core box between cores from successive runs. Where there is failure to recover core, or where specimens of recovered core are removed from the box for other purposes, this should be indicated by spacing blocks of appropriate length. Both the lid and the box should be marked to show the site location, borehole number, range of depth of the core within the box, in addition to the number of the box of the total sequence of boxes in the borehole. Core box marking should be such as to facilitate subsequent photography which, if required, should be carried out as soon as is practicable after recovery of the core, and before description, sampling and testing.

19.10.6 Block samples. After labels have been attached to the sample to indicate its location and orientation, the sample should be coated with a succession of layers of microcrystalline wax, and it may be advisable to reinforce these with layers of porous fabric, e.g. muslin. Additional labels should be fixed to the outside of samples. The sample should be packed in a suitable material and placed in a strong crate. Large samples should be protected with tight fitting formwork or packed in rigid cement or resin to prevent fissures from opening up under the weight of the samples.

20. Ground water

20.1 General. The determination of ground water pressures is of the utmost importance since they have a profound influence on the behaviour of the ground during and after the construction of engineering works. There is always the possibility that various strata, particularly those separated by relatively impermeable layers, will have different ground water pressures, some of which may be artesian. The location of highly permeable water-bearing strata and the measurement of water pressure in each is particularly important where deep excavation or tunnelling is required, since special measures may be necessary to deal with the ground water. To measure ground water pressures accurately, it is generally necessary to install special measuring devices called piezometers. The ground water pressure may vary with time owing to seasonal, tidal, or other causes, and it may be necessary to take measurements over an extended period of time in order that such variations may be investigated. When designing drainage works, it is normally desirable to determine the contours of the water table or piezometric surface to ascertain the direction of the natural drainage, the seasonal variation and the hydrological controls.

Borehole permeability tests are described in 21.4; packer, or Lugeon, tests are described in 21.5; and large-scale pumping tests are described in clause 25.

20.2 Methods of determining ground water pressures

20.2.1 Response time. All the methods described in 20.2 require some flow of water into or out of the measuring device before the recorded pressure can reach equilibrium

with the actual ground water pressure. For an excavation or a borehole, a large volume of water may flow before the water level reaches equilibrium with the ground water pressure. On the other hand, some types of piezometer require only a very small change in volume of water in order that the ground water pressure may be read. The rate at which water flows through the soil depends on the permeability. The time required for a measuring device to indicate the true ground water pressure is known as the response time and depends on the quantity of water required to operate the device, and on the permeability of the ground. The selection of a suitable method for measuring the ground water pressure will largely be determined by the response time.

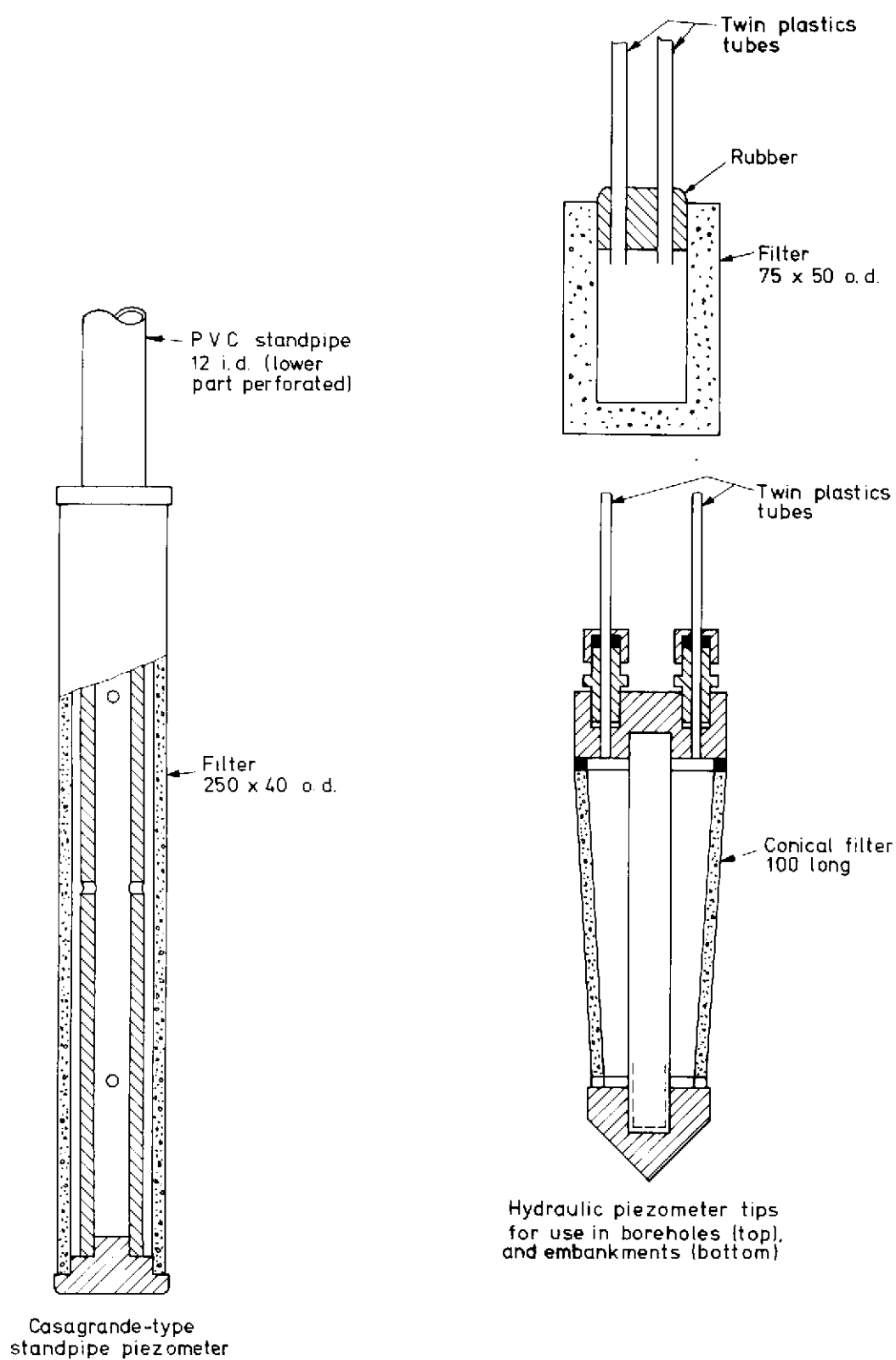
20.2.2 Observations in boreholes and excavation.

The simplest method of determining the ground water level is by observation in a borehole or excavation that is open or fitted with a perforated tube. This method may involve a considerable response time unless the ground is reasonably permeable, and observations should be made at regular intervals of time until it is established that the water level in the borehole or excavation has reached equilibrium. Often, the level of water measured in a borehole during or shortly after the completion of drilling is used to ascertain the ground water pressure. Such methods should be treated with great caution, since it is seldom that sufficient time is allowed for the water level to stabilize. Moreover, the levels from which the water is entering the borehole may not always be known, since these are affected by the use of temporary borehole casings. For most conditions, it is preferable to use a standpipe piezometer.

20.2.3 Standpipe piezometers. The standpipe piezometer is a device consisting of a tube or pipe with a porous element on the end, or with a perforated end section surrounded by or wrapped with a filter, which is sealed into the ground at the appropriate level. It is normally installed in a borehole. The tube should be of at least 12 mm diameter to allow air bubbles to rise freely. Access to the top of the standpipe is generally required for plumbing the water level, although this can be determined remotely using an air-bubbling system. In any case, standpipes need an air vent to allow the water level to come to equilibrium. A frequently used standpipe piezometer is the Casagrande-type [22] shown in figure 6. A small amount of anti-freeze added to the water in the piezometer tubing will prevent freezing in cold weather.

The main advantage of a standpipe is its simplicity. Moreover, it can also be used to determine the permeability of the ground in which the tip is embedded, see 21.4. The main disadvantage of the standpipe is the length of time taken to reach equilibrium or to respond to changes in pre-water pressure in soils of relatively low permeability.

20.2.4 Hydraulic piezometers. In hydraulic piezometers, the ground water pressure is detected in a small piezometer tip with porous walls and conducted through small-bore plastics tubes to a remote point where the pressure is measured usually with a mercury manometer or Bourdon gauge. Air in the tubes will cause erroneous readings, and because of this the tubes have to be kept full of water.



All dimensions in millimetres.

Figure 6. Typical piezometers

Various types of hydraulic piezometer are available, the most common being the twin-tube piezometer [22] shown in figure 6. In this, the piezometer tip is connected to the measuring point by two tubes, so that water can be circulated to flush out any air bubbles. This should be done in such a way that the pressure in the piezometer tip is left substantially at working pressure. For trouble-free operation, pressures throughout the connecting tubes should be above atmospheric pressure. Therefore, the tubes and measuring point should generally be below the equivalent water level being measured. The hydraulic piezometer is normally installed directly in trenches, but it can also be installed in boreholes where the piezometric pressure is sufficiently large.

