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British Standard

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BRITISH STANDARD

BS 5930:1999

Code of practice for site investigations

ICS 91.200

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Committees responsible for this British Standard

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Association of Geotechnical Experts
British Geological Survey
Chartered Institution of Water and Environmental Management
County Surveyors' Society
Department of the Environment, Transport and the Regions
Geological Society
Institution of Civil Engineers
Institution of Structural Engineers
Transport Research Laboratory

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Foreword

This British Standard has been prepared under the direction of the Sector Board for Building and Civil Engineering. It supersedes BS 5931:1981, which is withdrawn.

The original code of practice for civil engineers, *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 and a revised code was issued in 1981 as BS 5930. With one or two minor exceptions, this was a completely new text.

Under the continuing authority of the British Standards Institution, this revision has been prepared with the following objectives:

a) To clarify the scope of BS 5930:1981: in this code, the term "soil" has the meaning ascribed to it by common current usage in UK civil engineering and construction. A number of current British Standards define "soil" in relation to its use in agriculture and horticulture. In order to avoid any possible confusion, the title of BS 5930 has been revised in order to indicate its precise application.

b) To revise material in BS 5930:1981, which is now obsolescent. To this effect, there is new information on geophysical surveying and a new annex on site investigations on contaminated ground, as well as smaller amendments throughout the code.

c) To maintain compatibility with all the parts of BS 1377:1990, *Methods of test for soils for civil engineering purposes*. The 1990 edition of BS 1377 covers a much greater number of tests than the 1975 edition of BS 1377, so whenever one of these tests is referred to in the revised BS 5930, only a very brief description is given and reference is made to BS 1377:1990 for more detailed information.

d) To maintain conformity with European and International codes. In so far as information was available at the time of drafting, the revised BS 5930 is generally in agreement with comparable European and International codes.

NOTE 1 The numbers in square brackets used throughout the text of this code relate to the references in the Bibliography.

NOTE 2 This revision of BS 5930 was compiled prior to the major reorganization of the UK government departments subsequent to the UK General Election in May 1997. Throughout this text, the names of the various government departments are, in general, those appertaining prior to May 1997.

The European Committee for Standardization, CEN, has the responsibility for preparing a series of Eurocodes, which would harmonize practice throughout the European Community and eventually would replace the existing national codes. The British Standards Institution (BSI) represents British interests in CEN.

It is anticipated that a set of structural Eurocodes would relate to the structural and geotechnical design of civil engineering works.

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The proposed Eurocode 7, *Geotechnical design* and Eurocode 1, *Basis of design*, are likely to be relevant to BS 5930 and would give information that could be helpful in planning site investigations.

A further proposed group of European Standards, *The execution of special geotechnical works*, would give specifications for standardizing practice throughout the European Community.

Progress with the preparation of the Eurocodes is variable and as yet, none has reached final status.

As a code of practice, this British Standard takes the form of guidance and recommendations. It should not be quoted as if it were a specification and particular care should be taken to ensure that claims of compliance are not misleading.

A British Standard does not purport to include all the necessary provisions of a contract. Users of British Standards are responsible for their correct application.

Compliance with a British Standard does not of itself confer immunity from legal obligations.

Summary of pages

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1 Scope

This Code of Practice 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.

This code of practice consists of the following sections:

— *Section 1: Preliminary considerations*;

— *Section 2: Ground investigations*;

— *Section 3: Field investigations*;

— *Section 4: Field tests*;

— *Section 5: Laboratory tests on samples*;

— *Section 6: Description of soils and rocks*;

— *Section 7: Reports and interpretation.*

Section 1 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 2 discusses general aspects and planning of ground investigations, including the influence of general conditions and ground conditions of the selection of methods of investigation.

Sections 3, 4 and 5 discuss methods of ground investigation, sub-divided as follows: section 3 excavation, boring, sampling, probing and tests in boreholes; section 4: field tests; section 5: tests on samples. The division between sections 3 and 4 is somewhat arbitrary and has been based mainly on convenience in arranging the subject matter.

Section 6 deals with the terminology and systems recommended for use in describing and classifying soil and rock materials and rock masses.

Section 7 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.

Users of this Code of Practice, particularly those of limited experience, are advised against referring to the methods of ground investigation in sections 3, 4 and 5 without studying the preliminary considerations in sections 1 and 2. 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 continues 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 (see BS 1377), 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 Definitions

For the purposes of this Code of Practice, the terms and definitions given in BS 6100, *Glossary of building and civil engineering terms* apply, with the following exceptions.

2.1

soil and rock

have the meanings ascribed to them by common usage in UK civil engineering and construction and, as yet, there are no universally accepted definitions. An indication of the scope of these terms is found in section 6.

2.2

ground

general term covering soils, rocks and made ground

2.3

made ground

covers all deposits which have accumulated through human activity and may consist of natural materials, e.g. clay and/or man made materials, e.g. refuse

2.4

fill

"made ground" in which the material has been selected, placed and compacted in accordance with an engineering specification

2.5

groundwater control

has been adopted in preference to "dewatering", since the former is considered to convey a better description of the scope of the expedient

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3 Normative references

The following normative documents contain provisions, which, through reference in this text, constitute provisions of this Code of Practice. For dated references, subsequent amendments to, or revisions of, any of these publications do not apply. For undated references, the latest edition of the publication referred to applies.

BS 812-2, *Testing aggregates — Part 2: Methods for determination of density.*

BS 812-103, *Testing aggregates — Part 103: Method for determination of particle size distribution.*

BS 812-105, *Testing aggregates — Part 105: Methods for determination of particle shape.*

BS 882, *Specification for aggregates from natural sources for concrete.*

BS 1377-1:1990, *Methods of test for soils for civil engineering purposes — Part 1: General requirements and sample preparation.*

BS 1377-2:1990, *Methods of test for soils for civil engineering purposes — Part 2: Classification tests.*

BS 1377-3:1990, *Methods of test for soils for civil engineering purposes — Part 3: Chemical and electro-chemical tests.*

BS 1377-4:1990, *Methods of test for soils for civil engineering purposes — Part 4: Compaction-related tests.*

BS 1377-5:1990, *Methods of test for soils for civil engineering purposes — Part* 5: *Compressibility, permeability and durability tests.*

BS 1377-7:1990, *Methods of test for soils for civil engineering purposes — Part* 7: *Shear strength tests (total stress).*

BS 1377-8:1990, *Methods of test for soils for civil engineering purposes — Part 8: Shear strength tests (effective stress)*

BS 1377-9:1990, *Methods of test for soils for civil engineering purposes — Part 9: In-situ tests.*

BS 1881-6, *Testing concrete — Part 124: Methods for analysis of hardened concrete.*

BS 1924-1, *Stabilized materials for civil engineering purposes — Part 1: General requirements, sampling, sample preparation and tests on materials before stabilization.*

BS 4019-3:1993, *Rotary core drilling equipment — Part 3: Specification for System A — Metric units.*

BS 4019-4:1993, *Rotary core drilling equipment — Part 4: Specification for System A — Inch units.*

BS 5493:1977, *Code of practice for protective coating of iron and steel structures against corrosion.*

BS 6031, *Code of practice for earthworks.*

BS 6068-0:1995, *Water quality — Part 0: Introduction.*

BS 6068-2, *Water quality — Part 2: Physical, chemical and biochemical methods.*

BS 6316:1992, *Code of practice for test pumping of water wells.*

BS 7361-1, *Cathodic protection — Part 1: Code of practice for land and marine applications.*

BS 8004-1, *Code of practice for foundations.*

CP 2012, *Code of practice for foundations for machinery — Part 1: Foundations for reciprocating machines.*

4 Safety

This Code of Practice describes procedures for site investigations and all involve some risk to safety unless an appropriate safety plan has been prepared and implemented. Safety is considered in **9.4**. Elsewhere, safety is only mentioned where a special reminder has been considered necessary, e.g. work in contaminated ground. However, it is emphasized that safety is of paramount importance for every activity in site investigation.

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Section 1. Preliminary considerations

5 Primary objectives

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.

a) ***Suitability***. To assess the general suitability of the site and environs for the proposed works including, where applicable, the implications of any previous use or contamination of the site.

b) ***Design***. To enable an adequate and economic design to be prepared, including the design of temporary works.

c) ***Construction***. To plan the best method of construction; to foresee and provide against difficulties and delays that may arise during construction due to ground, groundwater and other local conditions; in appropriate cases, to explore sources of indigenous materials for use in construction (see **10.4**); 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.

f) ***Existing works***. Unless the contrary can be demonstrated, it should be assumed that site investigations are necessary in reporting upon the existing works (see **10.3**), and for investigating cases where failure has occurred (see **10.2**).

The ground is naturally variable and often the nature of these variations is not known in advance. A site investigation is a process of continuous exploration and interpretation, with the scope of the investigation requiring regular amendment in the light of the data being obtained. In order to evaluate properly the nature of the ground and the groundwater and so to achieve the objects of the site investigation, it is essential that the work be planned, undertaken and supervised by personnel who have appropriate qualifications, skills and experience in geotechnical work (see clause **17**). If this is not done, the results and conclusions of an investigation may be inadequate or even misleading and result in a considerable over-run of time and expenditure when the proposed works are under construction [1] [2].

6 Procedure

6.1 Extent and sequence

6.1.1 *General*

The extent of the investigation depends primarily upon the magnitude and nature of the proposed works and the nature of the site. The former use of a site, and the presence of contamination of the ground or groundwater can also have a significant impact on the extent of the investigation.

A site investigation should proceed in stages as follows:

Stage 1	Desk study and site reconnaissance.
Stage 2	Detailed investigation for design including ground investigation, topographic and hydrographic surveying and any special studies.
Stage 3	Construction review, including any follow-up investigations during construction, and the appraisal of performances.

Stage 1 should be undertaken at the start of every investigation. The desk study should include consideration of the possible existence of contaminated ground, where additional site safety procedures need to be established in advance of any intrusive investigation. It should also include environmental and ecological considerations, which might impose constraints on the execution of Stage 2. As far as possible, the assembly of the desk study information should be complete before the ground investigation (Stage 2) 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 **12**.

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 geotechnical advisor or appropriate specialist can help to formulate an efficient and economic plan.

6.1.2 *Adjacent property*

In view of the possibility that the construction of new works may affect adjacent property or interests, it is important that investigations for new works should consider all factors that may affect adjacent land or existing works and that, where possible and expedient, records of ground levels, groundwater 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.

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6.2 Desk study

It is essential to carry out a desk study as the first stage of a site investigation. The primary objectives of the desk study are to evaluate the ground conditions based upon existing information, and to plan the scope of the subsequent stages of investigations. Annex A indicates the kinds of information that may routinely 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 annex B, and a more detailed catalogue is given elsewhere [3].

Information based on local experience, including earlier uses of the site, may be available from local authorities and local or regional statutory undertakings (see clause **7**). 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 and 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 **8** and **B.6**). For sources of information about previous industrial uses, see annex E.

6.3 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 **6.1.2**). Annex C gives a summary of the procedure for site reconnaissance and the main points to be routinely considered. This procedure may need to be extended or modified, depending upon the particular circumstances of the site.

Nearby railway or road cuttings can reveal soil and rock types and their stability characteristics, as can old pits and quarries. Similarly, there may be embankments, buildings or other structures in the vicinity that have 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 annex E) or other underground excavations, such as old cellars, tunnels or sewers. The behaviour of structures similar to those planned also provides useful information, as does the absence of such structures, for example, a vacant site in the midst of otherwise intensive development may be significant. The type and abundance of vegetation on a site should be noted and can provide important information on chemical contamination of the ground, or the presence of soil gases (see annex F).

For instances of earlier uses of the site that may affect new construction works, see clause **7**.

6.4 Detailed examination and special studies (see also annex D)

For most projects, the design and planning of construction requires 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 2. 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**); for other environmental and ecological considerations.

Where mining and quarrying, whether past, present or prospective, is likely to be a factor affecting the site, reference should be made to annex E.1. Where contamination of the ground or the presence of soil gases is possible, reference should be made to annex F. Effects of tunnelling, where this is a possibility, should be considered.

6.5 Construction and performance appraisal

Construction and performance appraisals are discussed in clause **18**.

7 Earlier uses and state of site

7.1 General

If a site has been used for other purposes in the past, this can have a major 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 **7.2** to **7.7**.

The former uses of the site can sometimes be discovered by a study of old Ordnance Survey and earlier maps. In some cases, the forerunners of local authorities commissioned the making of maps of their areas long before Ordnance maps were produced, and these maps were often drawn using the most meticulous detail. Former occupiers of the site may also be able to provide detailed maps indicating the layout of facilities.

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

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7.2 Underground mining

Annex E gives details of the procedure that should be followed for enquiries and the subsequent site investigation for an area where underground mining has been or is currently being carried out, or is envisaged.

7.3 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 annex E. Other minerals have also been extracted by opencast methods.

7.4 Quarry operations

The quarrying of materials such as stone, chalk, sand, gravel and clay has been carried out since ancient times and, over the years, many excavations have been back-filled and then put to some other use. Many such sites have subsequently been used for waste disposal (see **7.5** and annex F).

7.5 Waste tips and landfills

The site may have been used for the tipping of mining waste, industrial waste, domestic refuse, chemical waste and miscellaneous refuse. Such sites require special consideration before any intrusive investigation is carried out to ensure the safety of site investigation workers and protection of the environment. Frequently, harmful chemicals and toxic or explosive gases may be present. Consultation with the Environment Agency in England and Wales and the Scottish Environmental Protection Agency in Scotland and Waste Regulation Authority is essential during the planning stage of investigations in such sites. See also annex F.

7.6 Industrial sites

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 including piles and sheet piled walls, engine beds, pits and chambers, often of massive construction, which can be major obstacles to redevelopment. The ground may still be affected by extremes of temperature from installations such as cold-storage plants, high-temperature kilns and furnaces. The ground is also frequently contaminated by spilled chemicals, leaking sumps and drains, the spreading or burial of waste, or the importation of contaminated fill. Historic aerial photography can be very useful in identifying the locations of physical obstructions and potential chemical contamination (see clause **8**).

7.7 Other earlier use

Redevelopment of urban areas may involve running into disused tunnels and encountering obstacles such as cesspits, sewers, wells, cellars, drains and communication passages.. It is essential that site investigations in such areas cover both the physical obstacles and the potential contamination of the ground.

7.8 Ancient monuments

If it becomes 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.

7.9 Ecology

Ecological issues need to be carefully considered, because they could impose significant constraints on both the execution of the site investigation and on the future development of the site. A variety of plants, animals and habitats is specially protected by law e.g. The Conservation (National habitats etc.) Regulations 1994.

8 Aerial photographs and satellite imagery

8.1 General

Aerial photographs and satellite imagery can be used both in the preparation and revision of maps and plans and for the interpretation of site features or earlier uses of a site. They are particularly useful for mapping large, undeveloped sites, notably for such projects as dams, power stations, and highways, although they can also be used for mapping smaller sites.

During the desk study stage they can 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. [4], [5] Although a non-specialist can often extract a substantial amount of relevant information from satellite imagery, trained interpreters are necessary if full information is to be obtained [6] Interpreters should be carefully briefed on the requirements of the interpretation. A considerable database of aerial photography has been available since the 1940s, particularly in urban areas, and existing sources should be checked (see **B.6**) before special photography is commissioned, which could prove to be very expensive and unnecessary.

8.2 Topographical mapping

Accurate, contoured maps can be produced from aerial photographs by a competent survey organization that normally carries 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. Normally, photography is available at the various scales considered appropriate for level map or plan making. The scales 1:500, 1:1 000, and 1:2 500 are most appropriate for investigations of limited areas, whereas scales of 1:5 000 to 1:20 000 are more appropriate to regional studies. Although much map revision of urban areas is carried out by the Ordnance Survey using aerial survey, specially commissioned surveys are not likely to be justifiable for small urban sites.

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Orthophotomaps, unlike conventional vertical photographs, are true to scale. They consist of rectified (true to scale) photographs overprinted with contours and grid, and for some purposes provide an alternative to a conventional map or plan; they may be of particular value to major site investigations.

8.3 Identification of features

Aerial photographs can often be used to identify such features of engineering significance as geological lineaments, for example, strata boundaries, faults, soil and rock types, landforms, 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 where 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.

8.4 Earlier uses of sites

Inspection of historic aerial photography can yield valuable information of previous construction and earthworks on a site, and is an essential part of the desk study of former industrial, quarrying and waste tip sites. The RAF carried out extensive photography in the early 1940s and many areas have been subject to repeated photography by commercial sources over the past 50 years. Annex B gives details of sources of historic aerial photography, and annex F describes particular features of importance to be noted on potentially contaminated sites.

8.5 Sophisticated techniques

Natural colour does have particular advantages. Multispectral techniques providing simultaneous images in selective wave bands, including the infra-red, are also available. One example of the uses of infra-red photography is to identify vegetation "distress" caused by landfill gas, chemical contamination, water deficiency or heat. The use of false colour to enhance such features as vegetation is possible when selective wave bands have been used [4]. 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.

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Section 2. Ground investigations

9 Introduction

9.1 Objectives

For new works, the objectives of ground investigations are to obtain reliable information to produce an economic and safe design, to assess any hazards (physical or chemical) associated with the ground, and to meet tender and construction requirements. The investigation should be designed to verify and expand information previously collected.

The objectives of ground investigations related to defects or failures of existing works (see **10.2**) or to the safety of existing works (see **10.3**), are directly related to the particular problems involved. The requirements for investigation of materials for construction purposes are discussed in **10.4**.

An understanding of the geology of the site is a fundamental requirement in the planning and interpretation of ground investigations. 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 **11**.

Of primary importance is the establishment of the soil profile or soil and rock profile, and the groundwater condition. The profile should be obtained by close visual inspection and systematic description of the ground using the methods and terminology given in section 2. Provided the profile is supplemented by limited in-situ or laboratory testing, in many cases this suffices. In others, it is necessary to determine the engineering properties of the soils and rocks in detail. The extent of the ground investigation is discussed in clause **12**. Where appropriate the geometry and nature of discontinuities should be established (see **14.10**).

The investigation should cover all ground in which significant 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 groundwater level and flow pattern; and changes in properties of the ground, such as strength and compressibility. Materials placed in the ground may deteriorate, especially in landfill and contaminated former industrial sites. It is therefore necessary to provide information from which an estimate of the corrosivity of the ground can be made (see clause **15**).

In certain districts, special measures need to be taken to locate underground cavities, such as old mine workings or swallow holes, which may collapse and cause damage to structures. Other hazards may arise on account of earlier uses of the site (see clause **7**).

9.2 Planning and control

Before commencing the ground investigation, all relevant information collected from the sources discussed in section 1 should be considered, in order to form a preliminary view of the ground conditions and the engineering problems that may be involved.

This assists 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 main investigation obtains the bulk of the information required, but it may be necessary to carry out supplementary investigations after the main work to gather more detailed information related to specific matters.

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 this.

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 groundwater 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 (for reasons of cost and time) of limitations on the amount of ground investigation to be undertaken 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.

It is essential that there be adequate direction and supervision of the work by a competent person who has appropriate knowledge, training and experience and the authority to decide on variations to the ground investigation when required (see clauses **5** and **17**).

Guidance on planning a site investigation is found in [7] and a suitable specification in [8].

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9.3 Quality management

This involves the construction and implementation of a managerial plan, designed to ensure that the results of the site investigation are submitted within the required standards for accuracy and presentation, as defined in the specification.

Depending on the objectives, the amount and types of investigation can vary from the very simple, e.g. boring, to establish the level of an easily recognized rock-head, to the highly complex, involving sophisticated techniques for boring, sampling and testing.

The quality management plan consists of a series of written procedures covering some or all of the following components of a site investigation:

a) equipment: standard of maintenance, frequency of checks on performance, frequency of calibrations;

b) testing: procedures including checking of observations and calculations;

c) personnel: all personnel should have qualifications, training and experience appropriate to their duties;

d) audits to check that the quality management plan is being fully implemented: depending on circumstances, these may be done by internal or external auditors.

Many organizations and personnel already have some form of quality accreditation from external authorities. Examples include:

1) Accreditation for Quality Assurance, QA: this certifies that an organization has quality management systems in operation which satisfy the requirements of BS EN ISO 9000 series, in respect of those parts appropriate to site investigations.

2) UKAS: this certifies that an organization carries out tests strictly in accordance with the appropriate British Standard or International Standard, e.g. BS 1377, *Methods of test for soils for civil engineering purposes*.

3) Personnel are usually accredited in terms of vocational, academic or professional qualifications together with suitable specialist experience or training. The British Drilling Association operates a scheme for the accreditation of site investigation drillers.

Guidelines on quality management of site investigations are given in [7].

9.4 Safety

Safety is relevant to all aspects of site investigation. This code cannot attempt to address all the very important and wide ranging aspects of safety, including procedures to be followed and precautions to be taken on site. It is only in this section that reference is made to general safety requirements; the guidance is not repeated in the other sections describing specific activities.

The Health and Safety at Work etc. Act 1974 [9] established formal rules for the management of safety and defined clearly the responsibilities of organizations, employers and employees. Within the umbrella of this Act there exists a framework of Regulations (see [9] to [18]), Approved Codes of Practice, Codes of Practice, Guidance Notes and other published information which define the requirement for safety on sites where investigation and construction work is being carried out.

The complexity and volume of EC directives and safety regulations make it difficult to communicate all the changes relating to safety in the work place. Safety directions are constantly under preparation. The most recent of the Regulations also have an Approved Code of Practice issued by the Health and Safety Executive and reference should be made to these Codes of Practices.

Other documents relating to safety in investigation work may be found in the Bibliography ([19] to [23]).

A combination of badly managed events often results in death and injury. Safety can always be improved by raising awareness of the dangers, provided that there is the management will to adopt a "Safety culture".

All employers of five or more persons have a statutory duty to prepare and revise as necessary a written safety policy and communicate this to all employees.

Under the Management of Health and Safety Regulations 1992, [12] firms employing similar numbers of employees (five or more) are required to implement a formal approach to safety. This is normally undertaken through a Safety Management System, which requires induction, risk assessments, method statements, consultation and audit.

The Construction (Design and Management) Regulations 1994, [16] effective from 31 March 1995, cover notification of the project, impose duties on all clients, contractors and employees, create additional duty holders of Planning Supervisor and Principal Contractor, require that a Site Safety plan is prepared before construction (and investigation) work can proceed and that all work is carried out to the safety plan or its subsequent revisions. The CDM Regulations apply to a construction project at a particular location and to all contracts that may be awarded as part of that project. They are designed to provide a continuous and integrated system for the avoidance, reduction and management of health and safety risks from conception to commissioning and handover of the finished project.

Most ground investigations, geotechnical advice and design and other associated work falls within the CDM Regulations even though they often form part of the concept, feasibility, design and planning stages, as well as being required to be undertaken on some occasions during the construction phase. Ground investigation is included in the listed definitions of construction work [11] and it therefore

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follows that the construction phase for any project may well commence with the ground investigation fieldwork. The project is notifiable if the construction phase is longer than 30 days or involves more than 500 person days of construction work.

It is essential that reference is made to regulation 3 to determine whether all or any of the regulations apply to any proposed investigation work. Projects that fall outside the full scope of the Regulations may include:

a) where the project is not notifiable and involves no more than four persons at work at any one time;

b) where the Local Authority is the enforcing authority for health and safety purposes [12] [13].

Where CDM Regulations do not apply, other Health and Safety Regulations still apply to the employers, employees and self employed carrying out construction work.

The Control of Substances Hazardous to Health Regulations 1988, cover the protection of people at work who may be exposed to health risks arising from hazardous substances.

In particular employers are required to:

— assess the hazard potential of all substances and work processes and record them in writing;

— identify how exposure is controlled;

— explain how any control measures are monitored;

— identify health surveillance needs;

— establish a record keeping programme;

— arrange for suitable training.

When investigating sites there are usually two potential sources of hazardous substances: those that exist on site (contaminated land); and those purchased by the contractor to enable the works to be carried out (cement, bentonite, fuels etc.). In the case of a potentially contaminated site the initial safety assessment for a potentially hazardous site should list the likely substances to be encountered. The assessment provides the basis of the site specific Safety Plan which includes the requirements for personal protective equipment (PPE).

In the case of purchased materials it is the responsibility of the contractor to consider each individual site and to prepare a "notification of use and assessment" for each hazardous substance. It is important to include on the sheet the procedures to be adopted to control the hazardous substance. Where products such as cement, bentonite, fuel etc. are used daily on various investigation sites, these substances may be dealt with by using general assessment sheets prepared in the Head Office, provided that they are retained on site and all site staff and operatives are aware of the existence of these assessments.

When work is being carried out on premises controlled by public authorities or large commercial or utility organizations, such as those in the railways, water, oil and gas industries, compliance is required with the organizations' own regulations, which include the established precautions relating to their particular business and its hazards.

Certain risks are attached to working on investigation sites, not least of which are road traffic accidents, which occur not only on site, but also to employees travelling long distances. The advice in the booklet *Safer Road Works Ahead* [24] should be followed when signing roads, except those on motorways and high speed dual carriageways with hard shoulders. Detailed traffic management procedures for major trunk roads and motorways are outlined in the *Traffic Signs Manual* [25]. Attention is also drawn to The New Roads and Street Works Act, 1991 [26] and The Street Works (Qualification of Supervisors and Operatives) Regulations, 1992 [27].

Risks associated with particular circumstances or hazards should also be assessed and provision made for safe working in the site safety plan. These include underground and overhead services, underground ducts and tunnels, work over and adjacent to water, geophysical surveys, diving services, work underground, work on contaminated land [28], abseiling and the use of equipment containing radioactive sources. All these types of work should be undertaken in accordance with current practices and the existing appropriate Regulations.

10 Types of ground investigation

10.1 Sites for new works

Investigations for new works differ from the other types of investigation mentioned in clause **9**, in that they are usually wider in scope; this is because they are required to yield information to assist in selecting the most suitable location for the works. For instance, when an excavation has to be carried out, a knowledge of the subsurface strata and groundwater conditions should indicate, for example:

a) whether removal of the material is difficult;

b) whether the side of the excavation is stable if unsupported or requires support (see BS 6031);

c) whether groundwater conditions necessitate special precautions such as groundwater or other geotechnical processes;

d) whether the nature of the excavated material will change;

e) whether any of the soil or groundwater is contaminated, therefore requiring special controls on excavation, movement, disposal, and additional safety measures;

f) whether environmental or ecological considerations might impose any constraints on the scope of the new works.

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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, earth pressures on supporting structures, and the effect of any chemically aggressive or hazardous 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 groundwater variation 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 factor.

10.2 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 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 is required to ascertain the condition of the strata and the groundwater conditions, both as they exist in-situ and as they existed before the works were constructed. Each problem needs to be considered individually. Indications of the probable cause of a failure 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 on the surface and underground. This is achieved by conventional survey methods and by instruments such as slip indicators, tilt meters, inclinometers etc. Automatic data recorders and warning systems may be used to monitor potentially unstable conditions. These techniques are fully described in BS 6031.

An investigation to determine the causes of a failure may be much more detailed than an investigation for new works.

10.3 Safety of existing works

10.3.1 *Effect of new works upon existing works*

Existing works close to, or even at a distance from, the proposed site may need to be investigated, to decide whether they are likely to be affected by changes in the ground and groundwater conditions brought about by the new works.

10.3.2 *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 and on ground deformation 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;

e) lowering of the general groundwater level by pumping from wells which causes an increase in the effective stress in the subsoil affected, and can lead to excessive settlement of adjacent structures; also, if wells do not have an adequate filter, the leaching of fines from the subsoil can easily result in excessive settlement of structures at a considerable distance from the well;

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 groundwater level; this can cause softening of cohesive strata and a reduction of bearing capacity of permeable strata, as well as giving rise to increased pore pressures affecting the stability of slopes and retaining walls; swelling may result in ground heave;

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;

k) disturbance of contaminated ground, which may allow aggressive leachates or noxious gases to migrate through the ground.

10.3.3 *Procedure*

Given problems of this kind, it is important to first establish what changes are likely to occur. Knowledge of the subsurface strata needs to be determined from the ground investigation and samples need to be tested and examined 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.

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10.4 Material for construction purposes

Investigations of sites are sometimes required:

a) to assess the suitability and quantities of material that become available from excavations or dredgings for construction work, e.g. whether spoil from cuts in roads and railway works is 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);

c) to assess the chemical characteristics of ground that may need to be removed for environmental reasons;

d) to locate suitable disposal sites for excess spoil and dredged materials.

In works such as railways and roads, a prior knowledge of the strata on the line of the 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 visual 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 will be necessary to carry out tests to determine what steps are required to make the best use of it (see [29]]. Published and unpublished records of the Institute of Geological Sciences and the British Geological survey are particularly recommended for investigations of this kind (see **B.2**).

11 Geological mapping for ground investigation

Information concerning the geological survey maps is given in **B.2**. If this is insufficiently detailed for a particular investigation, it may be necessary to carry out additional geological mapping. Nevertheless, even when the maps are considered to be sufficiently detailed, a walkover survey should still be undertaken where possible. Methods used for geological mapping [30] [31] are equally suitable for ground investigation purposes, and may be supplemented by geophysical investigations (see clause **35**) and aerial photographs (see clause **8**). The object of geological 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 therefore require the mapping of 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 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. Extrapolation of data should be avoided as a general rule. Interpolation should be made with care; 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 using a large scale and samples taken before backfilling takes place.

As the construction work proceeds, the actual geological conditions exposed in the various excavations should be logged so that the previous geological reports can be revised as necessary.

Further information and examples of engineering geological mapping are given elsewhere (see [32] and [33]).

12 Extent of the ground investigation

12.1 General

The extent of the ground investigation is determined by the character and variability of the ground and groundwater, the type of project and the amount and quality 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. It may be that the desk study of existing information will provide sufficient high quality data for a preliminary design to be started with confidence.

A range of "methods" is used to carry out ground investigations. These include intrusive methods such as excavations, boreholes and probing (see section 2), and geophysical surveying (see clause **35**).

The factors determining the selection of a particular method are discussed in clauses **13** and **14**, and in section 2. In general, the recommendations in **12.2** to **12.7** 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.

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Certain methods of investigation are more suitable than others on contaminated land. Also the density and layout of exploration points will need to recognize the location and depth of potential contamination on a site. Annex F provides further guidance on these matters.

12.2 Character and variability of the ground

The greater the natural variability of the ground, the greater the extent of the ground investigation required to obtain an indication of the character of the ground. The depth of exploration is usually determined by the nature of the works projected, but it may be necessary to explore to greater depths at a 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 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, from surface outcrops and from 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. Even an intensive investigation can only reduce uncertainties; a complete excavation is the only way to reveal the true nature of the ground. The use of angled drill holes 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 **35**).

12.3 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 be able to decide which of the various possible methods of construction would be desirable and, where appropriate, to suggest sources of construction 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.

12.4 Preliminary investigation

Depending on the results of the desk study, and the size and complexity of the project, it may be advisable to carry out the investigation in stages. The initial stage might involve widely spaced boreholes, probing, or trial pits designed to establish the general geological conditions, the suitability of different methods of investigation, the groundwater conditions, or an indication of the degree of chemical contamination. This preliminary investigation assists in producing an effective programme for the detailed investigation.

12.5 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 providing details of the engineering properties of the soils and rocks and of groundwater 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. In the absence of other criteria, a regular array of exploration points may be used in the initial design of an investigation. However, sufficiently close supervision and flexibility (in the contract) should be provided to allow amendments to be made to this pattern as the work proceeds. Sometimes it is not possible to locate structures until much of the ground investigation data has been obtained. In such cases, the programme of work should be modified accordingly.

When investigating tunnels and 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 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 and material obtained to test suitability for use as fill material.

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

12.6 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, are often appropriate for structures. For structures small in plan area, exploration should be made at a minimum of three points, unless other reliable information is available in the immediate vicinity.

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 related more closely to the detailed geology of the area than is usual for other works.

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12.7 Depth of exploration

12.7.1 *General*

The depth of exploration depends on how new building work significantly affects the ground and groundwater, or is affected by them. Normally exploration should be undertaken below all deposits that may be unsuitable for foundations purposes, e.g. made ground and weak compressible soils, including weak strata overlain by a layer of higher bearing capacity. The exploration should go through compressible cohesive soils likely to contribute significantly to the settlement of the proposed works, normally 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 clause **44**) 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 specific recommendations are given in **12.7.2** to **12.7.8**.

It is not always necessary that every exploration should be taken to the depths recommended in **12.7.2** to **12.7.8**. In many instances, it is 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 a little higher to explore more thoroughly the zone near the surface which the initial exploration had shown to be most relevant to the problem in hand.

12.7.2 *Foundations for structures*

For these, the depth of exploration should be at least one and a half times the width of the loaded area, unless the imposed stress change becomes insignificant when compared with the strength and stiffness of the ground at a lesser depth, e.g. in strong rock. For foundations near the surface, the loaded area is considered as either:

a) the area of an individual footing; or

b) the plan area of the structures, where the spacing of foundation footings is less than about three times the breadth, or where the floor loading is significant; 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, a preliminary analysis of the possible lengths of different types and sizes of pile should be made using the desk study information. In addition, the following factors should be taken into account when planning the depth of exploration.

1) Pile groups stress the ground below the pile points to a greater depth than will an individual isolated pile. The depth of exploration should be sufficient to identify any weak strata beneath the pile points, which might affect the bearing capacity of the pile groups.

2) 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, has to be borne by the stronger strata lying below the weak materials.

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, which 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 at underside of pile cap level 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 and the engineer directing the investigation should terminate the exploration once relatively incompressible strata have been reached, provided that the depth of exploration is sufficient to prove the strata well below the toes of the piles. Similar considerations apply to groups of end bearing piles in coarse grained soils, except that in this case the imaginary raft should be assumed to be at the pile toe level, with an area extent the same as the pile group.

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.

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Based on the information of the probable ground profile obtained from the desk study, the general guidance given in 1) to 5) and the 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 be necessary. In any event, exploration should at some points be taken below the depth of which it is considered likely that the longest piles 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.

12.7.3 ***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.

12.7.4 ***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 require proving the full depth of any relatively weak strata. Groundwater conditions, including the possibility of artesian water, should also be determined.

12.7.5 ***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 is probably sufficient in most cases. A greater depth may be needed where a road is to pass over a shrinkable subgrade through a heavily vegetated area.

12.7.6 ***Pipelines***

For small, shallow pipelines, it is frequently sufficient to take exploration to 1 m below the invert level. For deeper pipelines the depth of exploration should be sufficient to enable dewatering and to allow any likely difficulties in excavating trenches and supporting the pipelines to be investigated. This means that a depth of 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.

12.7.7 ***Maritime works***

For maritime works, the possible effects described in **12.7.2** and **12.7.3** should be considered as well as the effects of tidal variations.

12.7.8 ***Tunnels***

For tunnels, it is important to take the exploration to an adequate depth below the proposed invert level, both 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. It is also necessary to take some of the boreholes to a depth sufficient to establish the overall geological structure (note also **12.5** on the location of the boreholes).

12.8 Contaminated ground

When establishing the extent of the site investigation for developments on ground that has been contaminated by former use the design of remedial ground treatment and the geotechnical design of foundations and services need to be taken into account. The type, spacing and depth of investigation points are determined by the objectives of the investigation, as well as the prevailing geological conditions, the nature and extent of the contamination, and the proposed end use. Further guidance on the design of site investigation on contaminated ground is given in annex F.

13 General considerations in the selection of methods of ground investigation

13.1 Introduction

Although the character of the ground, the objectives and technical requirements are the most important aspects 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.

13.2 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 exploratory holes, as well as probing, in-situ testing, etc. 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 are required. On very steeply sloping open sites, it may be necessary to construct an access road and lower the equipment down the slope or haul it up. Where the working position is on steeply sloping ground, it is necessary to form a horizontal working area by excavation or the use of a staging. Gaining access to sites covered by water also presents special problems (see clause **16**). 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

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confined space. Investigations inside existing buildings are usually carried out with cut-down rigs, portable drive sampling equipment or hand excavation. In such cases, the depth of such investigation is usually restricted.

Environmental and ecological conditions also need to be taken into account since these may impose constraints on both the locations and on the equipment that can be used.

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 [34] controls permissions to take water from any natural source. On a site where water is not available, it is necessary to arrange for a temporary supply to be provided. This usually involves pumping through pipelines or transporting the water in bowsers. 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 considered instead of water flush, see **20.7**.

In built-up areas the avoidance of noise nuisance, the limitation of the working area, and the enclosure of the works by hoardings or safety barriers can restrict the methods and progress of the investigation. Surface obstruction and buried services may also interfere with geophysical survey methods as well as restrict intrusive investigation.

14 The effect of ground conditions on the selection of methods of ground investigation

14.1 Introduction

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, are considered in **14.2** to **14.10**. Frequent reference is made to classes of sample quality; these are defined in **21.2**.

The determination of the groundwater conditions is a most important part of a ground investigation. It usually involves the installation of piezometers and the performance of field testing in piezometers or boreholes (see clauses **22**, **23** and **27**). In general the investigation of the groundwater conditions does not govern the choice of the method of exploration. Where the ground is contaminated, however, specific investigations are usually required to define the chemical constituents of the groundwater at various depths across the site, as well as to examine the hydraulic characteristics of the ground and the direction of flow (see annex F).

The presence of chemical contamination, or of hazardous soil gases, is a significant factor in the selection of the method of investigation, or in the conduct of the work. It is vital for the investigation to be carried out under strict health and safety guidelines, in order to prevent the creation of contaminants and their migration pathways and in order to provide reliable samples for chemical analysis.

14.2 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 **20.1** and **20.2**). If it is necessary to investigate the ground below the groundwater table, dewatering is 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 **31** and **32**).

When boring, there may be difficulty in advancing the boreholes and consequently in obtaining samples of adequate quality. Normally, the most suitable method of boring is with light cable percussion (see **20.5**) using the shell and by 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, which 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 on 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 that use high-powered machines may sometimes be used to obtain a faster rate of progress than is obtained by light cable percussion. Both methods require 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 **22.4** to **22.7** 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 **24.2**), with the 60° cone replacing the cutting shoe, gives some indication of the relative density in gravel. Occasional high values that are unrepresentative of the true relative density are 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.

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The borehole permeability test (see **24.4**), 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 is obtained from a pumping test (see clause **27**). The static cone penetration test (see **26.3**) 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. Dynamic probing (see **26.2**) may give useful results provided the density is not too high.

14.3 Sand

Boreholes in sand are usually sunk by the light cable percussion method using a shell. This requires sufficient water in the borehole to cover the shell, as it is 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 sands tend to "blow" up the borehole, therefore loosening the ground below the bottom of it. Keeping the borehole full of water, and using an undersized shell carefully 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 water, and are therefore only class 5. Samples suitable for particle size distribution, class 4 are usually obtained using the split barrel standard penetration test sampler (see **22.4.5**). Larger class 4 samples can usually be obtained by using the 100 mm open-tube sampling equipment with two or three tubes coupled together and with a core catcher fitted above the cutting shoe (see **22.4.4**).

The action of forcing a sampler into sand tends to cause a change in volume, even if the area ratio is small (see **22.4.1.2**c)), and hence the density of the sample may not be representative of the stratum. In some cases, a piston sampler is effective (see **22.5**); 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.

In shallow investigations above the water table, excavations or hand augering (see **20.4**) may be used.

A guide to the relative density of sands is obtained by the standard penetration test. However, the results can easily be invalidated if, as previously discussed, sand is loosened below the water table. 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 static cone penetration test (see **26.3**).

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 static cone penetration test. Pressuremeter tests are also useful. A more direct determination requires the use of plate tests carried out in a dry excavation (see **25.6** and clause **31**).

Permeability may be assessed from borehole permeability tests (see **25.4**) or by pumping tests (see clause **27**).

14.4 Silt

Silt is a difficult material to sample and 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. The shell sample may be deficient in fines owing to the action of the water and therefore only class 5. Above the water table, it may be possible to use the clay cutter and recover samples of class 3 or 4. Silt is usually sufficiently cohesive to allow the recovery of samples using the 100 mm open-tube sampler. 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 usually only class 3. In low or medium density silt, class 2 samples can be obtained using a thin-walled piston sampler (see **22.5**).

In preliminary investigations, the standard penetration test is used to obtain an indication of the relative density and is usually 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 static cone penetration test (see **26.3**).

14.5 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 are class 4 provided that lumps of intact clay can be recovered. Samples taken with an open-tube sampler 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 or 3. 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.