An hydraulic piezometer has a small response time and can be used for measuring pressure changes due to tidal variation or to changes of stress induced by superimposed loads or excavations. It can also be used for in situ measurements of permeability. In strata with high permeability, care should be taken to see that the limiting permeability of the porous tip is considered.

20.2.5 Electrical piezometers. Electrical piezometers have a pressure transducer located close to the porous element. Very rapid response times can be achieved provided the tip is de-aired. Where long-term stability is required, or the signal is to be transmitted over a long distance, the transducer is usually of the vibrating wire type. The main disadvantage of the electrical piezometer is that it requires calibration, which cannot easily be checked after installation. It should be noted that some transducers have temperature-sensitive elements, so that check calibrations should be at ground water temperature. Moreover, it is not always easy to check that the instrument is behaving reliably. De-airing is not possible after installation, and misleading results can be obtained, particularly in soils containing gas, e.g. methane in organic soils. The electrical piezometer cannot be used for in situ permeability measurements [22].

20.2.6 Pneumatic piezometers. Pneumatic systems comprise two air-filled tubes connecting the measuring point to a valve located close to the porous element. When the air pressure in the input line equals the water pressure in the porous element, the valve operates, thereby holding constant the pressure either in the return line or in the supply line. The operation of the valve requires a small volume change in the porous element, and in stiff impermeable clays this can lead to difficulties. Also, dirt entering the lines can prevent valve operation. The pneumatic piezometer is cheap and easy to install and has a rapid response. It cannot be used for in situ permeability measurements [24]. Pneumatic piezometers have the same limitations as electrical piezometers in that they cannot be checked and the porous tips cannot be de-aired after installation.

20.2.7 Installation of piezometers. The success of pore-water pressure measurement depends upon the care taken during installation and sealing of the piezometer or standpipe. The porous element should be fully saturated and filled with de-aired water before installation. In soft ground, the porous element can often be pushed or driven into position. It is, however, necessary to avoid clogging the porous element if it is pushed through cohesive soils. This can be achieved by using a drive-piezometer which has a removable sleeve which covers the element during driving [25]. In clay, a pushed or driven piezometer shears

and remoulds the clay, destroys the permeable fabric in the clay adjacent to the porous element, and can lead to erroneous measurements of in situ permeability. It should also be noted that the action of pushing or driving may set up high excess pore-pressures, which in soils of low permeability may take a long time to dissipate. In harder ground, the instrument is installed in a borehole with the porous element surrounded by well-graded sand. Above the sand, the borehole should be sealed off, preferably with grout.

The most successful method of installation in a borehole is to lower the porous tip into position, about 200 mm above the bottom of the borehole, with a grout pipe loosely tied to the standpipe or leads just above the tip. A sand : water mixture is then pumped down the grout pipe until the piezometer tip is covered with sand to a depth of about 300 mm. The grout pipe is then pulled up by about 400 mm and the borehole is sealed off, preferably filled, by pumping down a suitable cement : bentonite grout. Care should be taken to avoid impregnating the sand with the grout. One way of avoiding this is to seal off immediately above the sand layer with soft bentonite balls pressed into a layer with a ring punner. This is followed by grouting. The installation of more than one piezometer in a single borehole is not recommended.

The composition of the grout mix will depend on a variety of factors, such as the availability of materials, the required permeability, the type and make of bentonite, the condition of the borehole and the ground water levels. The grout should be easily pumped, of the required permeability and flexible. The constituents should not segregate while the grout is still liquid. A typical mix might be 4 parts of bentonite mixed thoroughly with 8 parts to 12 parts of water to which is added one part of ordinary Portland cement. Special mixes and chemical additives may be necessary if the grout is to be used in sea water or very acid water.

20.2.8 Varying ground water pressures. Water pressures may show seasonal variation, may respond to tidal changes or may be affected by abstraction from neighbouring wells or by other causes. Where it is important to take account of these effects, adequate periods of observation should be adopted.

20.3 Ground water samples. Care should be taken to ensure that the samples are representative of the water-bearing stratum and that they have not been contaminated or diluted by surface water or water used for boring. Water samples should be taken as soon as possible after the water-bearing stratum has been met in the borehole. If other water-bearing strata occur at higher levels, these should be sealed off by the borehole casing. As far as is possible, all the water in the borehole should be removed by pumping or baling, and the sample taken from water which collects by seepage. About one litre should be collected in a clean glass or inert plastics bottle, rinsing the bottle three times with the water being sampled before filling. More stringent requirements may apply in certain cases, e.g. use of sterilized containers, see clause 13. Even when precautions are taken, water samples from boreholes may be unrepresentative. Better results can be obtained if samples can be taken from a standpipe piezometer sealed within the stratum. Water samples may deteriorate rapidly and should therefore be tested as soon as possible after sampling.

21. Tests in boreholes

21.1 General. 21.2 to 21.7 describe the various forms of test that are commonly conducted as supplementary to a ground investigation carried out by borings. Inevitably, there is some overlap with section five. For example, pumping tests are described in clause 25 because the borings are incidental to the test, rather than the reverse. Again, vertical loading tests may be carried out near the surface or at the bottom of a borehole; the main details of these tests are described in 29.1, and 21.5 covers only those aspects particular to boreholes. Clause 21 and section five are in a sense complementary to each other, and where a particular test is not described in one, it should be sought in the other.

21.2 Standard penetration test

21.2.1 General principles. The standard penetration test is a dynamic penetration test carried out using a standard procedure which is described in test 19 of BS 1377 : 1975. The disturbed sample of soil obtained is used for identification purposes. When the test is being performed in gravel or coarser soil or in rock, the cutting shoe of the split-barrel sampler may be replaced by a solid cone of the same outside diameter and an included angle of 60° . It is important that the test is carried out precisely as described in test 19 of BS 1377 : 1975, since even minor variations from the specified procedure can seriously affect the results [26]. The test is empirical and has been widely practised. There is much published information linking the results of the test with other soil parameters and with the performance of structures, see 21.2.4. The main purpose of the test is to obtain an indication of the relative density of sands and gravels, but it has also been used to obtain an indication of the consistency of other soils (silts and clays) and of weak rocks (e.g. chalk).

The ISSMFE (International Society of Soil Mechanics and Foundation Engineering) European Sub-committee on penetration testing has produced a revised draft specification for the test which differs in some minor details from BS 1377. No doubt, this specification will be taken into account when BS 1377 comes to be revised.

21.2.2 Preparation for carrying out test. It is necessary to clean out the bottom of the borehole. When the test is carried out below the ground water level, certain types of soil may be loosened below the base of the borehole by the action of the boring tools and by pressure differences between the ground water and water in the borehole. This effect can be particularly severe in sands. The effect can be reduced by keeping the borehole topped up with water and by very careful operation of the boring tools (the use of a small diameter shell may be advantageous), but often these expedients will not be completely successful.

21.2.3 Advantages and limitations. The great merit of the test, and the main reason for its widespread use is that it is simple and inexpensive. The soil strength parameters which can be inferred are very approximate, but give a useful guide in ground conditions where it may not be possible to obtain borehole samples of adequate quality, e.g. gravels, sands, silts, clay containing sand or gravel and weak rock. In conditions where the quality of the 'undisturbed' sample is suspect, e.g. very silty or very sandy clays, or hard clays, it is often advantageous to alternate the sampling with standard penetration tests, thereby obtaining a check on the strength. If the samples are found to be unacceptably disturbed, it may be

necessary to use a different method for measuring strength, e.g. the plate test described in 21.6 and 29.1.

When the test is carried out in granular soils below ground water level, the soil may become loosened, even when the test is carried out in strict accordance with BS 1377 and the borehole has been properly prepared. In certain circumstances, it can be useful to continue driving the sampler beyond the distance specified, adding further drilling rods as necessary. Although this is not a standard penetration test, and should not be regarded as such, it may, at least, give an indication whether the deposit is really as loose as the standard test may indicate. When there is good reason to believe that unrealistically low values are being recorded, consideration should be given to the use of some other test which can be performed independently of a borehole, e.g. the Dutch cone test described in 23.3.

In the construction of bored piles, the test is sometimes carried out in boreholes considerably larger in diameter than those used for ground investigation work. The result of the standard penetration test is dependent upon the diameter of the borehole, and these tests should not be regarded as standard penetration tests. They may, however, provide useful information to a piling contractor, particularly if he has considerable experience in their use.

21.2.4 Interpretation. Although interpretation is to a certain extent outside the scope of this code and belongs more properly to foundation design, an interpretation is often given in a site investigation report. Much has been written on this subject with regard to both conventional foundations and piles. [27 to 32].

21.3 Vane test

21.3.1 General principles. A cruciform vane on the end of a solid rod is forced into the soil below the bottom of the borehole and then rotated. The torque required to rotate the vane can be related to the shear strength of the soil. The method of carrying out the test is described in test 18 of BS 1377 : 1975. The test can be extended to measure the remoulded strength of the soil. This is done by removing the torque-measuring instrument from the extension rods and turning the vane through six complete rotations. A period of 5 min is permitted to elapse and the vane test is then repeated in the normal way. The degree of disturbance caused by rotating the vane differs from that obtained by remoulding a sample of clay in the laboratory, and the numerical value of the sensitivity of the clay determined by these procedures is not strictly comparable with the results obtained from laboratory triaxial tests.