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The vane test (see **25.3**) is useful for the measurement of the undrained shear strength of normally consolidated or lightly overconsolidated clays. Conventional oedometer tests have severe limitations in respect of determination of rate of settlement (see clause **39**, which deals with tests on soils, in particular Table 10 under "Soil deformation tests"). 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 hydraulic odometer tests can be conducted in the laboratory. Probing may be used to determine the thickness of superficial soft clays and peats.

The static cone penetration test can be used to provide continuous in-situ shear strength profiles.

14.6 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 although this can lead 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 usually suffer only a small amount of disturbance, and are generally accepted as classes 1 or 2. With very stiff and hard clays, however, there is significant disturbance. Thin-wall samplers jacked into the ground can produce better quality samples. Where the nature of the problem warrants greater accuracy, the 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 **25.6**).

Standard penetration tests can give a useful indication of shear strength (see **25.2**).

There may be a significant amount of disturbance in laminated clays, particularly if there is no water head in the boreholes. in addition, 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.

14.7 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 enables 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 are broken up by the chisel and much of the fine matrix is 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.

An alternative borehole method, for use when the clay matrix is stiff or hard, is rotary core drilling (see **20.7**). 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 are 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 give an indication of the structure of the ground, but the moisture content might be altered by the use of the drill flush.

The standard penetration test (see **24.2**) is useful in these soils to give a rough indication of strength, which may be adequate in some problems. Pressuremeter tests can also be useful (see **24.7**).

14.8 Filled ground

Filled ground can consist of selected materials placed and compacted in a controlled manner and generally known as "fill". Alternatively, it can consist of replaced natural ground, waste soils, demolition rubble or other materials of various origins, laid with little or no supervision and generally known as "made ground". In the latter case the quality and consistency of the material, and the method of placement and compaction affects its uniformity. Normally, there is little or no control of the fill and the major problem in planning the ground investigation is to assess the variation in character and quality across the site. Often the variation is random.

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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 fill at the particular locations of the boreholes or in-situ test. It is essential to ensure that boreholes are always fully cased through uncontrolled fill to avoid contamination of the natural groundwater and underlying strata. Pits and trenches are particularly useful for investigating the nature and variability of made ground (see clause **20**).

In combustible fills, temperature measurements should be carried out as a routine part of the investigation. It should be noted that on waste tips, burning materials below ground may give rise to toxic or flammable gas 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.

Further guidance on the investigation of colliery waste tips is given in annex E. Particular guidance on the investigation of landfill sites and contaminated land is given in annex F.

14.9 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 are 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 the sampler usually 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 often in the range class 3 to 5. 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 drilling or excavations. Rotary core drilling with diamond or tungsten carbide bits is normally used in rock strata (see **20.7**). In weathered weak rock and in harder rocks that are closely bedded, 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, helps to improve the core recovery. Where weathered rock occurs at shallow depth, pits are probably the most effective means of investigation, because block samples can be obtained from these 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. Rotary core drilling usually produces 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 connection. In many cases, the sample disturbance associated with rotary drilling causes any infilling in the discontinuities to be lost and hence the rotary core samples give little or no indication of the character of the infilling. In-situ permeability by packer testing (see **24.5**) may give a useful indication.

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

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 is necessary to take into account the probable effects of disturbance of the ground by the boring process. The standard penetration test (see **25.2**) can give a rough indication of the variation of strength and compressibility in weak rock. The permeability test (see **25.4**) or the packer or Lugeon test (see **25.5**) 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 **25.5**) and dilatometers such as the pressuremeter (see **25.7**) 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 in-situ inspection, and field tests carried out in excavations, shafts, or headings (see **20.1, 20.2** and **20.3**).

14.10 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.

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No satisfactory drilling or boring techniques are available that ensure the core recovered can be oriented 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 nature. After initial investigations using interpretation of aerial photographs, surface outcrop logging and the drilling of vertical and inclined oriented 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 oriented large scale tests. The orientation of the excavations controls their intersection with the discontinuities and, consequently, the discontinuity data that can be obtained. Normally, 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**).

15 Aggressive ground conditions

15.1 Introduction

In certain localities, groundwater, soil and rock may contain constituents in amounts sufficient to cause damage to Portland cement concrete, particularly thin members. The constituents principally concerned are sulfates, which are most common in clay soils, and acidic waters, which are found in or derived from peaty soils and from the breakdown of pyrites in many other rocks.

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, from contaminated materials brought onto the site as fill, or from liquid sources such as leaking tanks, sumps and chemical drains. In river and maritime works, the possible corrosive action of fresh 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. Disturbed soils are often more aggressive than undisturbed soils.

15.2 Investigation of potential deterioration of concrete

Laboratory tests to assess the aggressiveness of the ground and groundwater against Portland cement concrete include determination of pH value and sulfate content (see clause **39**, which deals with tests on soils and its corresponding Table 10, in particular “Chemical and electro-chemical tests”). [35]

15.3 Investigation of potential corrosion of metals

The likelihood of corrosive ground conditions can be assessed from geophysical testing (see **35.5**), from in-situ measurement of redox potential and electrical conductivity (see BS 1377:1990), and from laboratory tests on undisturbed soil specimens and groundwater samples. The samples should be placed in air-sealed containers immediately after sampling (see clause **39**, which deals with tests on soils and its corresponding Table 10, in particular “Soil corrosivity tests”). (See also [35].) For specimens for bacteriological tests, the containers should be sterilized. When designing a testing programme it is important to model the correct conditions. Disturbance to the ground and the use of imported backfill materials can have a significant impact on the evaluation. Local enquiry is desirable to ascertain whether corrosion of metals has previously occurred.

15.4 Industrial causes of aggressive ground

Many types of industry are known to cause contamination of the ground through the disposal of solid and liquid wastes, and through the inevitable leaks and spills of chemicals. A very wide range of substances may be present in the ground, and often occur in localized concentrations. Some chemicals, such as acids, are highly aggressive to both concrete and metal in underground structures. Other substances such as solvents attack epoxy cements, and even low concentrations of phenols permeate plastic water mains and taint the supply.

A thorough investigation of potential chemical contamination is necessary as an essential part of any site investigation on a former industrial site. Guidance on the planning and execution of such an investigation is given in annex F.

16 Ground investigations over water

16.1 Introduction

Site investigations conducted over water are more expensive and time consuming than comparable investigations conducted on 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. Geophysical surveys are extensively used at the planning stage to provide additional geological information for the ground beneath the construction site and the positioning of the investigation boreholes (see **35.4**).

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The sinking of boreholes below water presents special difficulties particularly with regard to rotary diamond core drilling. Working platforms can be fixed, floating or heave compensated. A conductor pipe is usually suspended from the platform to protect the flexible investigation string from the force of currents. Percussion boring and rotary drilling techniques, both conventional and wireline, may be employed. Geophysical logging techniques are often employed to augment the information obtained from the borehole programme (see **35.5**).

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 by 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.

Overwater work is always subject to a risk of delay because of unsuitable weather. For any given weather condition, the amount of delay depends on the type and size of the installation. In general, the larger the staging or floating craft, the smaller the risk of delay due to weather but, on the other hand, the greater the operating costs. When planning an overwater investigation, it is important that a realistic allowance is made for the possible cost of weather delays. 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 health and safety requirements, navigational warnings, and the regulations of governmental departments and other authorities.

16.2 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 normally 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 or other tower at the borehole location, in which case some means of transporting the boring equipment to the tower is needed. 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. These fulfil the requirement for a fixed working platform and provide manoeuvrability.

The design of all staging, towers and platforms should take into account the capability of the sea-bed strata to withstand the foundation loads. The design should also include the effects of 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, other currents and floating debris.

16.3 Floating craft

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

An auxiliary vessel is 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. Normally, four or six point moorings are required and anchors should have the best holding capacity feasible for the expected ground conditions. In water depths greater than about 80 m conventional moorings are difficult, and the use of vessels that can be maintained in position by computer-controlled thrusting devices, i.e. dynamically positioned, should be considered.

In order to achieve high quality coring it is necessary to maintain constant pressure between the drill bit and the bottom of the hole. For shallow water investigations coring is usually carried out using an hydraulic power swivel suspended in the drilling mast. For the deeper waters, particularly with swells, special techniques of heave compensation are 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.

16.4 Inter-tidal working

Sinking boreholes between high and low tide levels may be achieved using scaffold stagings or platforms (see **16.2**), by using flat-bottomed pontoons or shallow draft jack-up rigs or by moving boring rigs to the location during periods permitted by the tides.

16.5 Setting-out and locating borehole positions

Modern positioning systems, such as the Global Positioning System (GPS) and the Differential Global Positioning System (DGPS) are now commonly used, because they are simple to operate, provide accurate surveying data and eliminate unnecessary risks to personnel in terms of health and safety.

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Close inshore, it is often possible to set out boreholes satisfactorily by using sextant observations to features onshore or by lining in from previously placed shore markers. Where necessary theodolite observations can be taken from land based stations. Further offshore, it is necessary to use electronic methods of position fixing. Electronic methods are also advantageous in bad visibility conditions.

16.6 Determination of reduced level of bed and strata boundaries

Reduced levels are usually transferred to a boring vessel from shore by setting up a tide gauge close inshore; this is read at frequent intervals throughout the tidal cycles and readings of water depth are taken at the same time on the boring vessel. Corrections may be necessary to allow for tidal variations when the tide gauge reading and the one from the vessel vary significantly. Some methods of heave compensation on the drilling vessel automatically make this correction. The depth of water can be difficult to determine where the seabed is very soft and reduced levels of strata boundaries would then become less accurate.

17 Personnel for ground investigation

17.1 Introduction

In view of the importance of ground investigation as a fundamental preliminary to the proper design and the efficient and economical construction of all civil engineering and building works, it is essential that personnel involved in the investigation should have appropriate qualifications, skills and specialist experience and that they should be familiar with the purpose of the investigation. A guide to the qualifications required for the various levels of seniority is given in [7].

17.2 Direction, planning and execution

For most projects, the client appoints a principal technical adviser who has direct responsibility to the client for the design, planning and construction of the project. This adviser is usually a chartered engineer or architect who probably does not have specialist expertise in geotechnics and/or geology.

A geotechnical adviser should be appointed who is directly responsible to the principal technical adviser for the planning, direction, execution and supervision of the ground investigation. It is suggested (see [7]) that the geotechnical adviser should have a first degree in engineering or engineering geology, preferably a postgraduate degree of at least MSc level in a geotechnical topic, should have achieved the qualification of chartered engineer or chartered geologist and have considerable practical experience in geotechnics.

The geotechnical adviser should determine the extent and adequacy of the investigation, 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 to the geotechnical adviser for the execution of the ground investigation may be on the staff of the geotechnical adviser, or may be a geotechnical consultant or contractor. The person should be a suitably qualified geotechnical specialist, geotechnical engineer, or engineering geologist as required by the character and scope of the ground investigation. Details of the required qualifications for each of these proposed designations are given in [7]. As a minimum, all require a first degree, together with some further experience, which would eventually lead to the designation of Geotechnical Adviser (see **17.1**). In particular, the person should have had experience in carrying out ground investigation and in the interpretation of the results.

17.3 Supervision in the field

The supervision of the work in the field should be either the full-time or the part-time responsibility (depending on the size of the investigation) of a suitably qualified and experienced geotechnical engineer or engineering geologist. This person may be assisted by:

a) assistant geotechnical engineers or engineering geologists;

b) field engineers and senior field technicians who are skilled in the work described in **17.7** and who are competent to supervise such work by others, and in addition may be required to supervise work described in **17.8.1** and **17.8.2**;

c) drilling supervisors who are skilled in the work described in **17.8.1** and who are competent to supervise such work by others, and to supervise the work described in **17.8.2**.

17.4 The logging of excavations and boreholes and the describing of soils and rocks

The driller should normally be responsible for recording the information obtained from the borehole as it arises (see **17.8.1** and clause **41**); 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 6) should be made by a suitably qualified and experienced geotechnical engineer or engineering geologist. For 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 form part of the report preparation).

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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 involved, the detailed recording of the soils and rocks, their stratification, structure and fabric should be made by a suitably qualified and experienced geotechnical engineer or engineering geologist.

17.5 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 **36.4**). Laboratory technicians should have received training and have skill and experience in the type of test they are conducting.

17.6 Interpretation

The preparation of the report, including the factual information obtained and the engineering interpretation of it, is described in section 7. This work should be prepared by suitably qualified and experienced geotechnical engineers or engineering geologists and the engineering interpretation should be made by the geotechnical specialist, geotechnical engineer or engineering geologist directing the investigation under the supervision of the geotechnical adviser (see **17.2**). Except where the geology is simple, the geological aspects should be reviewed by a suitably qualified and experienced engineering geologist.

17.7 Field technicians

Field technicians who carry out special sampling and testing in boreholes, piezometer installations and probing (described in section 3) and field tests (described in section 4) should have received training and have skill and experience in the work.

17.8 Operatives

17.8.1 *Driller*

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, making groundwater observations in boreholes, and properly recording the information obtained. Where available they should also have appropriate accreditation.

NOTE In the UK, the British Drilling Association operates an accreditation scheme for drillers.

17.8.2 *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.

18 Review during construction

18.1 Introduction

There is an inherent difficulty in forecasting ground conditions from ground investigations carried out before the works are started because, no matter how intensive the investigation and whatever methods are used, only a small proportion of the ground is examined.

18.2 Purpose

The primary purpose of the review during construction is to determine, in the light of the conditions newly revealed, to what extent conclusions drawn from the ground investigation are required to be revised, if at all. The maximum benefit is obtained when the review is directed by the geotechnical advisor (see **17.2**).

In some cases, additional information is found which may necessitate amendment of the design or the construction procedures. As a result, in certain cases it may 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 purpose of the records is to:

a) check the adequacy of the design;

b) check the safety of the works during construction and to assess the adequacy of temporary works;

c) check the findings of the ground investigation and to provide feedback so that these findings may be reassessed;

d) check initial assumptions about ground conditions related to construction methods, which may include groundwater;

e) provide agreed information about ground conditions in the event of dispute;

f) check the suitability of proposed instrument installations;

g) enable the best use to be made of excavated materials;

h) reassess the initial choice of construction plant and equipment.

18.3 Information required

18.3.1 *Soil and rock*

Accurate engineering descriptions of all strata encountered below ground level should be made in accordance with section 6. 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 geotechnical engineer or engineering geologist competent in geotechnics. It is 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.

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18.3.2 ***Water***

It is most important to record accurately all information about the groundwater obtained during construction, so that this may be compared 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 groundwater, and any unforeseen seepage under or from water-retaining structures. The effect of groundwater lowering should also be recorded in observation holes to determine the extent of the cone of depression.

18.4 Instrumentation (see also **33.2**)

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.

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Section 3. Field investigations

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19 Introduction

There is considerable variety in ground investigations 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 2. The collection and recording of data to be obtained is discussed in section 7. In-situ tests that are carried out in boreholes as part of a borehole investigation are described in clause **25**. 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 4.

In planning ground investigations, particular attention should be paid to the safety of personnel and the public at all stages of the investigation. 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.

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 and 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 **12.5** and **47.2.8**).

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

20 Excavations and boreholes

20.1 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 can temporarily stand unsupported and in suitable conditions. For practical reasons, the maximum depth of excavation is 4 m to 5 m. Where personnel are required to enter pits, it is essential that the sides are safe or made safe, particularly from sudden collapse, by supporting the sides. Ideally, the support system should consist of purpose-made metal frames that can be quickly inserted and extracted. Alternatively, it may be possible to excavate the sides to a safe profile by means of a series of benches [36]. Entry by personnel into unsupported pits deeper than 1.2 m is not allowed for health and safety reasons. The safety aspect is discussed more fully in BS 6031.

By providing access for taking samples and carrying out in-situ tests, shallow trial pits permit the in-situ condition of the ground to be examined in detail both laterally and vertically; they also provide a means of determining the orientation of discontinuities in the ground. The field record should include a plan giving the location and orientation of the pit with details of which face(s) was logged, and a dimensioned section of each side and the floor (see [36]). Whenever possible, the record should include photographs. Further details of reporting are given in **47.2.6.6.** 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 then 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, so 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. There can be advantages though in leaving pits fenced, shored and open (at least overnight and possibly up to 7 days), as this can allow the excavated surface to partially dry, exposing fissures and fabric better than immediately after excavation. Recommendations on backfilling are given in **20.9**. Any pits that have to be left open and unattended should be securely fenced off.

20.2 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. When making inspections, however, 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

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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 a deep shaft is dangerous unless the appropriate safety precautions are strictly followed and these are discussed in BS 8008. 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 **19** and annex F). Oxygen-consuming engines that emit toxic exhaust fumes, such as petrol-driven pump motors, should not be employed in shafts. The safety precautions to be followed and the safety measures to be adopted when human entry into the shaft is envisaged are covered in BS 8008:1996.

20.3 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, as well as 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 major construction problem in strata that are not naturally self-supporting during the period required to erect support elements. Below the water table, it is probable that headings are only an economical means of ground investigation in rock and stiff impermeable clay.

20.4 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 particles. Hand auger boreholes can be used to obtain disturbed samples, small open-tube samples and for groundwater observations.

20.5 Light cable percussion boring

Light cable percussion boring is an adaptation of standard well-boring methods, and normally uses a mobile rig specially designed for ground investigation work. For most investigations, the rig has a winch of 1 tonne to 2 tonne capacity, which 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 cohesive soil in a damp or dry borehole. The shell is used in cohesionless soils and requires there to be sufficient water in the bottom of the borehole to cover the shell (about 2.5 m). It is therefore necessary to add water to a borehole in order to bore through dry cohesionless strata that 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 groundwater 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 such as mechanical augering.

Light cable percussion boring is suitable for soil and weak rock. The sizes of borehole casings and tools are usually 150 mm and 200 mm. For special projects 250 mm and 300 mm are available. This gives a maximum borehole depth of about 60 m in suitable strata. This type of rig may have an hydraulic power take-off to drive a rotary drilling attachment for coring rock. The drill tools, which are 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 usually sufficiently representative to permit identification of the strata.

20.6 Mechanical augers

Mechanical augers for ground investigations normally use a continuous-flight auger with a hollow stem and these 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 may be 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 therefore usually 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.

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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.

20.7 Rotary open hole drilling and rotary core drilling

20.7.1 *Introduction*

Rotary drilling methods, in which the drill bit is rotated on the bottom of the borehole, are used to drill rocks and sometimes soils for investigation purposes. The drilling fluid, which is passed from the surface through hollow drill rods to the face of the bit, cools and lubricates the bit, transports drill cuttings to the ground surface and, when using particular types of drilling fluids, stabilizes the borehole. Drilling fluids are commonly clean water, air, or a mixture of both. In some cases mud, polymers or foam are used to maintain or assist borehole stability, aid the transport of drill cuttings to the surface and maximize core recovery, particularly in superficial deposits and weak rock formations. It is essential that the cleaning and recirculation of the drill fluid is arranged so that the cuttings transported from the bottom of the borehole are not recirculated and that the condition of the drill fluid is maintained to achieve its objectives.

There are two basic types of rotary drilling: open hole (or full hole) drilling, where the drill bit cuts all the material within the diameter of the borehole; and core drilling, where an annular bit, fixed to the bottom of the outer rotating tube of a core barrel, cuts a core, which is recovered within the innermost tube of the core barrel assembly and brought to the surface for examination and testing. Rotary drilling for ground investigation is usually core drilling.

When open hole drilling or coring, temporary casing is normally used to support unstable ground or to seal off fissures or voids, which cause excessive loss of drilling fluid. Drilling fluid additives or cement grouting may sometimes be satisfactory alternatives.

The rotary drilling rig should be well maintained and should be capable both of controlling rotational speed and providing axial load and torque to suit the nature and hardness of the material penetrated, the diameter of the core barrel and drill string, drilling fluid and flushing system, weight of drill string and installation of temporary casing(s).

Drilling is in part an art, and its success is dependent upon good practice and the skill of the driller, particularly when coring partially cemented, fractured, weathered and weak rocks or superficial deposits. Good drillers have had adequate training, together with considerable experience.