The test is normally restricted to fairly uniform cohesive fully-saturated soils, and is used mainly for clay having an undrained shear strength up to about 100 kN/m^2 . The results are questionable in stronger clays, or if the soil tends to dilate on shearing or is fissured.

It is to be noted, however, that the undrained shear strength determined by an in situ vane test is, in general, not equal to the average value measured at failure in the field, e.g. in the failure of an embankment on soft clay. The discrepancy between field and vane shear strengths is larger the more plastic the clay [33].

21.3.2 Advantages and limitations. The main advantage is that the test itself causes little disturbance of the ground and is carried out below the bottom of the borehole in virtually undisturbed ground. This is particularly apparent

in sensitive clays, where the vane test tends to give higher shear strengths than those derived from laboratory tests on samples obtained with the general purpose sampler described in 19.4.4. In these conditions, the vane test results are generally considered to be more representative. If the test is carried out in soil that is not uniform and contains even only thin layers of laminations of sand or dense silt, the torque may be misleadingly high. The presence of rootlets in organic soils, and also of coarse particles, may lead to erroneous results.

Other types of vane are available that are used independently of boreholes. With the penetration vane test apparatus described in test 18 of BS 1377 : 1975, the vane and a protective casing are forced into the ground by jacking. At the required depth, the vane is advanced a short distance ahead of the protective casing, the test conducted, and the casing and vane subsequently advanced to the next required depth. With this type it is not always possible to penetrate to the desired stratum without the assistance of preboring. Small hand-operated vane test instruments are available for use in the sides or bottom of an excavation.

21.4 Permeability

21.4.1 General principles. The determination of in situ permeability by tests in boreholes involves the application of an hydraulic pressure in the borehole different from that in the ground, and the measurement of the flow due to this difference. The pressure in the borehole may be increased by introducing water into it, which is commonly called a falling head or inflow test, or it may be decreased by pumping water out of it in a rising head or outflow test. The pressure may be held constant during a test (a constant head test) or it may be allowed to equalize to its original value (a variable head test). The technique is strictly applicable only to the measurement of permeability of soils below ground water level, although an approximate assessment may be made above this level [34]. A great variety of tests are included under this heading, varying from the very crude, where simple problems can be solved by simple means, to the sophisticated when the nature of the problem demands more refined data.

21.4.2 Preparation for carrying out the test. In the simplest form of test, preparation consists in cleaning out the bottom of the borehole. The test is then conducted by measuring the rate of flow of water out of the borehole into the soil, or vice versa, through the open end of the casing. The borehole may be extended beyond the bottom of the casing, thus increasing the surface through which water can flow. If necessary, the uncased part of the borehole is supported by a suitable filter material. Water leaking through the casing joints has at time been found to cause misleading results and the problem has been overcome by the use of fibre rings.

For more accurate measurements, a perforated tube or a suitable piezometer tip is installed, which is then surrounded by a granular filter to prevent erosion of the ground, and the casing withdrawn. It is essential that the filter material used has a permeability much greater than that of the soil being tested. Recommendations for sealing the borehole above the granular filter are given in 20.2.7. To avoid errors in flow measurement due to compression and solution of trapped air in the leads, ceramic piezometer tips should be saturated with de-aired water before installation.

The period required for constant head tests is decreased and the interpretation simplified if short lengths of borehole are used for the test. Pore pressures should be in equilibrium before the test is performed, and with clays of low permeability it can take several months for the pore pressures set up by the drilling of the borehole to equalize.

Before a permeability test is conducted, it is essential to determine the level of the natural ground water table by one of the methods described in clause 20.

21.4.3 Variable head test. The first operation is either to fill the piezometer tube with water (falling head test) or to bale or pump out the water (rising head test). The head in the borehole is then allowed to equalize with that in the ground, the actual head being measured at intervals of time from the commencement of the test. The depth of the borehole should be checked to determine whether any sediment has come out of suspension or whether the bottom of the borehole has heaved during the test period.

21.4.4 Constant head test. A constant head test is normally conducted as an inflow test in which arrangements are made for water to flow into the ground under a sensibly constant head. It is essential to use clean water. It will not be possible to achieve a constant head if the ground water level is not constant or the head lost by friction in the pipes is significant. Where a high flow rate is anticipated and where the installation comprises a piezometer tip surrounded by a filter material, two standpipes should be installed, one to supply the water and the other to measure the head in the filter material surrounding the piezometer tip. The rate of flow of water is adjusted until a constant head is achieved and, in the simplest form of test, flow is allowed to continue until a steady rate of flow is achieved. In some ground, this may take a long period of time, and, in such cases the method suggested by Gibson [35] may be used, in which the actual rate of flow is measured and recorded at intervals from the commencement of the test.

21.4.5 Analysis of results. There are numerous published formulae for calculating permeability from these tests, many of them partly empirical. Those given by Hvorslev [36], which are reproduced in outline in 21.4.6, are much used and cover a large number of conditions. They are based on the assumption that the effect of soil compressibility is negligible. The method given in Gibson [35] for the constant head test is also indicated. This gives a more accurate result with compressible soils.

21.4.6 Formulae for borehole permeability tests

21.4.6.1 Method 1 (after Hvorslev [36])

(a) Constant head test

$$k = \frac{q}{FH_c} \quad (\text{Time lag analysis})$$

(b) Variable head test

$$k = \frac{A}{FT}$$

or

$$k = \frac{A}{F(t_2 - t_1)} \log_e \frac{H_1}{H_2} \quad (\text{General approach})$$

where

k is the permeability of soil;



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- q is the rate of flow;
 F is the intake factor (see figures 7 and 8);
 H_c is the constant head;
 H_1 is the variable head measured at time t_1 after commencement of test;
 H_2 is the variable head measured at time t_2 after commencement of test;
 A is the cross-sectional area of borehole casing or standpipe as appropriate;
 T is the basic time factor (see figure 9).

NOTE. The above formulae assume that the natural ground water level remains constant throughout the test. For the case where the natural ground water varies, see [36].

21.4.6.2 Method 2. Constant head test (after Gibson [35])

$$k = \frac{q_{\infty}}{FH_c}$$

$$C = \frac{q_{\infty}^2 r^2}{\pi n^2}$$

where

- k is the permeability of soil;
 C is the coefficient of consolidation or swelling;
 q_{∞} is the steady state of flow (read off the $q, 1/\sqrt{t}$ graph as illustrated in figure 10);
 F is the intake factor (see figures 7 and 8);
 H_c is the constant head;
 r is the radius of a sphere equal in surface area to that of the cylindrical tip
 n is the slope of the $q, 1/\sqrt{t}$ graph.

NOTE 1. Heads are referred to natural ground water level before the test.

NOTE 2. Method makes allowance for the compressibility of the soil and also permits the coefficient of consolidation or swelling to be calculated.

NOTE 3. The flow q has, in theory, a linear relationship with $1/\sqrt{t}$. In practice, it may take some hours for the plot to come on a straight line. The line can then be extrapolated to give q_{∞} and n , where the test would otherwise take too long.

21.4.7 Advantages and limitations. For most types of ground, field permeability tests yield more reliable data than those carried out in the laboratory, because a larger volume of material is tested, and because the ground is tested in situ, thereby avoiding the disturbance associated with sampling. The appropriate choice of drilling method and careful drilling technique are necessary to avoid disturbing the ground to be tested. In non-cohesive soils, the ground may be loosened below the bottom of the borehole; in laminated ground, a skin of remoulded mixed material may be formed on the walls of the borehole, thus blocking the more permeable laminations; in fissured rock, the fissures may be blocked by the drilling debris.

Constant head tests are likely to give more accurate results than variable head tests, but, on the other hand, variable head tests are simpler to perform. The water pressure used in the test should be less than that which will disrupt the ground by hydraulic fracturing. It has been shown that serious errors may be introduced if excessive water pressures are used [39]. In general, it is recommended that the total increase in water pressure should not exceed one half the effective overburden pressure. With soils of high permeability, greater than about 10^{-3} m/s, flow rates are likely to be large and head losses at entry or exit and

in the borehole may be high. In this case, field pumping tests, where the pressure distribution can be measured by piezometers on radial lines away from the borehole, will probably yield more accurate results; these are described in clause 27. When the test is carried out within a borehole using the drill casing, the lower limit of permeability that can be measured reliably is determined by the watertightness of the casing joints and by the success achieved in sealing the casing into the ground. In soil, the reliable lower limit is about 10^{-8} m/s. In lower permeability soils and in rock, it is advisable to carry out the test using a standpipe or piezometer which is sealed within the test length using grout. In ground of low permeability, the flow rate may be very small, and measurement may be subject to error owing to changes in temperature of the measuring apparatus.