20.7.2 *Open hole drilling*

This technique is sometimes used in soils and weak rocks as a rapid and economical means of making holes in the ground for the purpose of taking soil samples, for carrying out in-situ tests, for installing instruments etc. The technique may also be used to probe for voids such as mine workings, solution cavities etc. While drilling, the only samples recovered are the drill cuttings returned in the drill fluid. These constitute very low quality samples and it is usually difficult to detect a change in strata, unless there is a strong contrast in properties such as colour or hardness. Where such contrast prevails, open hole drilling can be used as a probing technique. The use of suitable instrumentation, in order to record the progress of the drilling rig can considerably enhance the results obtained (see [37]).

20.7.3 *Rotary core drilling*

This is normally carried out using conventional or wireline double or triple tube core barrels (a triple tube system may be simulated by a double barrel fitted with a semi-rigid plastic liner) fitted with diamond or tungsten tipped core bits. Specifications of all the major components found in core barrels, rods and casings may be found in BS 4019. The objective of core drilling is to achieve optimum core recovery and core quality consistent with cost. The choice of drill and compatible in-hole and surface equipment, their condition and suitability for the work anticipated, are all most important if the objective is to be achieved. Detailed guidance on the selection of core bits for particular formations or the suitability of various techniques in different types of soil and rock are beyond the scope of this document and advice should be sought from a drilling specialist.

The conventional double tube core barrel consists of two concentric barrels. The outer is rotated by the drill rods and, at its lower end, carries the coring bit. The inner barrel is mounted on a swivel so that it does not rotate during the drilling process. The core, cut by the coring bit, passes up into the inner barrel and, at the end of the coring run, the core barrel assembly is lifted to the surface by raising and removing each drill rod individually. This becomes increasingly time consuming as the borehole becomes deeper. The core is prevented from dropping out of the core barrel by a core catcher made of spring steel and located just above the core bit. The drill fluid flows down through the annulus between the inner and outer barrels on its way to the core bit. The core itself is only in contact with the drill fluid as it passes through the core bit.

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Table 1 — Sizes of rotary core barrels

Ref. on borehole record	Core barrel design	Nominal diameter of core mm	Nominal diameter of hole mm
Sizes given in BS 4019-1:1974			
B	BWF, BWG or BWM	42.0	60.0
N	NFW, NWG, NWM	54.5	76.0
H	HWF or HWG (HWAF)	76.0 (70.5)	99.0
P	PWF	92.0	121.0
S	SWF	112.5	146.0
U	UWF	140.0	175.0
Z	ZWF	165.0	200.0
Miscellaneous sizes			
TBX	TBX (thin wall)	45.0	60.0
TNX	TNX (thin wall)	61.0	76.0
NQ3	NQ3 (triple tube wireline)	45.0	76.0
NQ	NQ (wireline)	47.5	76.0
HQ	HQ (wireline)	61.0	99.0
PQ	PQ (wireline)	83.0	121.0
NMLC	NMLC (triple tube)	52.0	76.0
HMLC	HMLC (triple tube)	63.5	99.0
Mazier		75.0	101.0
		108.0	146.0
Metric sizes			
T2 56	T2 56	42.0	56.0
TT 56	TT 56	45.5	56.0
T6 66	T6 66	47.0	66.0
T2 66	T2 66	52.0	66.0
T6 76	T6 76	57.0	76.0
T2 76	T2 76	672.0	76.0
T6 86	T6 86	67.0	86.0
T2 86	T2 86	72.0	86.0
T6 101	T6 101	79.0	101.0
T2 101	T2 101	84.0	101.0
T6 116	T6 116	93.0	116.0
T6 131	T6 131	108.0	131.0
Geobor S	SK6L	102.0	146.0
T6 146	T6 146	123.0	146.0

KEY
TT is extra thinwall water barrel
T2 is water barrel
T6 is mud/water
SK is wireline barrel air/water/mud

With triple tube core barrels, the non-rotating inner barrel contains a removable sample tube or liner. At the end of each coring run this liner, together with the core it contains, is extracted and stored in a core box. It should be recognized, however, that the provision of such liners, which may be made from a variety of materials and take the form of split tubes, seamless tubes or rigid and semi-rigid plastic liners, does not in itself increase core recovery, but is much more likely to preserve the core in an intact condition, thus enabling the logging geologist to extract more information than would otherwise be the case. Triple tube barrels can be purpose designed, but more commonly conventional double tube barrels can be modified to take an inner semi-rigid plastic liner and thus act as a triple tube core barrel.

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Wireline core barrels are rotated from the surface by drill rods, which are normally of the same diameter as the outer core barrel. The core is brought to the surface within the inner core barrel on a small diameter wire rope or line attached to an "overshot" recovery tool. Larger diameter wireline systems are particularly suitable where superficial and weak deposits are anticipated, as any vibration created from the drilling action on the surface is minimized due to the close fitting nature of the rods within the borehole. The smaller wireline sizes do not show this characteristic and are normally used for hard rock drilling at speed. The borehole wall is constantly supported during the drilling process and when recovering the inner core barrel to the surface, which makes core retrieval quicker and improves production. The main disadvantage with large diameter wireline drilling in weak materials is that it is necessary to remove the string to change the drill bit. In unstable holes not supported by drilling mud or in gravels, collapse of the hole may occur and redrilling of the collapsed material is then necessary.

Some standard core barrel sizes and the core sizes they produce are listed in Table 1. When a double core barrel is fitted with an additional plastic liner to simulate a triple barrel, the core diameters in Table 1 reduce, because different core bits are required to accommodate the liner. In most rocks, satisfactory core is recovered by using double tube swivel type core barrels with a core size of not less than about 75 mm diameter. A reduction in core size to 50 mm diameter might be appropriate in massive rocks and an increase in core size to about 100 mm diameter may give better results in weak, weathered or fractured rock and superficial deposits. The use of larger diameter core barrels and temporary casings usually demands greater axial load, torque, lifting capacity and slower rotational speeds to achieve satisfactory results. In addition, when either open hole drilling or coring with conventional core barrels, an increase in drill rod diameter is required. This is because the size of the annulus needs to be reduced to maintain optimum uphole velocity of the drilling fluid, in order to prevent excessive whip of the drill string and to minimize the risk of borehole instability.

A very wide range of coring bits is available and the type which gives the best results in any given ground conditions may have to be determined by trial. It should be recognized that the factors affecting bit selection for optimum rate of drilling may be different from those that determine 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 normally do not provide for face discharge of the flushing fluid and some disturbance may occur as the fluid passes between the inside of the bit and the core, although this may be obviated by the use of protruding or retractor type core barrels. Stepped bits offer advantages of drilling rates and quality in some circumstances; they also tend to produce straighter holes. For strong abrasive rocks diamond quality and matrix selection are important considerations.

Careful selection of the flushing medium, which is compatible with the ancillary surface and in-hole equipment employed, and which is suitable for the anticipated geological conditions to be drilled, is very important. Water and air are the simplest and most commonly used. Drilling muds consisting of water with clay or bentonite, water with an additive such as sodium chloride, foam, polymer mixtures and air/water mist are also used as flushing media. Polymer mixtures and water with additives have an advantage over water and air in that the cuttings may be removed at a lower flushing velocity, thereby reducing disturbance and washing out at the cutting bit or within the borehole in weak or uncemented formations. Using air as the flushing medium may be more advantageous in some geological formations or where limitations are imposed on the use of other fluids in respect of aquifer or groundwater contamination. Whereas water and polymer may tend to increase the natural moisture content of the core though, the use of air may tend to reduce it. In some formations air may fracture weak formations and create paths of weakness, possibly causing leakage of the air flush and a permanent change to groundwater paths.

20.7.4 ***Core extrusion and preservation***

After the recovery of the core barrel to the surface, every effort should be made in handling to ensure that the recovered core is maintained in a condition as near as possible to its natural state until it is finally stored. It is at this stage that the expense and effort to recover high quality cores can be negated by careless or inappropriate handling. 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. When using a double tube core barrel the core should be extruded with the tube in a horizontal position by using a purpose-made core extruder. Extruders should be of the piston type, preferably mechanically activated, since water or air pressure type extruders can lead to water contact with the core, impulse stressing of the core and uncontrolled explosive ejection of the core from the barrel. It should be noted that in weak, weathered or fractured rocks and superficial deposits extrusion can lead to core disturbance, however carefully it is done.

The core should preferably be extruded in the same direction as it entered the barrel. The core should be extruded into a half-round rigid receiving tray (e.g. plastic guttering) in such a manner that it is completely supported. When it is required to preserve the core such that it does not dry out, a convenient method is to extrude it into thin gauge polythene sheet or tube ("layflat") placed within the half-round tray, which can then be wrapped and

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sealed with plastic tape. Where selected lengths of core are to be preserved at their natural moisture content for laboratory testing, any drilling mud contamination and softened materials should first be removed and the sample should then be wrapped in foil and coated with successive layers of waxed cheese cloth and labelled. The difficulties of extrusion and preservation can be overcome by the use of triple-tube core barrels or double tube core barrels with plastic liners. Split liner tubes are another method of examining the recovered core without further damage after the drilling process. Seamless metal or plastic liners are particularly useful where core is to be removed from site for logging, or where confined undisturbed samples are required for sample observation and subsequent laboratory testing, although care should be taken to ensure that any retained drilling fluid is drained off as quickly as possible.

It is usual to preserve all core obtained from the borehole for the period of the main works contract to which the core drilling relates. This is conveniently achieved with wooden or plastic 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 is minimal movement of the cores when the box is closed [38]. The box should be fitted with stout carrying handles, a hinged lid and strong fasteners and should be designed so that two persons can lift the box when it is full of core. In removing the core from the barrel and placing 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 with the shallowest core to the top left hand corner, the top being considered adjacent to the hinged section. Core retained in the core catcher box shall also be placed in the core box at the correct relative depth. 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 in 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 practicable after recovery of the core and before description, sampling and testing. A graduated scale running parallel to the core axis, for the full length of the core box should be included in photographs. The core should fill at least 50 % of the photograph frame and care should be taken to ensure that the core is in sharp focus with good even lighting (this may well require some form of artificial lighting). When using colour photography, a suitable colour standard indicator should be included in each photograph.

20.8 Wash boring and other methods

20.8.1 *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 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 (see **22.4.5** and **25.2**). Open-tube samples or piston samples can be taken in cohesive strata. This method can be extended in sands and silts to holes up to 150 mm in diameter but its use in gravels is impractical and should be avoided.

20.8.2 *Other methods of boring*

There are many methods of boring, which have been developed mostly to obtain maximum penetration speed, e.g. rotary percussive drilling for blast holes and grouting. When such boreholes are sunk for site investigations, limited information about ground conditions may be obtained, provided that the boreholes are drilled under controlled conditions: rate of penetration should be measured, drilling characteristics should be observed, and samples should be taken of the drilling flushings (see [37] and [39]).

20.9 Backfilling excavations and boreholes

Poorly-compacted backfill causes settlement at the ground surface and can act as a path for groundwater. 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 may be 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 does not necessarily seal a borehole, on account of shrinkage, and it is often preferable to use a cement and bentonite grout, 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. The need and specification for the grout is project dependent.

It is possible to compact the backfill of excavations by means of the excavator bucket or other mechanical means. In special cases, weak concrete or a specified granular fill may be required.

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21 Frequency of sampling and testing in boreholes

21.1 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 covers 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 and the ground water conditions;

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.

21.2 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 soft clay, loose silt, and loose silty sand (see **22.6**).

In coarse non-cohesive soil, such as gravel, it is necessary to take disturbed samples from the drill tools (see **22.3**), together with any split barrel standard penetration test samples at about 1 m intervals (although in gravels it is likely that the solid cone is used) (see **22.4.5**).

In rock, continuous rotary core sampling (see **22.8**) should be undertaken and, if the core recovery is poor, disturbed samples should be taken from the return drill fluid to provide some information about the nature of the material in the sample. 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 rotary coring, which are not required for undisturbed tests, may usefully be split along their longitudinal axis and carefully examined and described first 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 initially to 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.

Ground water conditions are determined from the water levels in the boreholes, from the identification of water bearing strata and from observations in piezometers set at the appropriate depths (see clause **23**).

21.3 The determination of strata 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 **21.5**. 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, either using a light cable percussion boring rig together with the general purpose 100 mm diameter sampler and the standard penetration test equipment; or, alternatively, 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; if the soil is suitable a split barrel sampler should be used.

At the top of each new stratum and thereafter at 1.5 m interval 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.

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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 **20.7**).

21.4 Double-hole sampling

In this method, a borehole is sunk to ascertain the full sequence of strata. A second borehole is sunk adjacent to the first, but at least 1 m from it to avoid the disturbed zone; samples are then 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 change of method at an inappropriate level.

21.5 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 or penetration vane test; thin-walled open or piston sampler; continuous soil sampler; large diameter samplers
hard stony clay	plate bearing test; pressuremeter; rotary core sampling
rock	rotary core drilling using larger core sizes than recommended in **21.3**.

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 is normally determined by similar considerations to those in **21.3**. If, however, 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 stony clay or rock are usually taken continuously.

In the borehole vane test, only the small volume of clay that is rotated by the vane is tested, and individual results can show a considerable scatter. For this reason, vane tests should be carried out as frequently as possible. The vertical interval is 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 not less than four times the borehole diameter is usually adequate.

22 Sampling the ground

22.1 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 is disturbed by the sampling process. It should also be borne in mind that the behaviour of the ground in mass is often dictated by the presence of weaknesses and discontinuities. It is therefore possible to obtain a good sample of material that 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 **14.10**).

There are four main techniques for obtaining samples (see [40]):

a) taking disturbed samples from the drill tools or from excavating equipment in the course of boring or excavation (see **22.3**);

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 **22.4** to **22.7**);

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 **22.8**);

d) taking block samples specially cut by hand from a trial pit, shaft or heading (see **22.9**).

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Samples obtained by techniques b), c) and d) are often of sufficient quality 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 exhibit some degree of disturbance. A method for classifying the quality of the sample is given in **22.2**, Table 2. Clauses **22.3** to **22.9** 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.

When taking samples for chemical testing and in particular, on potentially contaminated sites, additional care is needed to avoid cross-contamination and chemical or biological reactions, which may affect the result. The risks of cross-contamination are reduced by:

1) using dry drilling or air flush methods for progressing the boreholes;

2) using casing to isolate upper layers of soil and groundwater;

3) ensuring that all sampling and boring equipment are clean;

4) implementing strict sample handling protocols.

A detailed consideration of investigations on potentially contaminated sites is given in annex F. Any sample of ground that might be contaminated by substances hazardous to health should have a warning to that effect on the sample label so that personnel can follow appropriate safety procedures (see **36.1**).

22.2 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.
Table 2 shows a classification for soil samples developed in Germany. A useful basis for classifying samples in terms of quality is provided in [41].

Table 2 — Quality classification for soil samples

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)

In general, Classes 1, 2 and 3 are achieved with tube samplers, rotary coring or block samples [see **22.1**b), c) and d)]. Such are often known as intact samples. Samples from the drill tools [see **22.1**a)] are class 4 or 5. 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 5.

Whichever sampling methods are used, it is sometimes only 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.

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” [42]. 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 that have been specially oriented. 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 because the results of laboratory tests may be more representative of the ground mass. In special cases, samples of 150 mm and 250 mm in diameter or block samples of larger size are used [42].

Precise details of the mass of sample required for each test specimen can be found in the relevant parts of BS 1377:1990 and BS 1924, with more general guidance in BS 1377-1:1990, Table 5. 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 3 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 812-1.

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Table 3 — Mass of soil sample required for various laboratory tests

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

22.3 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 normally 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 3. 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.

22.4 Open-tube samples

22.4.1 *Principles of design*

22.4.1.1 *General*

Open-tube samplers consist essentially of a tube that is open and made sharp 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, which has a single sample tube and simple cutting shoe. The use of sockets and core catcher is discussed in **22.4.4**. 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 when forced into the ground. Three features of the design control the degree of disturbance: the cutting shoe, the inside wall friction and the non-return valve. The sampling procedure (see **22.4.2**), is also an important factor in controlling sample disturbance.

22.4.1.2 *The cutting shoe*

The cutting shoe should normally be of a design similar to that shown in Figure 1 and 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 this is typically 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.

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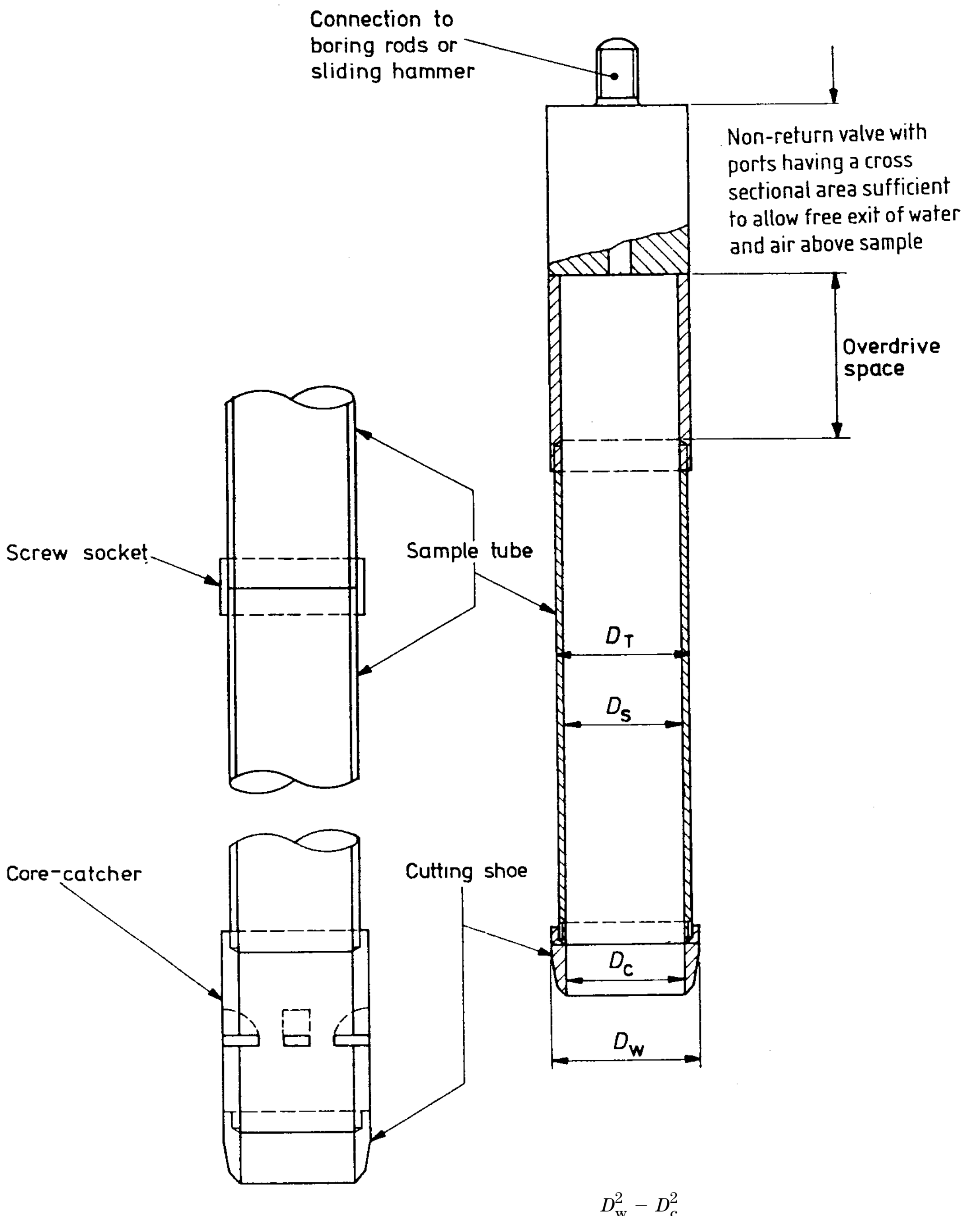

$$\text{Area ratio} = \frac{D_w^2 - D_c^2}{D_c^2} \times 100\ \%$$

Figure 1 — Basic details of open-tube sampler

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 samplers 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 that has a long outside taper and is considerably less than that which would be expected from the calculated area ratio (see Figure 2).