The permeability of a compressible soil is influenced by the effective stress at which it is measured, and there may be significant differences between the results of inflow tests, in which effective stress is reduced, and the results of outflow tests, in which it is increased. The permeability of soil around the borehole may also be influenced by changes in its stress history owing to installation of the borehole and any previous permeability tests performed on it.

The compressibility is similarly influenced, and this may affect the results achieved. The accuracy with which the coefficient of permeability may be deduced from variable head tests decreases with the coefficient of consolidation of the soil being tested. In principle, the coefficient of consolidation or swelling may be deduced from the results of both constant and variable head tests. In practice, results of only limited accuracy can be obtained, owing to difficulties in interpretation and the extent to which the stress history of the soil adjacent to the borehole is modified by the installation of the borehole.

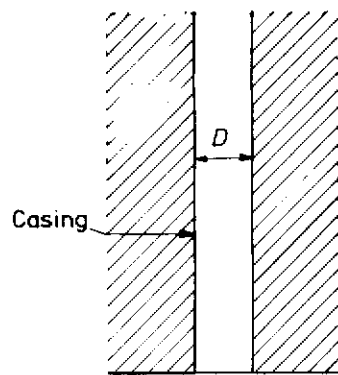
More reliable values of the coefficient of consolidation may be obtained either by direct measurement in the large Rowe consolidation cell [40], see soil deformation tests in table 4, or by use of the expression:

$$C_v = \frac{k_f}{m_v \gamma_w g}$$

where

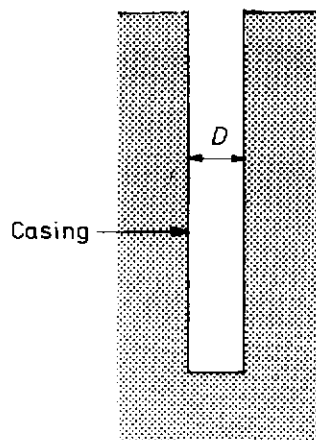
- C_v is the coefficient of consolidation;
 k_f is the field permeability;
 m_v is the coefficient of volume decrease as measured in the standard oedometer test;
 γ_w is the density of water;
 g is the acceleration due to gravity.

Execution of the borehole permeability test requires much expertise, and small faults in technique lead to errors of up to one hundred times the actual value. Even with considerable care, an individual test result is often accurate to one significant figure only. Often, an improved accuracy may be obtained by analysing the results of a series of tests. However, in many types of ground, particularly stratified soil or fissured rock, there may be a very wide variation in permeability, and the permeability of the mass of ground may be determined by a relatively thin stratum of high permeability or a major fissure. Very considerable care is needed in interpreting the test data; where a reliable result is required, the programme of borehole permeability tests is generally followed by a full-scale pumping test.



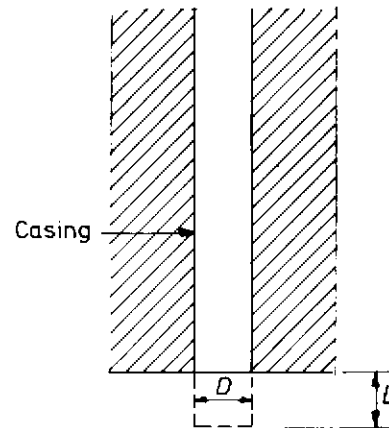
$$F = 2D$$

(a) Soil flush with bottom at impervious boundary



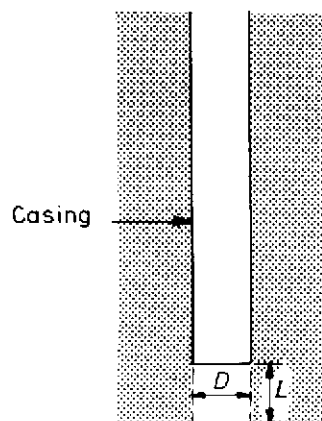
$$F = 2.75D$$

(b) Soil flush with bottom in uniform soil



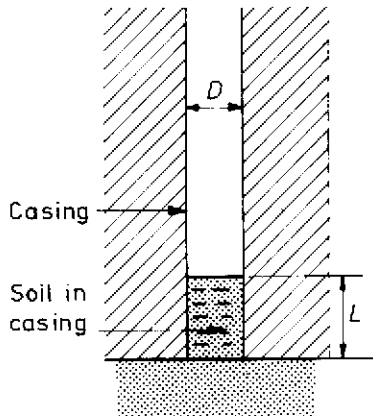
$$F = \frac{2\pi L}{\log_e [(2L/D) + \sqrt{1 + ((2L)^2/D^2)}]}$$

(c) Well point or hole extended at impervious boundary



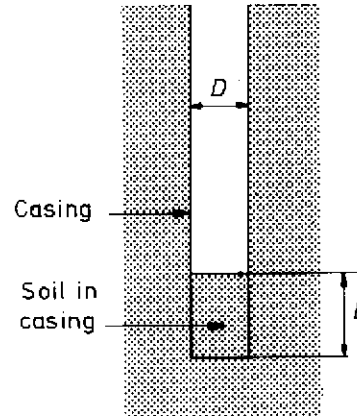
$$F = \frac{2\pi L}{\log_e [(L/D) + \sqrt{1 + (L/D)^2}]}$$

(d) Well point or hole extended in uniform soil



$$F = \frac{2D}{1 + (8/\pi) (L/D)}$$

(e) Soil in casing with bottom at impervious boundary



$$F = \frac{2.75D}{1 + (11/\pi) (L/D)}$$

(f) Soil in casing with bottom in uniform soil

NOTE 1. Expressions come from Hvorslev [36].

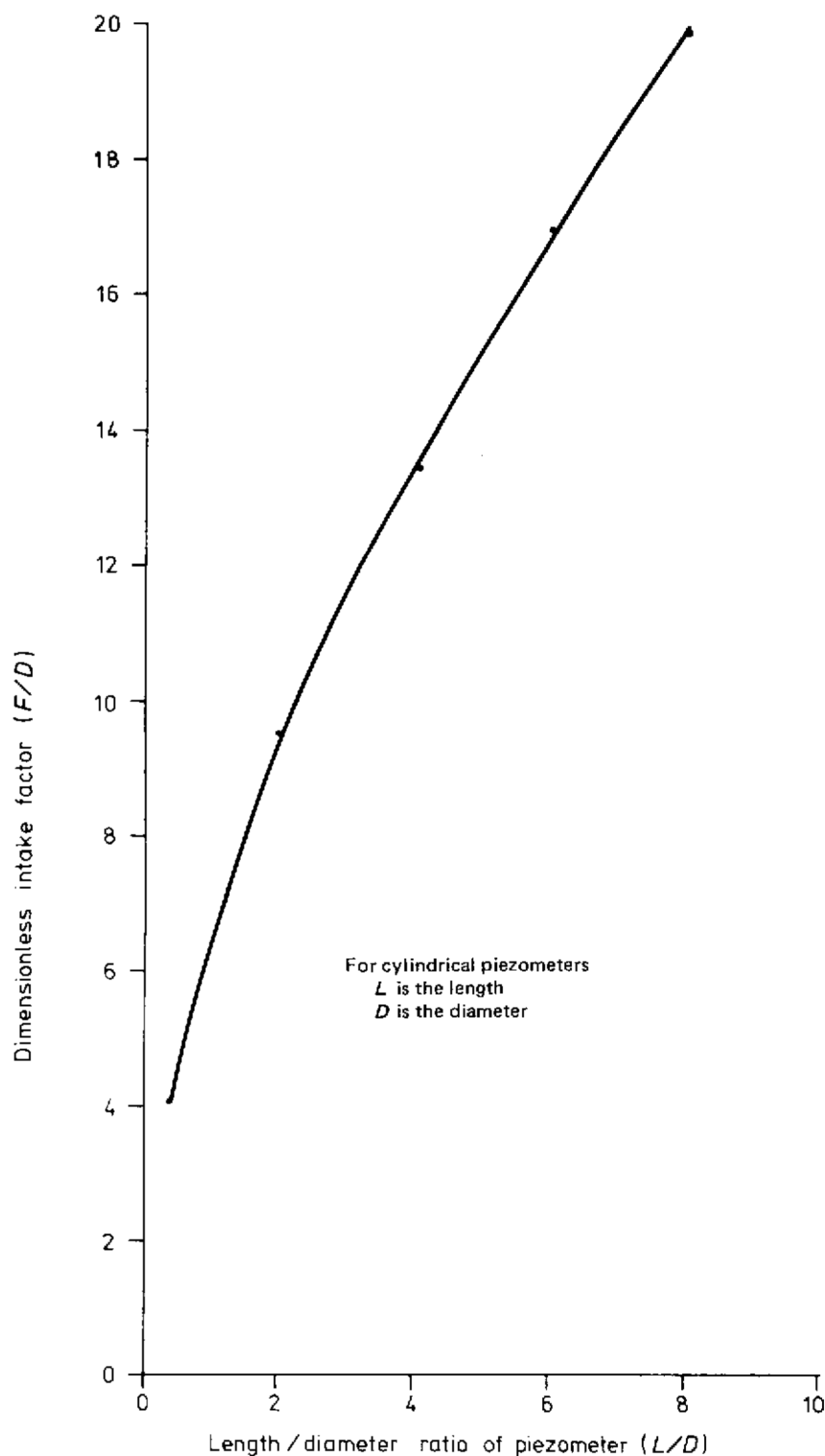
NOTE 2. Values are for use primarily for tests carried out through the open ends of bore holes.