22.4.1.3 *Wall friction*

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

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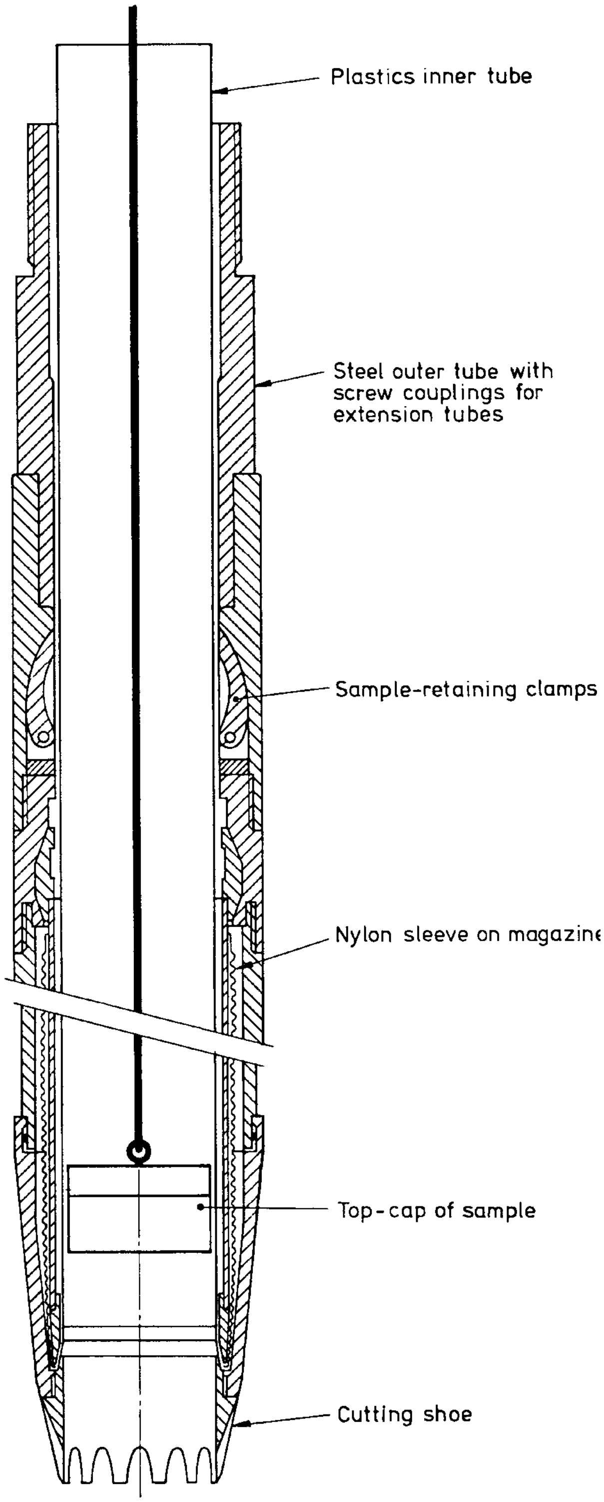

Figure 2 — Delft continuous sampler — 66 mm

22.4.1.4 *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.

22.4.2 ***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 normally passes into the overdrive space of the sampler.

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 reduce this effect, it is necessary to keep the level of the borehole water above the groundwater 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 an 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 because, if driven too far, the soil is compressed in the sampler. A sampling head with an "overdrive" space (see Figure 1) allows 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.

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22.4.3 ***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 or without a small inside clearance. The area ratio should be about 10 % or less. They are pushed into the soil by continuous static thrust from hydraulic jacks or pulley block and tackle. These samplers are usually only suitable for fine soils up to a firm to stiff consistency, and free from large particles, although samples have been successfully obtained from very stiff soils. They normally give class 1 samples of all fine cohesive soils, including sensitive clays, provided that sinking the borehole has not disturbed the soil. Samples between 75 mm and 100 mm in diameter are usually obtained and samples up to 250 mm in diameter are sometimes obtained for special purposes. It is to be noted that disturbance at the base of the borehole occurs 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 **22.5**). A typical thin-walled sampler is illustrated in Figure 3.

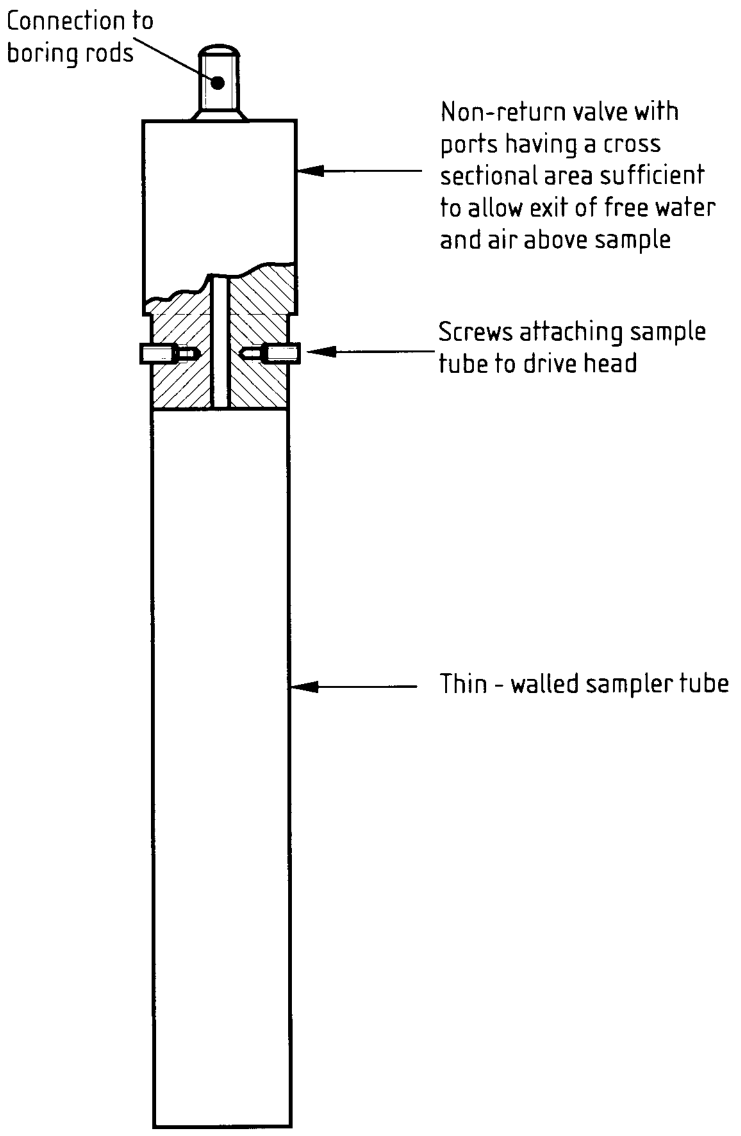

Figure 3 — A typical thin-walled sampler

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22.4.4 ***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 is usually driven into the ground using a sliding hammer or drop weight. It 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 standard open tube sampler (see Figure 1) may sometimes give class 1 samples but, more often, class 2 samples. In sensitive clay, class 2 samples may be obtained. In brittle or closely fissured materials, such as certain weak rocks and hard clays, and also in stony materials, the sampler gives class 3 samples; this is because sampling disturbs these materials, reducing the 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 rigid steel or alloy sample barrel, about 450 mm in length, with a screw-on cutting shoe and drive head. The area ratio should not normally exceed 30 %. The use of a liner system increases the area ratio above 30 % reduces the sample qualities referred to above by one or more classes. Sample barrels can be coupled together with screw sockets to form a longer sampler. Two or three standard barrels coupled together are often used for sampling soft clays, 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 the ground. Sample recovery can be improved by inserting a core catcher between the cutting edge and the sample barrel, as shown in Figure 1.

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.

22.4.5 ***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 1377:1990. 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 samples (see also **25.2**).

22.5 Stationary piston sampler

22.5.1 ***Thin-walled stationary piston sampler***

The thin-walled stationary piston sampler consists of a thin-walled sharpened 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 level, 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 4 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. Typically samples up to 1 m long can be taken.

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 pushed or pulled down by a static thrust until the drive head encounters the upper face of the piston. An automatic clamp in the drive head prevents the piston from dropping down and extruding the sample while the sampler is withdrawn. The application of air under slight pressure via an air line fastened to the outside of the sample tube can relieve suctions at the base of the sample tube during withdrawal. Hydraulically operated piston samplers are also available.

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 **22.4.3**. Although normally used in soft clays, special piston samplers have been designed for use in stiff clays [42].

22.5.2 ***Other stationary piston samplers***

Variations on the piston sampling method are also available, for example, as used with cone penetrometer systems. These samples are pushed into the ground with a cone locked in front of the sample cutting tube. At the desired depth a fishing tool is lowered down the tube string. By pulling the fishing tool the cone and piston are unlocked and held stationary. The sample cutting mouth with the rest of the assembly goes further into the soil and so the sample is taken. The sample is packed in a nylon stocking that is connected to the piston. The stocking acts as a friction reducer between the sample and the sample tube. Samples can be taken from any depth within the capacity of the penetrometer unit used. Typically samples are 1 m long and 35 mm to 65 mm in diameter.

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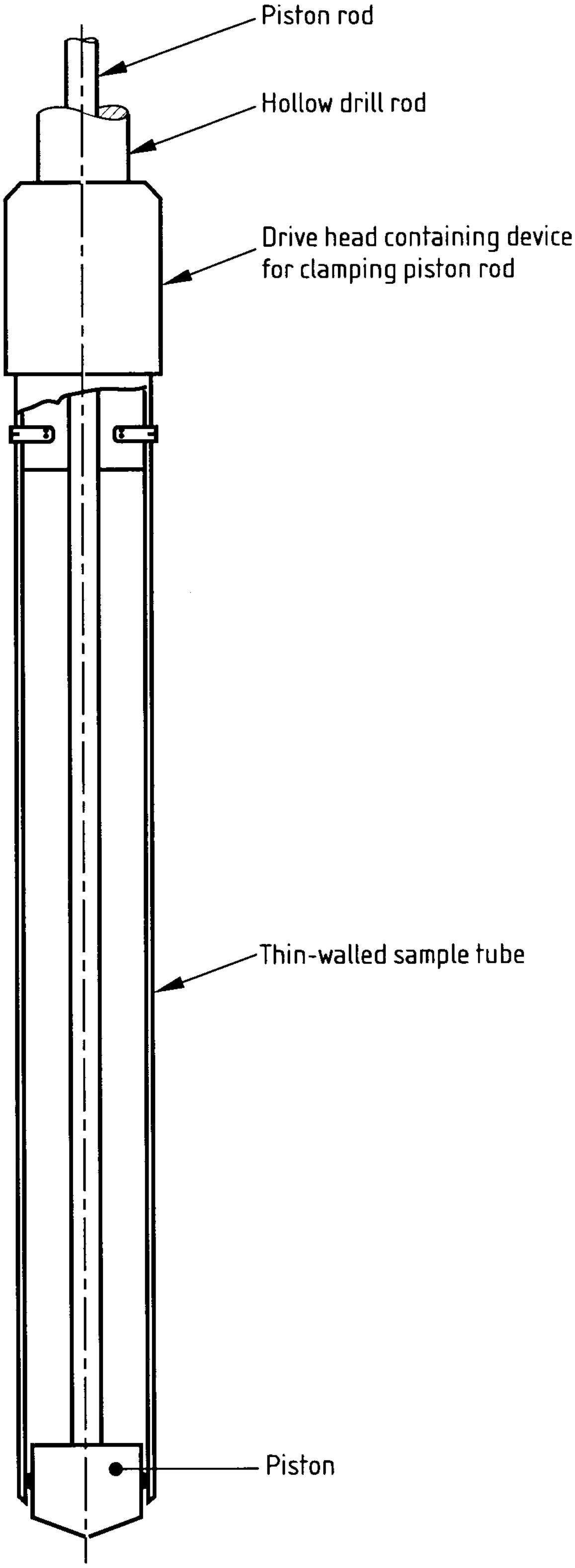

Figure 4 — Basic details of piston sampler

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22.6 Continuous soil sampling

22.6.1 ***General***

Continuous soil sampling can produce samples up to 30 m in length in soils such as fine alluvial deposits. This is of particular value for identifying the soil fabric [42] and gives results superior to those that can be obtained by consecutive drive sampling. The Swedish system [43] 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 sample, is described more fully in **22.6.2**.

22.6.2 ***The Delft continuous sampler***

The Delft continuous sampler, developed by Delft Geotechnics [44] is available in two sizes to take continuous samples 29 mm and 66 mm in diameter. The 66 mm sampler shown in Figure 2 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 plastic inner tube filled to the appropriate level with a supporting fluid of bentonite: and baryte 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 about 18 m, but in suitable strata, with a modified magazine and increased thrust, samples up to 30 m in length have been 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 plastic 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.

22.6.3 ***Semi-continuous samplers***

Alternative systems exist for taking semi-continuous samples. Sampling systems have been developed to recover consecutive soil samples, typically 66 mm in diameter and 750 mm in length. The sampling tube with cutting shoe is lowered to the ground and a plastic liner inserted. The assembly is pushed hydraulically into the ground for a distance of 750 mm under controlled conditions, for example with a cone penetrometer rig. The liner tube with sample can then be recovered using a wireline tool. The sample is labelled and stored. Another liner is then dropped inside the sampling tube and latched into the end section. The procedure is repeated until the required depth is achieved or the limiting capacity of the equipment is reached.

22.7 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 [45] enables the sample to be removed from the ground into an air chamber and then lifted to the surface without coming into contact with the water in the borehole. The sampler is normally 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 no better than class 3. If static thrust is used, however, class 2 and sometimes class 1 samples can usually be recovered.

An open tube sampler fitted with a core catcher usually recovers intact samples of a cohesionless sand. A useful sampler can be made using components of the general purpose 100 mm diameter sampler (see **22.4.4**). Three sample barrels are coupled together and fitted with a core catcher and cutting shoe (see Figure 1). The sample in the central barrel is less disturbed than the material in the outer barrels and is normally quality class 3.

The 66 mm size Delft continuous sampler (see **22.6.2**) recovers quality class 2 samples of cohesionless sand.

22.8 Rotary core samplers

Samples are obtained by the core drilling procedures described in **20.7**. The quality of sample may vary considerably depending on the character of the ground and the type and size of coring equipment used (see BS 4019-1).

22.9 Window sampler

The window sampler is best suited to dry cohesive soils. In ideal conditions, it is possible to take a sequence of samples down to a maximum depth of about 8 m. [46].

The system uses sample tubes, normally 1 m long, which can be screw-coupled together or coupled to extension rods and fitted with a screw-on cutting shoe suitable for hard driving. The sample tubes and accessories are available in a range of diameters, a typical suite being 80 mm, 60 mm, 50 mm and 36 mm.

The sample tubes are driven into the ground by percussion, using a hand- or rig-operated hammer, and extracted manually, either using levers or the rig's hydraulics. Longitudinal slots or "windows" are cut in the wall on one side of each sample tube so that the retained sample can be examined and specimens taken for laboratory testing.

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In use, the first or uppermost sample is taken using the largest available sample barrels coupled together to accommodate the longest sample that can be recovered (typically between 1 m and 2 m). After extraction of the first sampler, the process continues by driving a reduced size sampler down the open hole left by the extraction of the first sampler. Repeating this process with successive reductions in the size of the sample tubes, a useful depth of investigation can frequently be achieved.

The system produces samples of quality class 3 or 4 and is often used for investigations in contaminated ground.

A variant of the window sampler is the "flow through sampler" in which soil passes through and out of the side of the sampler during driving. A small sample is only retained from the base of the drive. By repeat driving to a selection of depths, samples can be obtained down a profile.

22.10 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 oriented 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 **22.11.6.**

22.11 Handling and labelling of samples

22.11.1 *General*

Samples may 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, so it is important to establish a satisfactory procedure for the handling and labelling of the samples, as well as their storage and transport both to prevent their deterioration, and to ensure that they can be readily identified and drawn from the sample store when required.

The samples should be protected from frost, which would damage them, and from excessive heat and temperature variation, which could lead to deterioration in the sealing of the sample containers and subsequently damage the samples. The temperature of the sample store is 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.

22.11.2 *Labelling*

All samples should be labelled with a unique reference number immediately after being taken from a borehole or excavation. If they are to be preserved with their natural moisture content, they should be sealed in an airtight container or coated in wax at the same time. The label should show all necessary information about the sample. If the sample is of ground which might be contaminated and contain hazardous substances, then the label should carry a warning to that effect. The sample is normally recorded on the daily field report. It should carry more than one label or other means of identification so that the sample can still be identified if one label is damaged. The label should be marked with indelible ink and be sufficiently robust to withstand the effect of its environment and the transport of the sample. An additional record copy of the sample should also be kept separately.

22.11.3 *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 1 litre capacity, which the sample should fill with the minimum of air space. The container should have an airtight 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.

The sample containers should be numbered, and the tear-off slip or label, as described in **22.11.2**, 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 when 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 **22.11.2**.

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22.11.4 ***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 **22.4.5**). 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 of 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. There should be a close-fitting lid or screw-cap on each end of the tube or liner. If necessary, the lids should be held in position with adhesive tape.

b) Immediately after being removed from the sampling tool, samples that are not retained in a tube should be wholly covered with several layers of molten paraffin wax, preferably microcrystalline wax, and these should then 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.

Following the guidelines in **22.11.2**, a label bearing the number of the sample 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 minimizes damage by vibration and shock during transit. For certain materials, testing in a site laboratory would be preferable.

22.11.5 ***Rotary core samples***

The handling and labelling of core samples taken by rotary drilling is an essential part of the rotary core drilling technique and is described in **20.7.4**. The samples should be kept in a sample store as described in **22.11.1**.

22.11.6 ***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; it may be advisable to reinforce these with layers of porous fabric (e.g. muslin) or plastic film. Additional labels should be fixed to the outside of samples. The sample should be packed in a suitable material and placed in a strong box or crate. Large samples should be protected with tight-fitting formwork or packed in rigid cement, wax or resin to prevent fissures from opening up under the weight of the samples.

23 Groundwater

23.1 General

The determination of groundwater pressures is of the utmost importance, because these have a profound influence on the behaviour of the ground during and after the construction of engineering works. Various strata, particularly those separated by relatively impermeable layers, can have different groundwater 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 groundwater. To measure groundwater pressures accurately, it is usually necessary to install special measuring devices called piezometers. The groundwater 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 so that such variations may be investigated. When designing drainage works, it is often helpful 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 **25.4**; packer or Lugeon tests are described in **25.5,** and large scale pumping tests are described in clause **27**.

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23.2 Methods of determining groundwater pressures

23.2.1 ***Response time***

All the methods described in **23.2** require some flow of water into or out of the measuring system before the recorded pressure can reach equilibrium with the actual groundwater pressure. For an excavation or a borehole a large volume of water may flow before the water level reaches equilibrium with the groundwater pressure. On the other hand, some types of piezometer require only a very small change in the volume of water for the groundwater pressure to be read. The rate at which water flows through the soil depends on the permeability. The time required for a measuring system to indicate the true groundwater pressure is known as the response time and depends both on the quantity of water required to enter the system (including all pipes and tubes) to operate the pressure measuring device, and on the permeability of the ground. The selection of a suitable method for measuring the groundwater pressure is largely determined by the response time [47] [48].

23.2.2 ***Observations in boreholes and excavation***

The simplest method of determining the groundwater 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 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 groundwater pressure. Such methods should, however, be treated with great caution, because frequently insufficient 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, because these are affected by the use of temporary borehole casings. For most conditions it is preferable to use a standpipe piezometer.

23.2.3 ***Standpipe piezometers***

The standpipe piezometer is a device consisting either 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 normally 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 [48] shown in Figure 5. A small amount of anti-freeze added to the water in the piezometer tubing prevents freezing in cold weather.

Special standpipe piezometers can also be installed by jacking to depth in suitable ground conditions (see **23.2.7**).

The main advantages of a standpipe are that it is simple and it can be used to determine the permeability of the ground in which the tip is embedded (see **24.4**). The disadvantage, however, is the length of time taken to reach equilibrium or to respond to changes in porewater pressure in soils of relatively low permeability.

23.2.4 ***Hydraulic piezometers***

In hydraulic piezometers, the groundwater pressure is detected in a small piezometer tip with porous walls and conducted through small-bore plastic tubes to a remote point where the pressure is measured, usually with a pressure transducer and electrical readout, or more simply with a mercury manometer or Bourdon gauge. Air in the tubes causes erroneous readings, and because of this the tubes have to be kept full of water.