NOTE 3. Case (d) may be used for tests carried out using piezometer tips, but more accurate results will be obtained by using figure 8, especially for values of $(L/D) > 2$.

NOTE 4. Cases (e) and (f) assume the permeability of the soil in the casing to be the same as that below it. Where this is not so, see [36].

NOTE 5. Cases (a) and (b) tend to measure the mean permeability of the soil; (c) and (d) the vertical permeability; (e) and (f) the horizontal permeability. Where the horizontal permeability is much greater than the vertical permeability, all tests will tend to measure the former.

Figure 7. Values of intake factors, F , in borehole permeability tests



NOTE 1. Graph comes from Al-Dahir and Morgenstern [37].

NOTE 2. Where a piezometer tip is surrounded by a granular filter material, it is the dimensions of this filter which should be used to derive values of F .

NOTE 3. Where L is large compared with D , the test will tend to measure the horizontal permeability of the soil.

NOTE 4. Where the horizontal permeability of the soil is much greater than the vertical, the test will measure the former, whatever the relation between L and D .

NOTE 5. The intake factor may also be calculated from the expression:

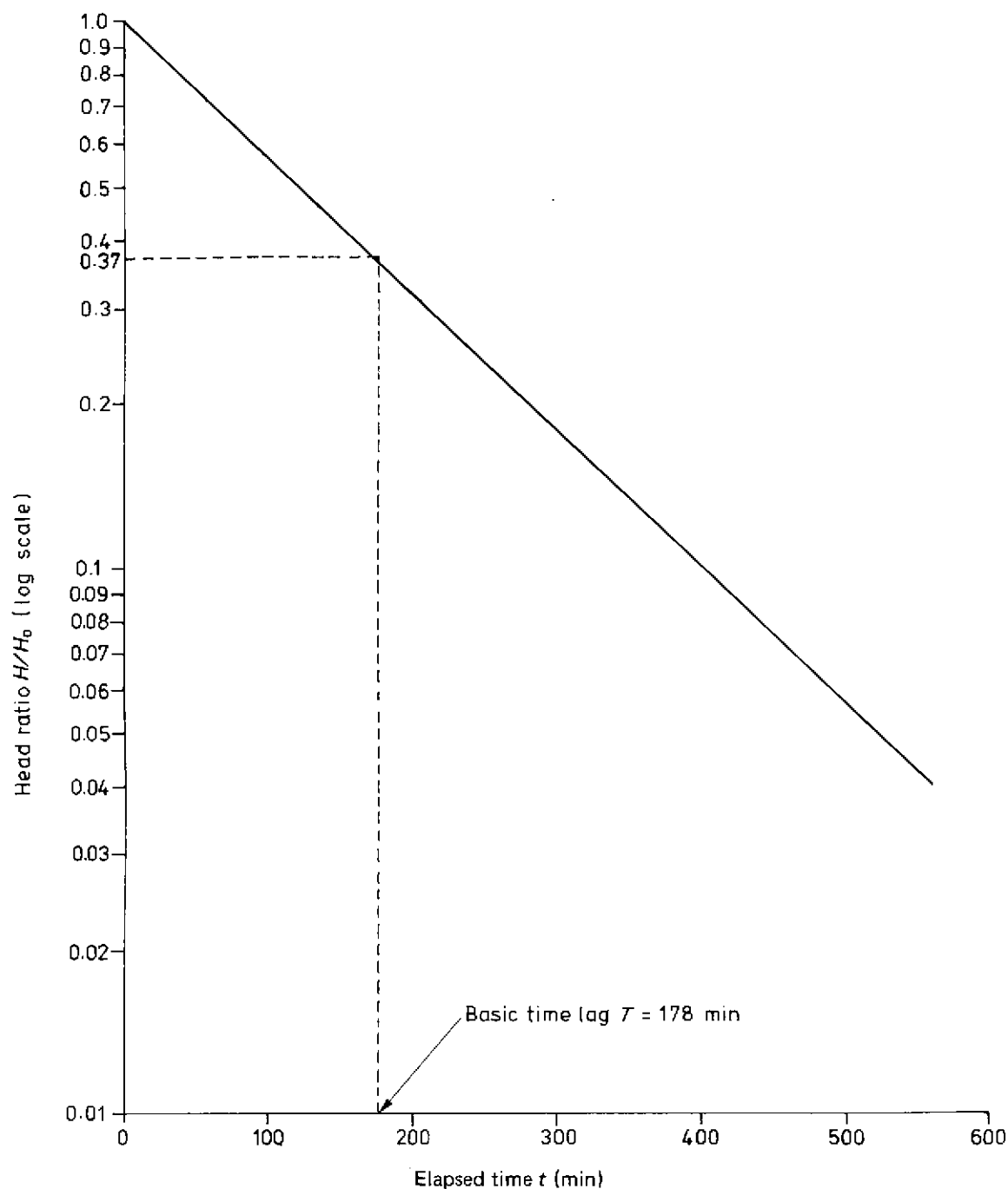
$$\frac{F}{D} = \frac{2.32 \pi (L/D)}{\log_e \{1.1 (L/D) + \sqrt{1 + 1.1 (L/D)^2}\}}$$

(see [38]).



Documents

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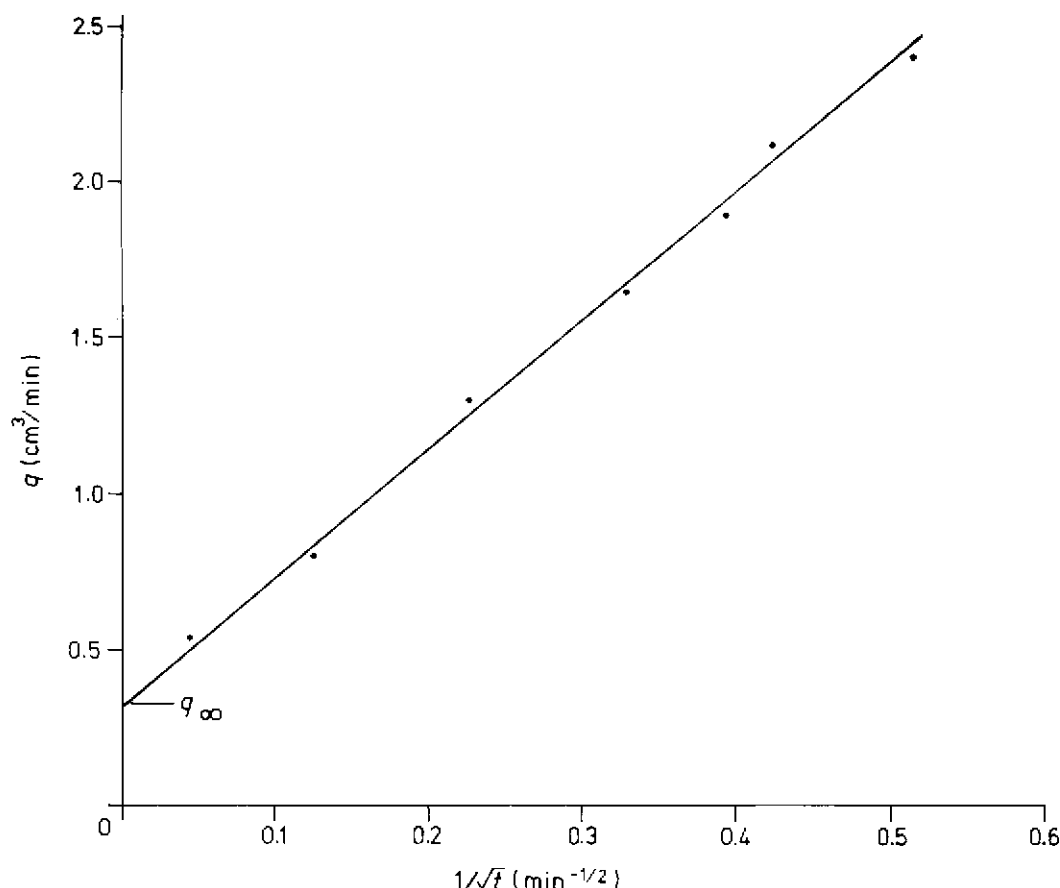
NOTE 1. H_0 is the head at commencement of test. H is the head at any time t which has elapsed since the test began.

NOTE 2. For each recorded value of t the ratio H/H_0 is computed.

NOTE 3. The ratio H/H_0 is plotted to a log scale against t to a natural scale.

NOTE 4. The basic time lag is taken to be the value of elapsed time t corresponding to a value of H/H_0 of 0.37.

Figure 9. Falling or rising head test. Example of calculation of basic time lag [36]



q is the rate of flow at any time t after commencement of test.
 q_{∞} is the steady state flow read off graph where $1/\sqrt{t} = 0$.

Figure 10. Constant head test analysis by Gibson's method [35]

21.5 Packer test

21.5.1 General principles. The packer or Lugeon test gives a measure of the acceptance by in situ rock of water under pressure. The test was originally introduced by Lugeon [41] to provide an acceptable standard for the permeability of dam foundations. In essence, it comprises the measurement of the volume of water that can escape from an uncased section of borehole in a given time under a given pressure. Flow is confined between known depths by means of packers, hence the more general name of the test. The flow is confined between two packers in the double packer test, or between one packer and the bottom of the borehole in the single packer test. The test is used to assess the amount of grout that rock will accept, to check the effectiveness of grouting, to obtain a measure of the amount of fracturing of rock [42] or to give an approximate value of the permeability of the rock mass local to the borehole.

The results of the test are usually expressed in terms of lugeon units. A rock is said to have a permeability of 1 lugeon if, under a head above ground water level of 100 m, a 1 m length of borehole accepts 1 litre of water per minute. Lugeon did not specify the diameter of the borehole, which is usually assumed to be 76 mm (N size), but the test is not very sensitive to change in borehole diameter unless the length of borehole under test is small.

A simple rule that is sometimes used to convert lugeon units into permeability is to take one lugeon unit as equal to a permeability of 10^{-7} m/s. An approximate value of permeability may also be calculated from the following formula, although the assumptions on which it is based are not always borne out in practice:

$$k = \frac{Q}{2\pi HL} \log_e \frac{L}{r}$$

where

k is the permeability;

Q is the rate of injection;

H is the pressure head of water in the test section;

L is the length of the test section;

r is the radius of the test section.

Tests to assess permeability by means of packer tests are usually carried out at varying values of Q and H , and the value of k determined from the slope of the flow versus pressure graph [43].

21.5.2 Packers. Several types of packer are in use, such as the mechanical tail pipe, the manual mechanical expanding packer and the hydraulic self-expanding packer, but by far the most commonly used is the pneumatic packer.

This comprises a rubber canvas duct tube which can be inflated against the sides of the borehole by means of pressurized gas. Bottled nitrogen or oxygen is fed down the borehole through a small diameter nylon tube. The inflation pressure has to be sufficient to expand the packer against the head of water in the borehole, but not sufficient to cause heaving of the ground surface or fracturing of the rock. A useful rule of thumb is that the pressure, in kN/m^2 , should lie between 12 times and 17 times the depth, in metres, of the borehole. The difference between the diameter of the uninflated packer and the diameter of the borehole should be such that the packer can be easily inserted. At the same time, the inflated diameter of the packer should be sufficient to provide an efficient seal. A double packer is two packers connected by a length of pipe of the same length as the test section. The test water is introduced between the packers.

21.5.3 Application and measurement of pressure.

The pressure is applied by a flush pump as used for diamond drilling. The maximum pressure which should be applied should not be sufficient to cause uplift of the ground or to break the seal of the packers in deep holes in weak rock. The standard head of 100 m above ground water level may not be attainable in these conditions.

The pressure to be determined for use in the calculation of permeability is that causing flow into the rock itself. This is sometimes measured directly, but it is more usual to measure it at ground level by means of a Bourdon gauge, with the readings adjusted in accordance with the following expression:

$$H_T = P + (H - H_g) - H_f$$

where

H_T is the pressure head causing flow into the rock (in m);

P is the Bourdon gauge reading converted to head (in m);

H is the height of Bourdon gauge above mid-point of test section (in m);

H_g is the height of natural ground water level above mid-point of test section (in m);

H_f is the friction head loss in the pipes (in m).

The pressure gauge should be so positioned that it will give a true reading without interference from local pressure variations induced by flow through the pipe work. The natural ground water level should be measured before the test begins. This is not always easy, especially when the rocks are of low permeability, and water has been used for flushing purposes during drilling. If necessary, separate observation wells should be installed, and the ground water levels measured over a period to establish the general ground water level. Friction head loss in the pipes is best established by means of a calibration test, with the pipe work laid out on the ground.

21.5.4 Measurement of flow. The rate of flow of water may be measured either by a flowmeter or by direct measurement of flow out of a tank of known dimensions by means of a dipstick or depth gauge. Where a flowmeter is used, it should be installed upstream of the pressure gauge, well away from bends or fittings in the pipework, and in accordance with the manufacturer's instructions. The accuracy of the meter should be checked before the test begins, and periodically afterwards, by measuring the

time taken to fill a container of known volume at different rates of flow. Where the flow out of a tank is to be measured, the use of one large tank can lead to inaccuracies where the plan area is large and the fall in level correspondingly small. A better arrangement is to use a number of small containers.

21.5.5 Execution of test. The test may be carried out either as a single or as a double packer test. The single packer test is generally to be preferred because any leakage past the packer can be detected, whereas leakage past the lower packer in the double packer test cannot. However, the single packer test has normally to be done periodically during the drilling of the hole, which makes it more costly. An important point is to ensure that the packer is properly seated in the borehole. Where a complete core has been recovered from the borehole, or where appropriate logging or television inspection has been carried out, a careful examination may reveal suitable places to seat the packer. Where the seating proves unsatisfactory, the length of the test section should be altered or test sections overlapped, so as to seat the packer at a different depth in the borehole.

Because of the limitation on the pressure referred to in 21.5.3, it may not be practicable to run the test at the specified head of 100 m above ground water level. The assumption is then made that the water flow is proportional to the pressure, although this is not necessarily true [44]. It is then possible to obtain the Lugeon value by extrapolation. It is customary to run a series of tests at different pressures. A series of five tests is desirable, with the maximum pressure applied in three equal increments and then reduced with decrements of the same amount. The data obtained from these measurements are particularly useful in assisting in the interpretation of the behaviour of the rock under test.

21.5.6 Recording of results and calculating permeability.

Two suggested field report forms are given in figure 25. The first records the field data, and the second enables the permeability to be computed; the method is clear from the forms.

21.6 Plate tests

21.6.1 General. The plate test is one particular application of the vertical loading test, and the general procedures for the test are described in 29.1. Only the specific problems which arise from carrying out the test in the bottom of a borehole are discussed in this clause. Wherever practicable, the test should be conducted in a borehole which is of sufficient diameter for a technician to enter, clean out the bottom, and bed the plate evenly on undisturbed ground. Careful attention should be directed towards safety for operators working below ground, see 18.2 and appendix F. Where, for reasons of economy, the test is conducted in a small diameter borehole, the cleaning of the bottom and the bedding of the plate has to be done from the surface, so that it is very difficult to be certain that the plate is not resting on disturbed material. This would, of course, limit the value of the results.

The techniques for tests in large and small diameter boreholes differ in some respects and, where differences occur, the methods used are separately described in section five. The diameter of the plate used should, so far as practicable, be equal to that of the borehole, provided that care is taken to eliminate cohesion or friction on the side of the plate. Except in strong materials, the plate

should have a skirt as shown in figure 18. Where the diameter of the plate is significantly less than that of the borehole, the results of the test become difficult to interpret. At a hole-diameter to plate-diameter ratio greater than about 3:2, the parameters being measured are those pertaining to a load at a free surface and not at depth under confined conditions, which are usually the conditions of interest.

21.6.2 Limitations. The general limitations of the vertical load test are discussed in 29.1.2 and they apply similarly to the borehole test. Additionally, in the bottom of a borehole it is more difficult to achieve a satisfactory bedding of the loading plate on the test surface, and hence values obtained for the deformation parameters may be of limited significance.

21.6.3 Preparation. Where necessary, casing should be used to support the sides of the borehole and to seal off water seepages from strata that are above the test elevation. When the test is to be carried out below the prevailing water table, dewatering by pumping or baling from within the borehole may cause seepages which disturb the ground and adversely affect its deformation characteristics. It would then be necessary to resort to external dewatering, see 29.1.2. If the test is undertaken only for measuring the strength parameters, disturbance due to ground water seepage may be a less significant factor and the borehole may be emptied, if this is possible, while the plate is being installed. The water should be allowed to return to its normal rest level before the test is commenced. Alternatively, the plate can be installed under water, although it may not then be possible to set the plate sufficiently accurately for the deformation characteristics to be measured.

21.6.4 Bedding of the plate

21.6.4.1 Large diameter boreholes. Subject to safety requirements, see BS 5573, a technician should be lowered to the bottom of the borehole to remove all loose soil manually, after which the plate is bedded as described in 29.1.3.

21.6.4.2 Small diameter boreholes. The cleaning is carried out by means of a suitable auger or hinged bucket operated at the end of a drill rod assembly. A layer of neat cement mortar is then placed at the bottom of the borehole by means of a tremie or bottom opening bucket, and the plate lowered down the hole and lightly pressed on to the surface of the mortar. Plaster and resins can also be used for bedding.

21.6.5 Application and measurement of load. The plate is usually loaded through a column formed by a steel tube with the load applied to the column by means of an hydraulic jack operating against the resistance of kentledge, tension piles or ground anchors, as described in 29.1.4.