Various types of hydraulic piezometer are available, the most common being the twin-tube piezometer [48] shown in Figure 5. 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 at working pressure. For trouble-free operation, pressures throughout the connecting tubes should be above atmospheric pressure, so the tubes and measuring point should 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 usually has a small response time and can be used for measuring pressure changes due either to changes of stress induced by superimposed loads or excavations, or to tidal variation, although in the latter case it is essential that the system response time be checked [47, 49]. It can also be used for in-situ measurement of permeability. In strata with high permeability, care should be taken to see that the limiting permeability of the porous tip is considered.

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All dimensions are in millimetres

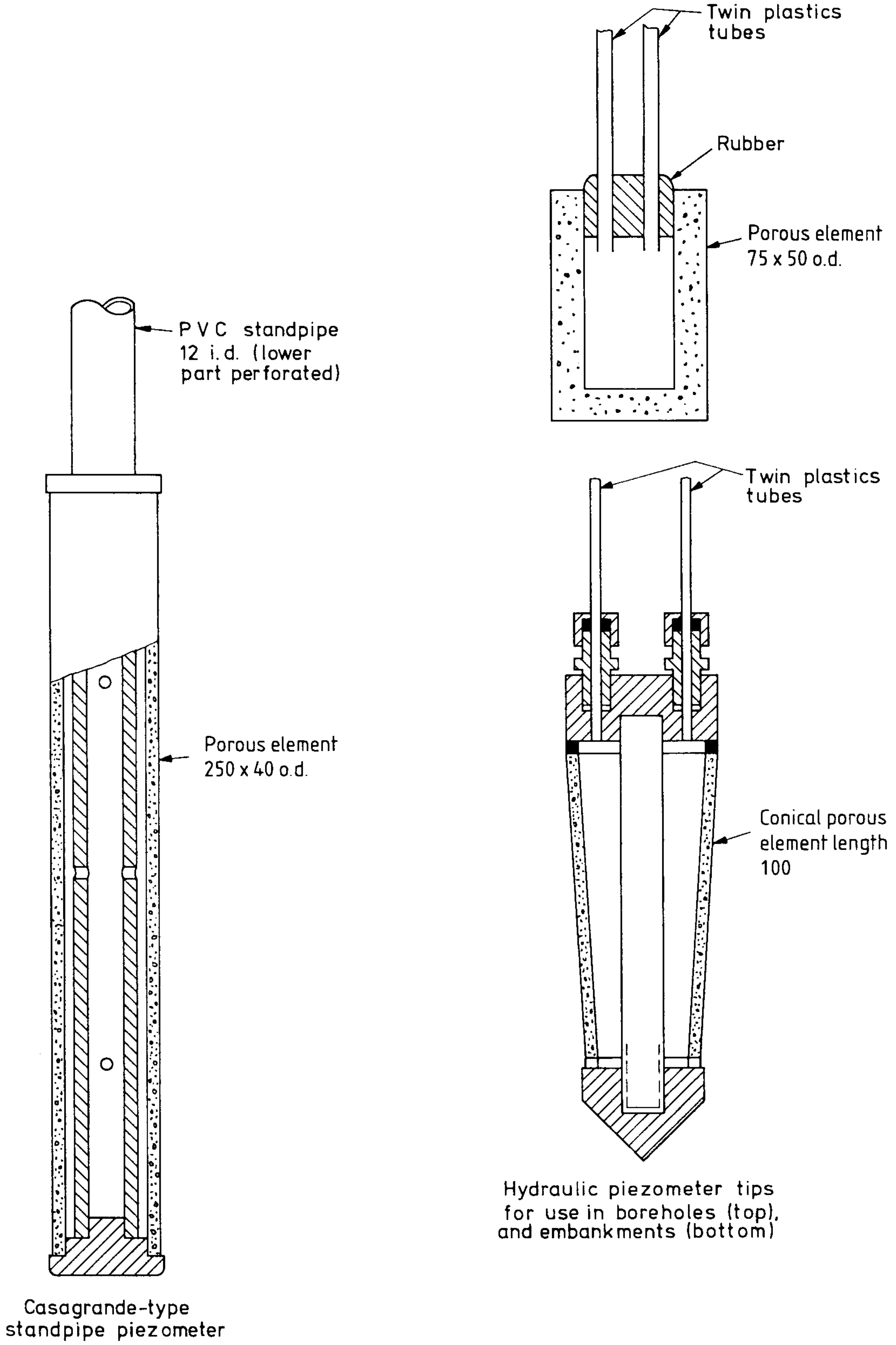

Figure 5 — Typical piezometers

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23.2.5 ***Electrical piezometers***

Electrical piezometers have a pressure transducer located close to the porous element. Provided the tip is de-aired, very rapid response times can be achieved which makes this method particularly suitable for dynamic measurements. 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 electrical piezometer has various disadvantages, however, the main one being that it requires calibration, which cannot easily be checked after installation; some transducers have temperature-sensitive elements, so check calibrations need to be at groundwater temperature and it is therefore not always easy to check that the instrument is behaving reliably. De-airing is not usually possible after installation and misleading results can be obtained, particularly in soils containing gas, e.g. methane in organic soils [50]. The electrical piezometer cannot be used for in-situ permeability measurements [48].

23.2.6 ***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 the pressure constant 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. Dirt entering the lines can also 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 [50]. In general, pneumatic piezometers have the same limitations as electrical piezometers in that they cannot be easily checked and the porous tips cannot be de-aired after installation.

23.2.7 ***Installation of piezometers***

The success of porewater 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 with a removable sleeve, which covers the element during driving [51]. 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 and the porous element is 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 and to have a grout pipe loosely tied to the standpipe or leads just above the tip. A sand and water mixture is pumped down the grout pipe until the piezometer tip is covered with sand to a depth of about 300 mm. The sand is then sealed either with bentonite balls or pellets. The grout pipe is then pulled up by about 400 mm and the borehole sealed off, preferably filled, by pumping down a suitable cement and bentonite grout. Care should be taken to avoid impregnating the sand with the grout. One way of avoiding this is to seal off the borehole 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 installation of the piezometer in sand in this way can significantly increase the response time of the system.

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The composition of the grout mix depends 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 groundwater levels. The grout should be easily pumped, flexible and of the required permeability. 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.

23.2.8 ***Varying groundwater pressures***

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

23.3 Groundwater samples

Care should be taken to ensure that the samples are representative of the water-bearing stratum and have not been contaminated or diluted by water entering the borehole from other strata, or by contact with any water or drilling fluid used to advance the borehole. The depth and method of sampling, as well as the subsequent storage and handling of samples, may influence the results of analyses undertaken on groundwater samples.

Sample containers should be made from an inert material, be clean and completely filled with the water sample so as to minimize contact with oxygen. The samples should be stored in the dark, at low temperatures and tested as soon as possible after sampling.

When groundwater samples are to be taken from a stratum that has been contacted while advancing the borehole, all water-bearing strata from higher levels should first be sealed off by borehole casings. As far as possible, all the water in the borehole should be removed by baling or pumping and the sample taken from water that collects by seepage. About one litre should be collected in a clean polyethylene, polypropylene or glass bottle, which should be rinsed three times with the water being sampled before filling. More stringent requirements may apply in certain circumstances, particularly when accurate or extensive chemical testing is to be undertaken in order to investigate possible chemical contamination (see annex F). Additional requirements may include special sampling techniques, multiple samples in different sample containers with different fixing agents, duplicate sampling, and special sample handling procedures.

Some investigations require the use of permanent monitoring wells from which groundwater samples can be taken at various times. The monitoring wells are constructed and installed in a similar way to standpipe piezometers (**23.2.3** and **23.2.7**), although slotted piping with externally wrapped filter screens are commonly used instead of porous elements. A typical design would be a 50 mm internal diameter standpipe of inert material, set in a borehole of approximately 150 mm. The slotted section of the standpipe is surrounded by a filter fabric and set in a sand or gravel backfill over the depth range from which the water sample is required. Above the sand or gravel pack is a seal of grout. Before taking a sample, it is essential that the well be "purged", i.e. the water standing in the well is removed by baling or pumping and groundwater allowed to flow in until the water in the standpipe is representative of the groundwater. The sample is then taken using a baler type sampler or a suitable pump. Further information is given in annex F.

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Section 4. Field tests

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24 Introduction

Field tests are often desirable where it is considered that the mass characteristics of the ground would differ appreciably from the material characteristics determined by laboratory testing. These differences normally arise from several factors, the most important of which are the extent to which the laboratory samples are representative of the mass, and the quality of sample that can be obtained for laboratory testing. Factors affecting sample quality are dealt with in clause **14** and attention is drawn to factors affecting the representative nature of a laboratory sample. These factors are partly related to the in-situ conditions of stress, pore pressure and degree of saturation, and can be altered from an unknown in-situ stage by the sampling processes. Consequently, their influence cannot be accounted for in laboratory testing.

The material tested in-situ by a field test is analogous to a laboratory sample, and can be considered as a "field sample". The in-situ conditions of a field sample may be affected by the process of gaining access to the position, i.e. digging a trial pit, but usually the effect is very much less than for a laboratory sample.

More obvious, however, are the controlling effects of the nature, orientation, persistence and spacing of discontinuities [32]; the nature of any filling; and the size of sample required for it to be representative. The selection and preparation of samples in the field is subject to the same requirements as for laboratory samples, to ensure that they are representative. Considerable attention should be given in the field to these aspects, because normally fewer field tests can be carried out than laboratory tests and it may not be possible to visually examine the field sample.

The size of sample tested in a field test depends on the nature of the ground and type of test, and may vary from a fraction of a metre, such as in-situ triaxial state of stress measurements, to several metres for field trials, to one or two kilometres in the pumping test.

Field tests may therefore be necessary where the preparation of representative laboratory samples is complicated by one or more of the following conditions:

a) The spacing of the discontinuities in the mass being considered is such that a sample representing the mass, including the discontinuities, would be too large for laboratory test equipment. The discontinuities are assumed to govern the geomechanical response of the material on the scale of the structure concerned.

b) It is difficult to obtain samples of adequate quality because of the lack of cohesion or irreversible changes in mechanical properties; these result from changes in the pore pressure, the degree of saturation and stress environments during sampling, and from physical disturbance resulting from the sampling procedure.

c) It is difficult to determine the in-situ conditions, such as pore water pressure, degree of saturation, and stress environments for reproduction in the laboratory testing.

d) Sample disturbance due to delays and transportation from remote sites is excessive.

e) The zone of interest may be inaccessible to sampling equipment.

Many field tests are expensive and should not be undertaken before obtaining a comprehensive understanding of the geology and nature of the ground. Few standard field tests have been evolved. Most tests have to be specially designed to take account of the nature of the works and the character of the ground in the mass based on the findings of the initial ground investigation. Precision and careful observation is required in all field tests; use of continuous recording equipment, e.g. to note small changes during the test, can increase the quality of the data obtained.

The full-scale interaction of ground with the works can be obtained by monitoring during construction and on completion.

Field tests described in this section 4 usually exclude the tests in boreholes described in section 3.

The locations and levels of all field tests should be recorded as described in clause **19**.

25 Tests in boreholes

25.1 General

The various forms of test that are commonly conducted as complementary or supplementary to a ground investigation carried out by borings are described in **25.2** to **25.7**. Inevitably, there is some overlap with section 4. For example, pumping tests are described in clause **27** because in that instance the borings are incidental to the test. In the same way, vertical loading tests may be carried out near the surface or at the bottom of a borehole, so the main details of these tests are described in **31.1** and **25.5** covers only those aspects particular to boreholes. Clause **25** and section 4 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.

25.2 Standard penetration test

25.2.1 *General principles*

The standard penetration test is a dynamic penetration test carried out using a standard procedure, which is described in BS 1377-9:1990, **3.3**. The test uses a thick-walled sample tube, the outside diameter of which is 50 mm. This is driven into the ground at the bottom of the borehole by blows from a standard weight falling through a standard

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distance. The blow count gives an indication of the density of the ground. The small sample that is recovered will have suffered some disturbance but can normally be used for identification purposes (see also **22.4.5**). When the test is being performed in gravel of 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 BS 1377-9:1990, **3.3**, since even minor variations from the specified procedure can seriously affect the results [52]. 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 **25.2.3**). 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 find out the consistency of other soils (silts and clays) and of weak rocks (e.g. chalk).

The test description in BS 1377-9:1990, **3.3** is in close agreement with the international reference test procedure of the ISSMFE (International Society of Soil Mechanics and Foundation Engineering) [53].

It is essential that the procedures of BS 1377-9:1990, **3.3** be adhered to, because the preparation prior to carrying out the test is crucially important and can significantly affect the results.

25.2.2 ***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 approximate, but may 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 to check 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 **25.6** and **31.1**. When the test is carried out in granular soils below groundwater 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 as to 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 static or dynamic probing described in **26.2** and **26.3**.

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 so 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.

25.2.3 ***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 the subject of conventional foundations and piles. Unfortunately, a lack of enforced and consistent standardization for the drilling technique and SPT tests equipment outside the UK can mean that SPT results and soil parameters derived from data from other countries may not correlate with results from SPTs derived in accordance with BS 1377-9. A comprehensive review of the relevant literature is given in [52].

25.3 Vane test

25.3.1 ***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 BS 1377-9:1990, **4.4**. 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 after which the vane test is 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 kPa. The results are questionable in stronger clays or if the soil tends to dilate on shearing or is fissured. Results are unreliable in materials with significant coarse silt or sand content. The borehole should be kept topped up with water in soft, layered or sensitive soils.

It is to be noted, however, that the undrained shear strength determined by an in-situ vane test is normally 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 found to vary with the plasticity of the clay and other factors [54] [55].

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25.3.2 ***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, because higher shear strengths tend to result from the in-situ vane test than from laboratory tests on samples obtained with the general purpose sampler described in **22.4.4**. If the test is carried out in soil that is not uniform and contains 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 BS 1377-9:1990, **4.4**, 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.

25.4 Permeability

25.4.1 ***General principles***

Intrinsic permeability is usually measured with respect to air and is independent of the fluid. Permeability, as used in this code of practice, is strictly the hydraulic conductivity, a measure of the rate of water flow.

Before carrying out any tests, it is important to identify the aquifer and understand whether it is confined or unconfined. 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 rate of 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 (constant head test) or it may be allowed to vary (a variable head test). The technique is applicable only to the measurement of permeability of soils below groundwater level. The measurement of the permeability of unsaturated or partially saturated zones is extremely difficult [56]. A variety of tests is available, ranging from the simple, which can nevertheless be used to investigate complex situations, to the sophisticated; the interpretation of the data is crucial. It is important to establish the normal fluctuations within the aquifer.

25.4.2 ***Preparation for carrying out the test***

The field procedures are well described by Hvorslev [57], who also shows the response time of each type of installation and the range of permeabilities over which they work. In the simplest form of test, preparation consists of cleaning out the bottom of the borehole, but cleaning out the borehole before conducting a test is not enough to obtain a reliable result, even if appropriately sized filter material is introduced in an unlined extension of the cased hole. In any field borehole, it is extremely difficult to be assured of completely clean water in the bore, because some suspended matter is always left and this settles into a sediment under static conditions. Fines of only a few grains thickness can form a filter cake with enhanced resistance to a falling head or outflow test, leading to errors of an order or more in assessment of permeability. If sediment has accumulated a rising head or inflow test might be similarly affected. The test would then have to be 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 can cause misleading results but the problem may be overcome by the use of fibre rings. The lower the permeability of the soil, the greater are the errors caused by leakage.

For more accurate measurements, either a centralized axial length of perforated tube or a suitable piezometer tip is installed within a granular filter pocket. The filter pocket prevents erosion of the ground when the casing is withdrawn to expose the filter to the soil. It is essential that the top of the filter be sealed against leakage to the borehole. Recommendations for sealing above granular filters are given in **23.2.7**. When a ceramic piezometer tip is used it should be saturated with de-aired water before installation.

This avoids errors in flow measurement due to the pressure of entrapped air in the leads. Clean water for the test is fed directly through an enclosed pipe to the perorated pipe or piezometer tip from which it flows through the filter to the soil, so borehole water that may be contaminated with fines is not involved in the test.

It is essential that the filter material used is properly designed and has a permeability between 10 to 100 times greater than that of the soil being tested.

Whether using the procedure for a variable head test (**25.4.3**) or a constant head test (**25.4.4**), it is important to restrict the head difference on inflow or outflow tests to avoid displacing the soil (hydraulic fracturing) and encouraging leakage along the outside of the casing. The maximum head difference should be less than the effective lateral or vertical earth pressure at the depth of the test. It is suggested that a safety factor of two should be applied to whichever earth pressure is the smaller.

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An alternative empirical rule is that the maximum head difference should not exceed 10 % of the equilibrium piezometric head.

When small heads are used to drive flow, accurate monitoring equipment is required to ascertain both flow rate and head difference.

The finer grained the soil, the lesser the test flow rate and the greater the consequence of unwanted leakage via a flow path not included in the mathematical model considered. When the hydraulic resistance of the leakage path is less than that through the filter or filtercake, it may represent the dominant but unsuspected part of measured flow.

Modern electronic pressure and flow transducers with data logging can give accurate values of head differences, even during rapidly falling or rising heads. Rotating float types of flow meter can also be calibrated for extreme accuracy over a range of flow rates. Continuous recording of the pressures in the test section is preferred and should be undertaken by the use of pressure transducers connected to data loggers.

For constant head tests it is important to obtain the value of head difference for the test by reference to a separate hole intercepting the stratum in which the measurement is to be made, at a distance greater than 50 times the test hole diameter. Measurements of water levels for determining the head difference should be made simultaneously in two boreholes. Successive tests can then conveniently be made in each bore using the other as reference.

If it is not practicable to employ a reference hole, the natural groundwater level should be determined accurately by one of the methods described in clause **23**.

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 at equilibrium before the test is performed, although with clays of low permeability it can take several months for the pore pressures set by the drilling of the borehole to equalize. Even so, "early" test results are often more reliable as clogging of the soil face occurs very quickly.

25.4.3 ***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.

25.4.4 ***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, though constant negative head tests may be carried out; these can yield more reliable results because the effects of clogging are reduced. It is essential to use clean water. It is not possible to achieve a constant head if the groundwater 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 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 time and in such cases the method suggested by Gibson [40] may be used, in which the actual rate of flow is measured and recorded at intervals from the commencement of the test. Unfortunately, clogging of the test section often makes the achievement of a constant head difficult, so when this happens accurate results from the early stages of the test should be used in the same way.

25.4.5 ***Analysis of results***

There are numerous published formulae for calculating permeability from these tests, many of them partly empirical. Those given by Hvorslev [57], which are reproduced in outline in **25.4.6**, 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 [58] for the constant head test is also indicated. This gives a more accurate result with compressible soils.

25.4.6 ***Formulae for borehole permeability tests***

25.4.6.1 *Method 1 (after Hvorslev [57])*

a) Constant head test

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

b) Variable head test

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

or

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

where

- k is the permeability of soil;
- q is the rate of flow;
- F is the intake factor (see Figures 6 and 7);
- 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 measurement 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 8).

NOTE These formulae assume that the natural groundwater level remains constant throughout the test. For the case where the natural groundwater varies, see [57].

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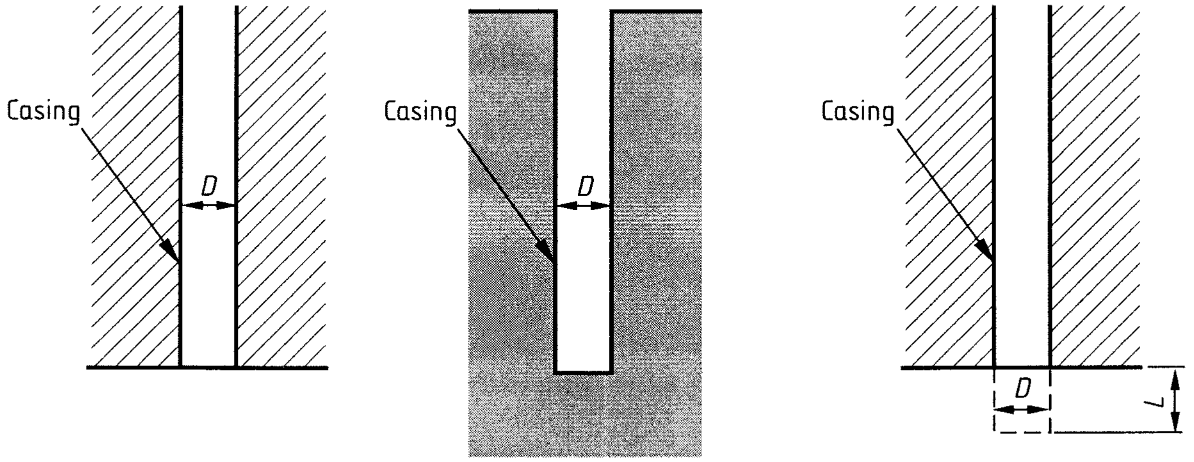
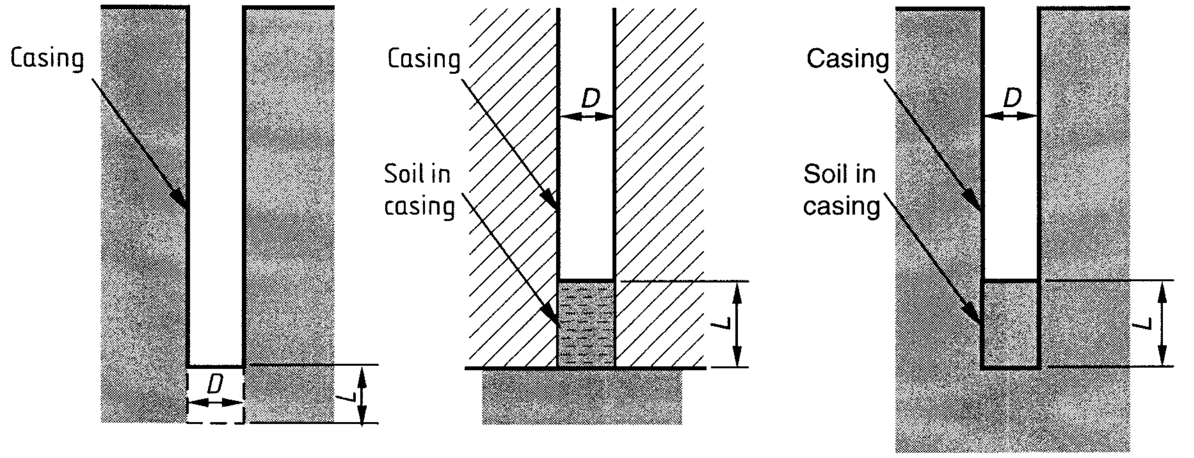


$F = 2D$ $F = 2.75D$ $F = \dfrac{2\pi L}{\log_e[(2L/D) + \sqrt{1 + (2L/D)^2}]}$

a) Soil flush with bottom at impervious boundary

b) Soil flush with bottom in uniform soil

c) Well point or hole extended at impervious boundary



$F = \dfrac{2\pi L}{\log_e[(L/D) + \sqrt{1 + (L/D)^2}]}$ $F = \dfrac{2D}{1 + (8/\pi)(L/D)}$ $F = \dfrac{2.75D}{1 + (11/\pi)(L/D)}$

d) Well point or hole extended in uniform soil

e) Soil in casing with bottom at impervious boundary

f) Soil in casing with bottom in uniform soil

NOTE 1 Expressions come from Hvorslev [57].

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

NOTE 3 Case d) may be used for tests carried out using piezometer tips, but more accurate results are obtained by using Figure 7, 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 [57].

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 tend to measure the former.

Figure 6 — Values of intake factors, *F*, in borehole permeability tests

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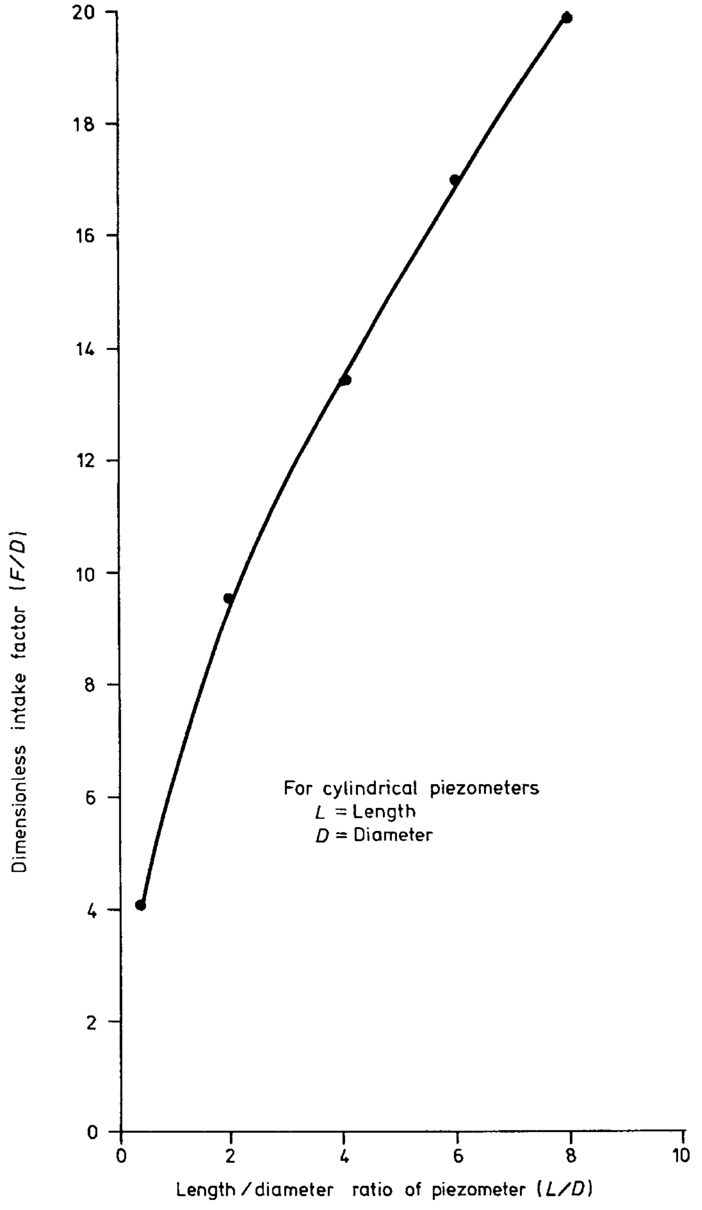

NOTE 1 Graph comes from Al-Dahir and Morgenstern [59].

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

NOTE 3 Where *L* is large compared with *D*, the test tends 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 measures the former, whatever the relation between *L* and *D*.

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

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

Figure 7 — Relationship between dimensionless intake factor *F/D* and length to diameter ratio of piezometer *L/D*

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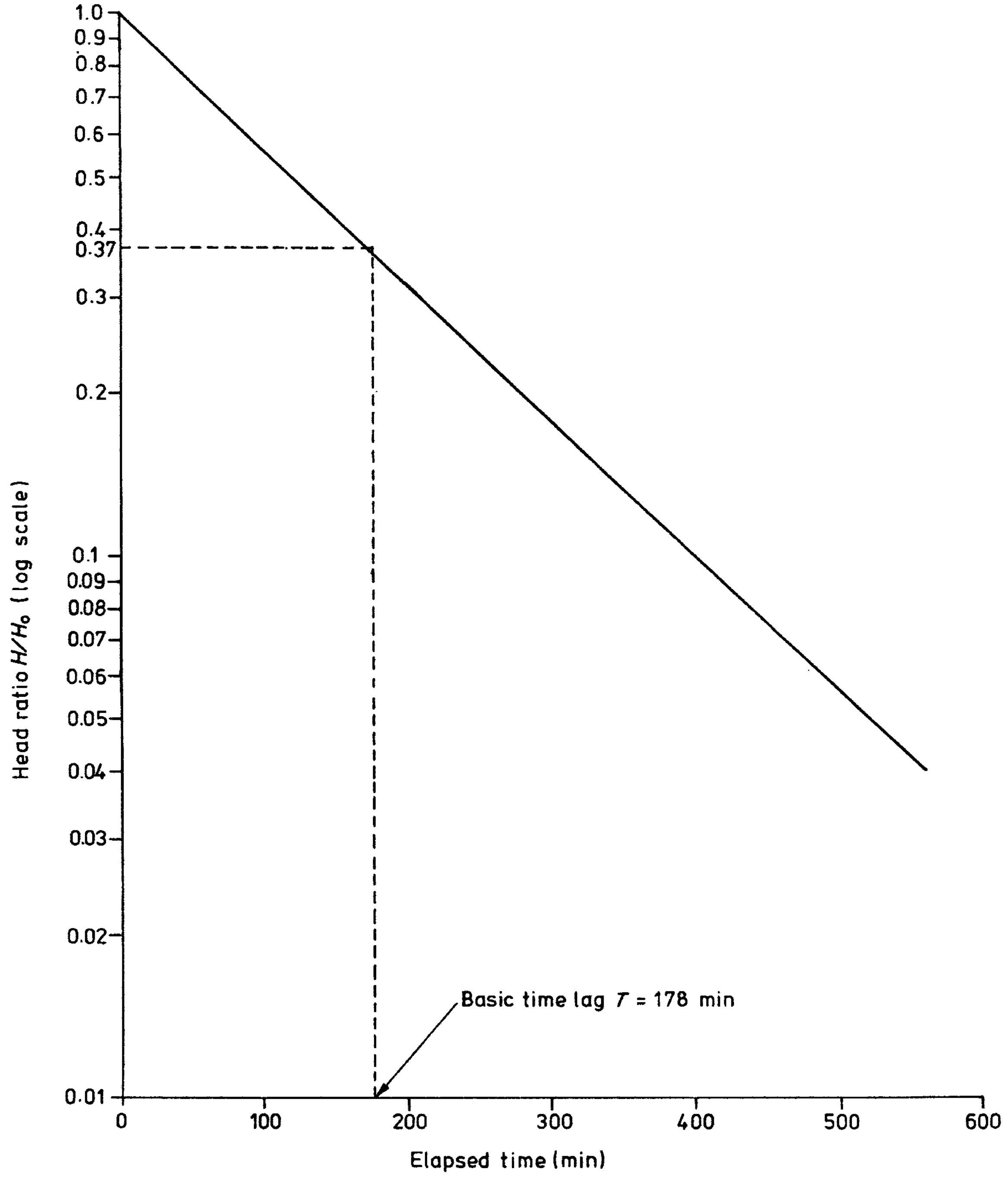

NOTE 1 H_o is the head at the commencement of test. H is the head at any time t, which has elapsed since the test began.

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

NOTE 3 The ratio H/H_o 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_o of 0.37.

Figure 8 — Falling or rising head test. Example of calculation of basic time lag [57]

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25.4.6.2 *Method 2*

Constant head test after Gibson [58].

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

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

where

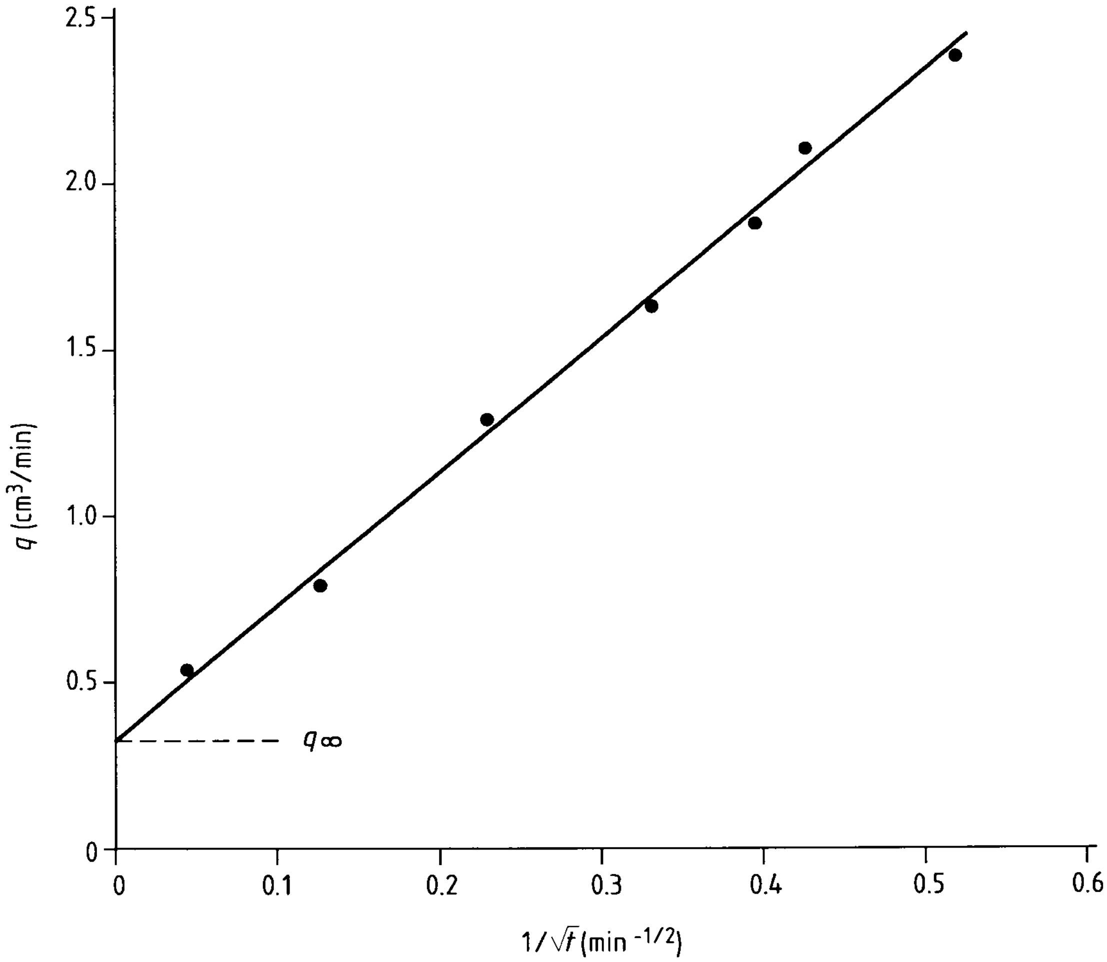
- k is the permeability of soil;
- C is the coefficient of consolidation or swelling;
- q_∞ is the steady state of flow (read off the q, $1/\sqrt{t}$ graph as illustrated in Figure 9);
- F is the intake factor (see Figures 6 and 7);
- H_c is the constant head;
- r is the radius of a sphere equal in surface area to that of the cylindrical tip;
- n is the slope of the q, $1/\sqrt{t}$ graph.

NOTE 1 Heads refer to the natural groundwater level before the test.

NOTE 2 This method makes theoretical allowance for the compressibility of the soil and also permits the coefficient of consolidation or swelling to be calculated, but this effect is often markedly less than the accuracy of the data.

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 become a straight line. The line can then be extrapolated to give q_∞ and n, where the test would otherwise take too long.

An alternative testing regime is to carry out a short constant rate test and then to stop the flow and carry out a short variable head test; the use of a water level/log time plot should allow identification of mechanisms operating in the aquifer.



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 9 — Constant head test analysis by Gibson's method [58]

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25.4.7 ***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 (although still modest) 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 is 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 may give more accurate results than variable head tests; but on the other hand variable head tests are simpler to perform, allow a number of different methods of analysis and can indicate other effects. The water pressure used in the test should be less than that which disrupts the ground by hydraulic fracturing. It has been shown that serious errors may be introduced if excessive water pressures are used [61] (see **25.4.2**). With soils of high permeability, i.e. 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, probably yields more accurate results; these are described in clause **29**. 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^{-7} 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 (see Hvorslev [57]). 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 in soils is influenced by the effective stress and the stress history. This should be considered in the test regime and in the analysis of results. 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, but this is usually very difficult to observe.

Execution of the borehole permeability test requires expertise, and small faults in technique can 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. In many types of ground, however, 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. Considerable care is needed in interpreting the test data. Where a reliable result is required, the programme of borehole permeability tests is normally followed by a full-scale pumping test.

25.5 Packer test

25.5.1 ***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 [62] to provide a standard for measuring the impermeability of grouted ground; it is also widely used as a packer test to measure the permeability of dam foundations [63]. 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 accepts, to check the effectiveness of grouting, to obtain a measure of the amount of fracturing of rock [64], 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 groundwater level of 100 m, a 1 m length of borehole accepts 1 l/min of water. Lugeon did not specify the diameter of the borehole, which is usually assumed to be 76 mm (see Table 1), 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.

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

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where

k is the permeability in metres per second (m/s);

Q is the rate of injection in cubic metres per second (m^3/s);

H is the pressure head of water in the test section in metres (m);

L is the length of the test section in metres (m);

r is the radius of the test section in metres (m).

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 [65].

25.5.2 ***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 compressed air is fed down the borehole through a small diameter nylon tube. The inflation pressure should be that required to just inflate the packer to the required diameter, to seat the packer and to overcome the hydrostatic pressure in the borehole. Excessive pressures should be avoided. 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 prove 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.

25.5.3 Application and measurement of pressure

It is essential that the maximum water pressure to be applied is not sufficient to cause uplift of the ground or to break the seal of the packers in deep holes in weak rock. The pressure to be determined for use in the calculation of permeability is that causing flow into the rock itself. The pressure can be measured directly with an electric pressure gauge set in the test zone or indirectly by gauges at ground level. The use of direct pressure measurement is preferred and pressures should be measured at the bottom and preferably below the test section. This avoids the difficulties associated with corrections for fluid density, friction losses etc. in indirect measurements. If it is necessary to take indirect readings at ground level, these are 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 metres (m);

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

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

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

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

Consideration should be given to the quality of water used in the test, especially when carrying out tests in the vicinity of aquifers.

It is also possible to carry out "pump out" packer tests, where a section of borehole is isolated using a similar packer arrangement as in conventional packer tests, but where water is removed from the test section using a small submersible pump. Pressures are measured with transducers.

25.5.4 ***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 fitting 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.

25.5.5 ***Execution of test***

Developing or cleaning the borehole before testing is vital. The test may be carried out either as a single or as a double packer test. Appropriate measurement devices should be included to allow detection of leakage past the packers; this is assisted by continuous logging of the readings. However, the single packer test is normally 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 boreholes. 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.

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Because of the limitation on the pressure referred to in **25.5.3**, it may not be practicable to run the test at the (Lugeon) specified head of 100 m above groundwater level. The assumption is made that the water flow is proportional to the pressure, although this is not necessarily true [66]. It is then possible to obtain the Lugeon value by extrapolation. For hydrogeological purposes, test pressures of less than about 5 m head are usually adequate. It is customary to run a series of tests at different pressures. Typically a series of five tests is desirable, with the maximum pressure applied in three or five equal increments and then reduced with decrements of the same amount. The full data record obtained from these measurements is particularly useful in assisting in the interpretation of the behaviour of the rock under test [67].

It is advisable to an appropriate suite of geophysical logs in boreholes in which packer or other permeability testing has been carried out. These are not unduly expensive and often greatly enhance the value of the test results (see clause **35**).

25.6 Plate tests

25.6.1 *General*

The plate test is one particular application of the vertical loading test (see clause **31.1**), and the procedures for the test are described in BS 1377-9:1990, **4.1**. Only the specific problems for 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 paid to the safety of operators working below ground (see **20.2**). 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, of course, limits the value of the results. Care should be taken to ensure that the plate movements are measured directly and that the effects of compression of the load column are eliminated. The strain distribution beneath the plate can be measured using an extensometer array [68].

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 4. The diameter of the plate used should, so far as practicable, be equal of that of the borehole, provided that care is taken to eliminate cohesion or friction on the side of the plate. 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.

25.6.2 *Limitations*

The general limitations of the vertical load test are discussed in **31.1.2** and they apply similarly to the borehole test. Additionally, in the bottom of a borehole that is too small to allow hand preparation of the test surface, it is more difficult to achieve a satisfactory bedding of the loading plate on the test surface, and therefore values obtained for the deformation parameters may be of limited significance.

Where necessary, casing should be used to support the sides of the borehole and to seal off water seepage 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 seepage, which disturbs the ground and adversely affects its deformation characteristics. It would then be necessary to resort to external dewatering. If the test is undertaken only for measuring the strength parameters, disturbance due to groundwater 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.

In 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 is lowered down the hole and lightly pressed on to the surface of the mortar. Plaster and resins can also be used for bedding.

25.6.3 *Uses of the test*

25.6.3.1 *Large diameter boreholes*

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

25.6.3.2 *Small diameter boreholes*

The deformation characteristics obtained can be of dubious value owing to doubts about the elimination of ground disturbance and errors resulting from unsatisfactory bedding of the plate, although satisfactory unload/reload parameters can be obtained. 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 gravelly clays and weak rocks. The plate diameter should be large in relation to the structure of the ground.