21.6.6 Measurement of deflection. The movement of the plate under load is generally transmitted to dial gauges at the surface by means of a settlement measurement rod that is located within the steel tube by which the load is applied; and the rod is restrained from lateral movement by rod guides fixed within the tube. Methods of supporting the dial gauges are given in 29.1.4.

21.6.7 Execution of test. The method of carrying out the test is given in 29.1.6.

21.6.8 Uses of the test

21.6.8.1 Large diameter boreholes. The main use of the test is to determine the strength and deformation characteristics of the ground. It is sometimes used to establish the working load of piles.

21.6.8.2 Small diameter boreholes. The deformation characteristics obtained are of very dubious value owing to doubts about the elimination of ground disturbance and errors resulting from unsatisfactory bedding of the plate. The main use of the test is for measuring the strength characteristics of those cohesive soils in which undisturbed samples cannot be obtained, e.g. some gravely clays and weak rocks. The plate diameter should be large in relation to the structure of the ground.

21.6.9 Supplementary test. Although not strictly a plate test, a test is sometimes made to determine by in situ methods the coefficient of friction between the ground and concrete as an aid to assessing shaft friction for pile design. At the bottom of a borehole is placed either a layer of compressible material or a suitably designed collapsible container. The shaft above this level is then filled with concrete while the casing is withdrawn. When the concrete has sufficiently matured, the load is applied, and the deflection measured in a manner similar to that described in 21.6.6. Where the shaft friction of only part of the ground profile is required, as in a rock socket, the concrete is first brought up to the level of the top of the stratum concerned, and the shaft is continued in smaller diameter.

21.7 Pressuremeter tests. In a pressuremeter test, a probe is inserted into a pocket below the bottom of a borehole or directly into the appropriate size of drillhole and expanded laterally by compressed air or gas. The applied pressures and resulting deformations are measured and enable the strength and deformation characteristics of the ground to be investigated.

The earliest instrument, and that in most general use, is the Ménard pressuremeter [45]. In this, the lateral load is applied by a probe consisting of a water-filled central measuring cell flanked by two guard cells, either gas-filled or water-filled, depending on the type of instrument.

Readings are taken on pressure and volume gauges at the ground surface; these gauges connected to the central cell by means of a back-pressured annular plastics tube. The pressure tube and probe require to be calibrated on site. The function of the guard cells is to ensure a condition of plane strain in the ground in contact with the central cell. The probes are manufactured in four sizes up to 75 mm diameter, and can be operated at considerable depths. The Ménard pressuremeter can be used in soil or weak rock, but is not suitable for stronger rock, since the instrument is limited by its sensitivity to the tube calibration. It can be used in granular soils where special means are used to insert it.

A pressuremeter with a 150 mm diameter probe expanded by gas in which the deformation is measured directly by potentiometers located in the centre of the probe has been developed primarily for use in glacial till [46]. It can also be used to determine the deformation characteristics of the more deformable weathered rocks.

In stronger rocks, it is necessary to use instruments of high sensitivity in which the deformation of the rocks is measured over small strain ranges by electronic transducers located within the probe. There are two types of

instrument: a flexible type, 73 mm in diameter, operated hydraulically by oil to about 14 MN/m² [47]; and a rigid type, consisting of a steel cylinder split vertically into two halves and called the Goodman jack [48].

The rigid type is also operated hydraulically by oil, but with a considerably higher pressure than the flexible type, and is therefore particularly suitable for rocks in the higher modulus range.

It is not possible to insert borehole pressuremeters into soils and weathered rocks without causing disturbance to the walls of the test pocket, a circumstance which particularly affects the deformation characteristics of the ground. A self-boring pressuremeter, the Camkometer, which can be installed itself with negligible disturbance has recently been developed [49]. This instrument is expanded by gas, the expansion being measured with great accuracy by electrical gauges attached to feelers within the probe, enabling the undisturbed stress/strain characteristics of the ground to be determined under conditions of undrained radial expansion. From these results, the undrained shear strength, the shear modulus or undrained modulus of elasticity and in situ total horizontal stress are obtained. The instrument can also operate in sands where the conditions tend to be drained. A similar instrument has been developed in France by the Laboratoire des Ponts et Chaussées [50].

22. Frequency of sampling and testing in boreholes

22.1 General principles. The frequency of sampling and testing in a borehole depends on the information that is already available about the ground conditions and the technical objectives of the investigation. In general, the field work will cover three aspects, each of which may require a different sampling and testing programme and may also require phasing of operations. These aspects are as follows.

- (a) The determination of the character and structure of all the strata.
- (b) The determination of the properties of the various strata whose locations have been determined in (a), using techniques for sampling and testing that are normally available in routine ground investigations.
- (c) The use of special techniques of sampling and testing in strata where the normally available techniques have given, or may be expected to give, unsatisfactory results.

22.2 Determination of character and structure of the strata. In areas where suitable information about the ground conditions has been built up from the results of previous investigations, it may be possible to omit this aspect. Without this information, it is necessary to determine, so far as is practicably possible, the location, character and structure of each stratum. Some of the strata may be quite thin, and continuous sampling to the depth of the borehole may be required in order that the necessary information may be obtained.

In fine cohesive soil, and some silty sand, consecutive drive samples can be obtained using the 100 mm diameter sampler, or similar. In soft clay, and sand, the barrels of the sampler can be coupled together to form a sampler 1.0 m in length and, if necessary, the core catcher should be used to help retain the samples. Special sampling equipment is available for taking long continuous samples in

In coarse non-cohesive soil, such as gravel, it will be necessary to take disturbed samples from the drill tools, see 19.3, together with split barrel standard penetration test samples, see 19.4.5, at about 1 m intervals.

In rock, continuous rotary core sampling, see 19.8, should be undertaken and, in addition, disturbed samples should be taken from the return drill fluid to provide some information if the core recovery is poor. Where a run with the rotary core barrel results in poor core recovery, it may be useful to try to recover a small drive sample using the split barrel standard penetration test sampler.

Some of the samples obtained by drive sampling or by rotary coring that are not required for undisturbed tests may with advantage in some problems be split along their longitudinal axis and carefully examined and described in their fresh condition and then again later in a semi-dried state when the fabric is more readily identified. Where highly variable ground conditions are expected, it may be advantageous to first sink one or more boreholes by rotary core sampling or by cable tool boring with continuous tube sampling. The cores or tube samples can then be examined to give guidance for sampling at selected depths in boreholes subsequently sunk close to the initial boreholes.

22.3 The determination of stratal properties using equipment normally available for sampling and testing.

After the strata whose properties are likely to be relevant to the technical objectives of the investigation have been identified, the properties of the strata are assessed using techniques that are normally available or special techniques such as those given in 22.5. The programme of sampling and testing can be varied to suit the requirements of a particular investigation and the drilling equipment that is in use. The following is a programme suitable for general use where the borehole is being sunk through soil using a light cable percussion boring rig together with the general purpose 100 mm diameter sampler and the standard penetration test equipment; or through rock by rotary core drilling, producing cores of approximately 55 mm or 70 mm diameter.

- (a) *Sand and gravel.* At the top of each new stratum, and thereafter at 1 m intervals of depth, a standard penetration test should be carried out using the split barrel sampler if the soil is suitable.

At the top of each new stratum and thereafter at 1.5 m intervals of depth, a disturbed sample should be obtained from the drill tools.

- (b) *Cohesive soil.* At the top of each new stratum and thereafter at 1.5 m intervals of depth, a 100 mm sample should be obtained.

If with the 100 mm sampler there is inadequate recovery or the sampler cannot be driven, this should immediately be followed by a standard penetration test using a split barrel sampler.

At each metre of depth, a disturbed sample should be obtained from the drill tools.

- (c) *Rock.* Continuous rotary core sampling should be used.

In cases where the core recovery is poor and the rock is weak, the split barrel standard penetration test sampler should be used after each core run in an attempt to recover a small sample of the rock. Depending on the

rock type, it may also be useful to take a disturbed sample from the drill fluid return, and thereafter at 1 m intervals of depth.

Notwithstanding the procedures outlined above, the object in rotary core drilling is to obtain full core recovery (see 18.7).

22.4 Double-hole sampling. In this method, a borehole is sunk to ascertain the full sequence of strata. A second borehole is then sunk adjacent to the first, but at least 1 m from it to avoid the disturbed zone, and samples are taken at predetermined levels. The following variation of this method is useful where it is difficult to determine the change from superficial deposits or weathered rock to relatively unweathered rock, or where it is difficult to decide when to change-over from cable-tool boring to rotary drilling. The first borehole is sunk using cable-tool boring and driven tube sampling to the depth beyond which the method ceases to be practicable. The second borehole is then sunk using rotary coring from the highest practicable level. This may obviate loss of sample or loss of core arising from a changeover of method at an inappropriate level.

22.5 Special techniques of sampling and testing. Special techniques of sampling and testing include the following techniques.

Type of ground	Special techniques
sand	sand samplers
soft sensitive clay	borehole vane test; thin-walled open sampler; piston sampler; continuous soil sampler
hard stoney clay	plate bearing test; pressuremeter; rotary core sampling
rock	rotary core drilling using larger core sizes than recommended in 22.3

Some of the equipment is fragile, and easily damaged if used in unsuitable ground. It should preferably be used only where it is known that ground conditions are suitable.

Using a special sampling technique, the frequency of sampling in sand and in soft sensitive clay will in general be determined by similar considerations to those in 22.3. However, if the stratum requiring the use of a special sampling technique is of limited thickness, it may be advisable to take samples at smaller intervals of depth to obtain a sufficient quantity of sample material. Rotary core samples in hard stoney clay or rock will normally be taken continuously.

In the borehole vane test, only the small volume of clay that is rotated by the vane is tested, and individual results often show a considerable scatter. For this reason, vane tests should be carried out as frequently as possible. The vertical interval will be determined by the depth at which the test is carried out below the bottom of the borehole; this interval is usually 500 mm. Closer spacing can be obtained using the penetration vane apparatus.

Where a series of plate tests is required at increasing depths, the minimum spacing is determined by the depth to which the soil has been stressed by the test. A vertical interval of about four times the borehole diameter is usually adequate.

23. Probing and penetration testing

23.1 General. Probing from the surface probably represents the oldest method of investigating the depth to

unduly thick. The simplest probe is a sharpened steel rod which is pushed or driven into the soil until it meets resistance. The method is still of use where other means of site investigation have disclosed relatively thin layers of very soft soils overlying much harder ones, when the thickness of the soft stratum may be determined over a wide area very quickly and economically. The method has many limitations, and a variety of more sophisticated apparatus has been developed in an attempt to overcome these and to extend the use beyond that of detecting a hard stratum to give some measure of the allowable bearing capacity of the soils present. Two distinct types of probe have been developed: one where the probe is driven into the soil by means of some form of hammer blow; the other where the probe is forced into the soil by a static load.

23.2 Dynamic probing. The apparatus for dynamic probing comprises a sectional rod with a cone fitted at the end whose base is of slightly greater diameter than the rod. It is driven into the ground by a constant mass that is allowed to fall on to the rod through a constant distance, and the arrangement should be such that the mass falls through the constant distance without the operator having to use his judgement in any way. A commonly used device is the Macintosh probe. The fact that the rod is slightly smaller in diameter than the base of the cone prevents, to some extent, shaft friction influencing the results, but at depth in certain soils this factor has to be taken into account. Shaft friction is eliminated by boring a hole to the depth required for each test or by providing the probe with a sleeve around the rod. The size of the cone, mass of hammer and the distance through which the constant mass is allowed to fall have not been standardized, and hence there is no substantial body of practical experience behind the test, although some published data are available linking the results of certain tests with soil parameters determined by other methods.

The main use of dynamic probing is to interpolate data between boreholes. Where a site investigation has been carried out by more conventional means, it may be possible to use dynamic probing to check rapidly and cheaply that conditions on neighbouring sites are similar. As with other types of penetrometer, probing can give unreliable results in soils containing occasional cobbles or boulders, which may easily be mistaken for bedrock.

23.3 Static probing or cone penetration test

23.3.1 General. Several types of static probing equipment have been developed and are in use throughout the world [28]. The basic principles of all the systems are similar in that a rod is pushed into the ground and the resistance on the tip of the rod is measured by a mechanical, electrical or hydraulic system. The resistance on a section of the side of the rod and the total force applied to the rod may also be measured. The reaction required to affect penetration may be obtained from kentledge or screw pickets installed into the ground or a combination of both of these. Static probing is known by a number of descriptive terms including static sounding, cone penetration testing, Dutch deep sounding and Dutch cone testing, as well as by other terms including the names of the manufacturer or operators of specific types of equipment. There is no British Standard for static probing but the ISSMFE Sub-committee on standardization of penetration testing in Europe has recommended a standard for a cone penetration test. This code, besides

recommending a standard penetrometer, recognizes a number of traditional types of penetrometer. Only two types of penetrometer are described in this code, as they are the most widely used. It is imperative that the type of equipment used is recorded, as the interpretation of the results depends on the type of equipment used.

23.3.2 Mechanical Dutch cone penetrometer. The Dutch cone penetrometer with a mechanically-operated cone was developed mostly at Delft Soil Mechanics Laboratory in the 1930s, and has been widely used, particularly for pile design. There are two types of penetrometer cone: the mantle cone and the friction jacket cone, as shown in figure 11. With both types, the cone is pushed to the required depth in the closed position by exerting pressure on the outer sounding tube. With the mantle cone, the force on the cone is then measured, generally by an hydraulic load cell at ground surface, while the cone is pushed downward by means of the inner pressure rod independently of the outer sounding tube. With the friction jacket cone, the same initial measurement is made and then the retaining sleeve on the cone engages with the friction jacket, and a second measurement is made of the force on the cone and the friction jacket. The friction is calculated by deducting the former reading from the latter. This procedure is repeated at regular intervals of depth, normally every 0.2 m or 0.25 m. An alternative quick continuous method of penetration may be used when a mantle cone is employed. In this, the sounding tubes are pushed into the ground with the cone permanently extended and connected to the load cell, taken at required depths. When using the alternative technique, accuracy is reduced and the free movement of the cone should be checked at minimum intervals of 1.0 m.

For accurate work, the mass of the pressure rod should be taken into account in calculations. In very soft soils when soundings are carried to a significant depth, the weight of the pressure rod may exceed the force on the cone or cone plus jacket; in these circumstances, it is impossible to obtain readings. This effect can be reduced by the use of aluminium pressure rods. The pressure rods should be free to slide inside the sounding tubes, and the cone, and friction jacket where used, should be checked for free sliding both at the start and at the end of each penetration test. All sounding tubes and pressure rods should be straight, clean and well oiled internally. The accuracy of the load and pressure gauges should be checked periodically by calibration.

23.3.3 Electrical Dutch cone penetrometer. A number of types of electrically-operated cone are in use, and these generally incorporate vibrating wire or impedance strain gauges for measuring the force on the cone and friction jacket. In use, the cone is advanced at a uniform rate of penetration by pressure on the top of the sounding tube, and signals from the load-measuring devices are carried by cable threaded through the sounding tubes to the surface. Forces on the cone, and friction jacket, can be displayed on a readout on the surface or recorded automatically on a chart recorder, punched tape or magnetic tape. Exclusive recording on punched tape or magnetic tape which does not allow direct access to the data during or immediately after sounding is not recommended. Provision should be made for calibration of the force-measuring system at regular intervals, preferably on site. An inclinometer built into the cone is available

jacket, where fitted, is immediately behind the point, as shown in figure 12. However, parallel-sided electrical cones do not give exactly the same results as those obtained with the mechanical Dutch cone penetrometer, although the difference is usually of little importance. Electrical cones are available with a modified profile which gives better agreement with the mechanical cone. The ISSMFE report covers a number of types of electrical cone.

23.3.4 Principal dimensions, accuracy of measurement and rate of penetration. The following recommendations apply to penetrometers with both mechanically-operated cones and electrically-operated cones:

- (a) cone cross-sectional area should be 1000 mm², and the cone apex angle should be 60°;
- (b) friction jacket surface area should be 15 000 mm²;
- (c) the rate of penetration should be 20 mm/s (± 5 mm/s);
- (d) the force measurements should be accurate to within 5 %.

23.3.5 Uses and limitations of the test. The cone penetrometer test is relatively quick to carry out, and cheap in comparison with boring, sampling and laboratory testing. It has traditionally been used to predict driving resistance, skin friction, and end bearing capacity of driven piles in non-cohesive soils.

In recent years, the cone penetrometer test has also been used to give indications of the continuous soil profile by interpretation of the ratio of frictional and cone resistances. In addition, there is substantial published information relating cone resistance value with other soil parameters.

It is also the preferred substitute for standard penetration tests in soil conditions where results of that test are suspect. The test is also commonly used as a rapid and economical means of interpolating between boreholes. Although it may be possible to estimate the type of soil through which the cone is passing as described above, it is preferable to carry out the test in conjunction with some other means of determining the nature of the soil present.

Penetration test equipment can be adapted to take small piston samples from specific horizons for identification purposes where required. The cone penetration test was originally developed for use in recent alluvial soils. Its application, however, has since been widened to other materials, such as overconsolidated clays and chalk. The depth of penetration is limited by both the safe load that can be carried by the cone, and the thrust available for pushing the penetrometer into the ground. Because of the cone capacity, penetration will normally have to be terminated when dense sand or gravel, coarse gravel, a cobble or rock is encountered. Similarly, build-up of skin friction along the rods, together with the load on the cone, can prevent further penetration because the total thrust capacity of the machine is reached. The friction on the sounding tubes can be reduced by the use of a friction reducing ring, shown in figure 11. Where a penetration test is stopped for any of the above reasons, it may be possible to prebore and continue the test from the bottom of the prebored hole. If this procedure is adopted, it is necessary to provide lateral support to rods in the prebored hole. Some penetration test rigs are equipped with rotary drilling and casing facilities in order