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25.6.4 *Supplementary test*

Although not strictly a plate test, a test is sometimes done 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 either a layer of compressible material or a suitably designed collapsible container is placed. 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 BS 1377-9:1990. 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.

25.7 Pressuremeter tests

25.7.1 *General*

A pressuremeter is defined as a cylindrical device with a flexible membrane, which imposes a uniform pressure on a borehole wall [69]. The applied pressure and the resulting deformation of the membrane are measured and enable in-situ stress, stiffness and strength of the ground to be investigated.

There are two approaches to the use of pressuremeters in site investigation. The first is based on the method developed by Menard [70] in which the pressuremeter is used to obtain design parameters directly. The second is to analyse pressuremeter tests to give properties of the ground.

25.7.2 *Pressuremeters*

Several types of pressuremeter have been developed. They fall into three categories: those that are lowered into prebored holes, those that are drilled in and those that are pushed in. They are normally between 40 mm and 100 mm in diameter and up to 1 m long, with the expanding section being between 5 and 6 times the diameter of the instrument. The displacement capacity of the instrument is a function of the instrument design but normally exceeds 10 % of the instrument diameter. There are three main pressure capacities: 0 MPa to 4 MPa for soils; 0 MPa to 10 MPa for weak rocks; and 0 MPa to 20 MPa for moderately strong rocks.

25.7.2.1 *Prebored pressuremeters*

Prebored pressuremeters are lowered into pockets drilled specifically for the tests. The first pressuremeter to be widely used was the Menard pressuremeter [70]. It consists of three expanding cells connected to the surface by drill rods and flexible hoses. The central cell is inflated with water under pressure and readings of the pressure and volume change are taken at the ground surface. The other cells, normally inflated by gas pressure, ensure a condition of plane strain around the central cell.

The other types of prebored pressuremeters [71] are mainly single expanding cell systems in which the radial or diametrical displacement is measured directly with transducers mounted in the instrument. The membrane is inflated with gas or oil under pressure, which is then measured with a transducer mounted in the instrument.

25.7.2.2 *Self-bored pressuremeters*

Self-bored pressuremeters are single cell instruments attached to a drilling head [72] [73] so that they can be drilled into the ground. The head contains a drill cutter, which is turned by inner rods that pass through outer rods. The outer rods connect the instrument to the surface and are used to push the probe into the ground. Mud or water is pumped down the inner rotating rods and back up the annulus between the inner and outer rods. This removes the parings from the cutter and retains pressure on the soil. Radial displacement of the membrane is measured directly with transducers mounted within the instrument. The membrane is inflated with gas or oil under pressure, which is then measured with a transducer mounted in the instrument.

25.7.2.3 *Pushed-in pressuremeters*

The push-in pressuremeter is a single cell pressuremeter most commonly mounted behind an electric cone penetrometer [73]. The membrane is usually inflated by oil or water under pressure, which is measured ideally by a transducer mounted in the instrument. Displacement of the membrane is either measured directly with transducers mounted within the instrument, recording radial displacement or by volume change measurements.

25.7.3 *Calibrations*

There are three groups of calibrations: transducer or line calibrations; membrane stiffness; and membrane compression [71] [74]. Line calibrations apply to volume change systems such as the Menard pressuremeter. Transducer calibrations are necessary for all those pressuremeters that contain displacement or pressure transducers within the probe. Membrane stiffness is the pressure required to inflate the membrane in air and is normally required for all instruments. Membrane compression is the change in thickness of the membrane under pressure. It is most applicable for tests at high pressures in rocks using instruments containing displacements transducers.

25.7.4 *Installation*

Pressuremeters are either placed in prebored holes, self-bored or pushed in. It is essential that preboring and self-boring be designed to minimize the disturbance to the surrounding ground. Pushing in is intended to produce the same amount of disturbance each time. The minimum spacing between tests is 1 m.

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25.7.4.1 *Prebored pressuremeters*

The probe is inserted in a prebored pocket at least 1.5 m long, drilled in such a manner that disturbance to the borehole walls due to erosion softening are minimized. The hole diameter should not normally be greater than 110 % of the probe diameter. The probe should be installed in the hole as soon as possible after drilling.

25.7.4.2 *Self-bored pressuremeters*

The probe is drilled either from the surface or from the base of a hole a sufficient distance, usually 1 m, to ensure that the test section is beyond the zone of influence of the base of the borehole. The probe can be drilled with a specially designed rig or by a rotary rig using a special adapter. The purpose of self-boring is to reduce the disturbance to the borehole walls to a minimum.

25.7.4.3 *Pushed-in pressuremeters*

The probe is pushed to the required depth. When the probe is mounted behind a cone penetrometer this is done using a cone penetration rig and at the same rate (2 cm/s) as that specified for a cone [73].

25.7.5 *Test procedures*

Tests can be either stress- or strain-controlled. In a test the pressure is increased in increments and the displacement of the membrane is recorded. In stress-controlled tests there are at least fifteen increments of pressure during the loading phase, each increment being held for 1 to 2 min. A strain-controlled test is also under stress control but a feedback system is used to ensure that the displacement of the membrane satisfies a preset strain rate [74].

The Menard method is based on a stress-controlled test [70]. The objective is that method specific parameters are obtained that can be used directly in design formulae developed from observations of full scale tests.

It is common practice to carry out at least one unload reload cycle within a test from which a value of stiffness may be obtained. In general two or three unload reload cycles may be preferable.

25.7.6 ***Presentation of results***

It is usual to present the data from a pressuremeter test as an applied pressure against cavity strain curve. This curve is interpreted to give parameters that are relevant to the particular instrument in use and the ground conditions.

25.7.7 ***Uses and limitations of the test***

Pressuremeters can be used in most ground conditions, but not all pressuremeters can be used in any one ground condition. It should be noted that the results from any individual test depend on the equipment used and the procedures adopted for installation, testing and interpretation. This means that results from different tests may not necessarily be compatible. Prebored pressuremeters can be used in most soils and rocks. The results obtained depend on the quality of the pre-drilling. The Cambridge type self-boring pressuremeters can be used in most clays and sands provided they do not contain excessive amounts of gravel size particles It may be necessary with the self-boring pressuremeter to use a second rig to clear obstructions encountered during drilling. The weak rock self-boring pressuremeter can be used in dense sands, hard clays and weak rocks. This instrument has to be used with a rotary rig. The cone pressuremeter can be used in those soils into which it is possible to push a cone.

The Menard method is an empirical design method based on pressuremeter tests. It is essential that the instrument, installation procedure, test procedure and interpretation be followed if these design rules are to be used [70].

Theories of cavity expansion are well documented [71]. The interpretation of a test is based on these theories but the parameters obtained may be empirical. Estimates of in-situ stress can be obtained from self-bored pressuremeter tests [71] and in some cases prebored tests [75].

Average cavity stiffness can be obtained from unload-reload cycles from all pressuremeter tests [69]. Recent developments in interpretation show that the average cavity stiffness can be converted to a material stiffness [76]. The values of stiffness vary with the strain and stress level over which they are carried out. The reliability of interpretations of deformation modulus therefore require the influence of strain magnitude on the modulus to be assessed and in soils where drainage occurs during a test the influence of changes in effective stress on the deformation modulus. Undrained shear strength can be obtained directly from self-boring pressuremeter tests and estimated from other pressuremeter tests [71] [73] [77]. Angle of friction of sands can be determined from self-bored pressuremeter tests [78] and estimated from pushed in pressuremeter tests [79].

Developments in the instruments and their operation as well as in the analysis and interpretation are still taking place.

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26 Probing and penetration testing

26.1 General

Probing from the surface probably represents the oldest method of investigating the depth to a hard stratum where the overburden is weak and not 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 bearing stratum within 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; and the other where the probe is forced into the soil by a static load.

26.2 Dynamic probing

26.2.1 *General principles*

The apparatus for dynamic probing comprises a sectional rod with a cone fitted at the base of a slightly greater diameter than the rod. It is driven into the ground by a constant mass that is allowed to fall on 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. This is usually achieved using a mechanical latch on machine-driven equipment, and mechanical indication on hand-operated apparatus.

The method of working for dynamic probing is given in BS 1377-9:1990, **3.2**.

Table 4 gives specifications for a range of configurations, including those from BS 1377 and the International Reference Test Procedure of the ISSMFE [53], though other configurations may be more appropriate in some circumstances [80]. The fact that the rod is slightly smaller in diameter than the base of the cone to some extent prevents shaft friction influencing the results. In some soils where the hole closes in on the rods this factor has to be taken into account. One method is to correlate the torque measurements, which have been taken as part of the test before each new extension rod is added, with the blow count and subtract this from the measured blow count [80]. Shaft friction can be eliminated or substantially reduced by boring a hole to the depth required for each test, or by providing the exterior of the rods with either a sleeve or a lubricating mud injected behind the cone. The size of the cone, mass of hammer and the distance through which the constant mass is allowed to fall have only recently been standardized in the UK, so there is only limited practical experience behind the test, although some published data are available linking the results of certain tests with soil parameters determined by other methods [80].

26.2.2 *Uses and limitations*

The main uses of dynamic probing are for preliminary investigations of a site using hand-operated equipment, followed by machine-operated equipment during the main investigations, thereby allowing the interpolation of data between boreholes using site specific correlations with known ground property data. 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. The main limitation of dynamic probing is that the soil being tested cannot be identified, although sampling techniques using the machine operated equipment are being developed. Owing to the limited practical experience so far in the UK, correlation between dynamic probing test results and soil properties are only just being developed [80] [81].

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Table 4 — Details of dynamic probing test specifications

Factor	Unit	Test specification				
		DPL	DPM15	DPM	DPH	DPSH
Hammer mass (M)	kg	10 ± 0.1	30 ± 0.3	30 ± 0.3	50 ± 0.5	63.5 ± 0.5
Height of fall (h)	m	0.5 ± 0.01	0.5 ± 0.01	0.5 ± 0.01	0.5 ± 0.01	0.75 ± 0.02
Mass of anvil and guide rod (max.)	kg	6	18	18	18	30
Rebound (max.)	%	50	50	50	50	50
Rod length,	m	1 ± 0.1 %	1 ± 0.1 %	1 ± 0.1 %	1 ± 0.1 %	1 ± 0.1 %
Mass of rod (max.)	kg	3	6	6	6	8
Rod eccentricity (max.)	mm	0.2	0.2	0.2	0.2	0.2
Rod OD	mm	22 ± 0.2	32 ± 0.2	32 ± 0.2	32 ± 0.2	32 ± 0.3
Rod ID	mm	6 ± 0.2	9 ± 0.2	9 ± 0.2	9 ± 0.2	9 ± 0.2
Cone apex angle	degrees	90	90	90	90	90
Cone area (A) (nominal)	cm^2	10	15	10	15	20
Cone diameter (new)	mm	35.7 ± 0.3	43.7 ± 0.3	35.7 ± 0.3	43.7 ± 0.3	50.5 ± 0.5
Cone diameter (worn, min.)	mm	34	42	34	42	49
Mantle length	mm	35.7 ± 1	43.7 ± 1	35.7 ± 1	43.7 ± 1	50.5 ± 2
Number of blows per “x” cm penetration		N_{10}:10	N_{10}:10	N_{10}:10	N_{10}:10	N_{20}:20
Standard range of blows		3 to 50	3 to 50	3 to 50	3 to 50	5 to 100
Specific work per blow ($M{\cdot}g{\cdot}h/A$)[a]	kJ/m	50	98	150	167	238

NOTE 1 DPL: Light; DPM: Medium; DPH: Heavy; DPSH: Super Heavy; DPM15: a hybrid to give better range of specific work per blow.

NOTE 2 DPL, DPM, DPH, and DPSH are all in ISSMFE [53]; DPH and DPSH are in BS 1377-9:1990.

[a] M, h, A are as defined above. g is acceleration due to gravity.

26.3 Static cone penetration test or static probing

26.3.1 *General*

Several types of static probing equipment have been developed and are in use throughout the world. The basic principle is that a cylindrical probe, fitted to the lower end of a string of hollow rods, is pushed into the ground at a slow uniform rate by a static thrust. The probe has a cone at its base, which is fitted with a sensor, so that its resistance to penetration can be measured. If required, probes can also incorporate a friction sleeve, by which the local frictional resistance can be measured, and also a piezometer for measuring the pore water pressure in the vicinity of the cone and sleeve [82] [83].

The most frequently used probe has electrical sensors, which can permit continuous recording throughout the test. This type of probe is included in the specification for the test given in BS 1377-9:1990, **3.1**.

The older type of mechanical penetrometer measures the cone and friction resistance by means of a system of internal rods, which thrust against an hydraulic load capsule set at ground surface. Mechanical penetrometers are occasionally used in very isolated sites, where the more sophisticated electrical read-out systems are not readily applicable, and for doing preliminary probing to assess whether the ground conditions are suitable for the use of the much more expensive electrical probe (the probe can experience severe damage when penetrating some types of grounds (**26.3.5**).

26.3.2 *Standard test*

Full details of the equipment and procedures are given in the BS 1377-9:1990. The salient details of the standard test are:

— diameter of cone, probe and rods 35.7 mm;
— area of cone, probe and rods 1 000 mm^2;
— apex angle of cone 60°;
— peripheral area of friction sleeve 15 000 mm^2;
— rate of penetration 20 mm/s.

Larger diameter, but non-standard, equipment having a cross-section area of 1 500 mm^2 is also in common use [84]. The advantage of this equipment is that it is more robust and can be less liable to damage in difficult ground conditions.

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26.3.3 ***Electrical static cone penetrometer***

A parallel-sided probe, fitted with a friction sleeve is referred to in BS 1377:1990. The electrical sensors, for measuring cone and friction resistance, normally incorporate resistance strain gauges mounted within the body of the probe. When pore water pressure is to be measured, the probe is also fitted with a piezometer intake and pressure transducer; this is often described as a piezocone.

At present, there is no standard for the location of the piezometer intake for piezocones. Two alternative positions are normally used:

a) on the shoulder of the cone, just behind the cone;

b) on the face of the cone.

The first of these, on the shoulder, is given as the preferred location in the International Reference Test Procedure [85]. It allows the measured values of cone resistance to be corrected for errors induced by pore water pressures acting on the surfaces of the penetrometer. These vary with the geometry of the penetrometer.

The pore water pressure measurements can vary substantially, depending on the location of the intake. The response appears to depend on the material being tested and its overconsolidation ratio. The optimum location depends on the soils to be investigated and the purpose of the investigation [82] [83]. On the front face the element is less likely to desaturate, however it is more vulnerable to damage and pore pressure corrections cannot be made directly. With the element on the shoulder, pore pressure corrections can be directly carried out for the subsequent determination of soil parameters, although the element is more vulnerable to desaturation if it passes through dense or stiff layers. In order to obtain the correct responses, it is essential that care be taken to ensure complete saturation of the filter element and pressure measuring system with a suitable fluid. When the piezocone is used, regular checks should be made to verify saturation of the element, and the preliminary results should be inspected to detect signs of premature desaturation. The risks of desaturation can be minimized by the use of pre-drilled water-filled holes through the layers of partially saturated soils near the surface.

In use, the probe is advanced at a uniform rate of penetration by thrust on the sounding rods and the electric signals from the various measuring devices are normally carried by a cable threaded through the penetrometer rods to the surface. Other systems are available, which either transmit the data to the surface using acoustics, or store the test data in the probe so that it can be downloaded when the probe is recovered at the end of each test. Data from the test is displayed for immediate assessment, recorded automatically at selected intervals on a chart or print-out, and then stored on disc or tape for later processing. Sole dependence on data that is recorded electrically, which cannot be assessed during or immediately after test completion, is not recommended.

Penetrometer tests can be deflected offline by some ground conditions, leading to significant errors in the reporting of vertical depths. It is recommended that whenever possible and certainly where the test exceeds about 15 m depth, the probe should be fitted with an inclinometer so that the location of the probe can be tracked and true vertical depths computed.

Load sensors, pressure transducers and other instrumentation should be of suitable capacity and sensitivity for the ground conditions likely to be encountered. They should be calibrated regularly and proof of calibration should be available on-site, (see the relevant British Standard for details on calibrations). In addition, the test procedure should include a means to check that the probe to be used has been correctly identified and is working satisfactorily. This forms a part of the quality management system.

26.3.4 ***Mechanical static cone penetrometer***

Two types of probe are used: the mantle cone, which measures only cone resistance; and the friction jacket cone, which measures both cone resistance and local friction. For both probes the cone and the friction sleeve can slide along the body of the probe and thrust against pressure rods located inside the hollow penetrometer rods.

The probe is pushed to the required depth by thrust on the outer penetrometer rods, the cone and friction sleeve having been tipped to make them slide into the closed position. Thrust is then applied to the inner pressure rods and measured, usually by a hydraulic load cell. The cone advances ahead of the body of the probe and has sufficient travel to enable a measurement to be taken of its ultimate cone resistance. In the friction jacket cone, further travel of the cone makes it engage with the friction sleeve. Both cone and friction sleeve are now advanced and have sufficient travel to give a measurement of the combined resistance of the cone and friction sleeve. This procedure is repeated at regular intervals of depth, which are normally 0.2 m.

The mechanical cone is now almost completely superseded by the electrical cone except for one or two special applications (see **26.3.1**).

26.3.5 ***Uses and limitations of the test***

The static cone penetration test is relatively quick to carry out and the results can be made available immediately following the completion of the test. It is usually cheap by comparison with boring, sampling and laboratory testing.

The results of a test are presented as plots versus depth of cone resistance, local friction and friction ratio (local friction divided by cone resistance), together with pore water pressure if a piezocone has been used. The frequency of data recording can be varied to suit the needs of the investigation. An assessment of these data can give a useful indication of the ground profile, together with many parameters

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such as strength, relative density and modulus of elasticity. Many correlations are now available for a wide range of soils and others are being developed [82] [83]. For any site, however, it is important to ensure that the correlation being used is valid for the particular strata being investigated.

The static cone penetration test is now widely used as an integral part of a site investigation and is commonly used as a rapid and economical means of interpolating the ground profile between boreholes. It can accurately detect the presence of thin soil layers of less then 100 mm thickness, which may easily be undetected by conventional boring and sampling. The test can reliably identify variations in strength across a site and with depth. The results can then be used to plan a programme of selective boring sampling and laboratory testing in zones of special interest. The test is the preferred substitute for the standard penetration test (see **25.2**) in soil conditions, particularly sands below the water table, where borehole disturbance may have rendered suspect the results of the standard penetration test. When the piezocone is used, dissipation tests, which monitor the decay of the porewater pressure measured by the cone during a pause in driving, can facilitate the assessment of the coefficient of consolidation and its variation across the site and with depth.

Penetration test equipment has been adapted to take piston samples (see **22.5**) in soft or loose soils from specific horizons and to operate the Delft continuous sampler (**22.6.2**). The samples are used for strata descriptions and conventional laboratory testing. The equipment can also be used to install piezometers and to carry out various in-situ tests including the push-in pressuremeter (**25.7.2.3**) and sampling, and environmental tests [83].

The cone penetration test was originally developed in the 1930s for use in recent alluvial soils. Its application, however, has since been widened to other materials, such as dense sands, overconsolidated clays, gravelly clays, chalk, other weak rocks and many more [83]. 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. It is also limited when ground is contacted which might damage the probe. Penetration is usually terminated when dense gravel, coarse gravel, a cobble or rock is encountered. Similarly, a 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 reducer. 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. For soft strata over hard strata the CPT rig can install casing to support the cone rods through the soft strata, allowing penetration of the hard strata. In very soft clays and silts the cone calibrations become very important. Special calibrations may be necessary with the calibrations biased towards the appropriate load range of the cone.

The last decade has seen extensive developments in the technology of the static cone penetration test with probes to measure a variety of other parameters such as, electrical conductivity, temperature and the velocity of seismic waves [83]. Probes for carrying out chemical testing of soil and ground water are also under development for use in contaminated ground [86].

27 Pumping tests

27.1 General principles

In principle, a pumping test involves pumping at a steady known flow from a well and observing the drawdown effect on ground water levels at some distance away from the pumped well. Pumping tests are described in BS 6316:1992. In response to pumping, phreatic and piezometric levels around the pumping well fall, creating a “cone of depression”. Cross-sections of typical cones are indicated in Figure 10. The permeability of the ground is obtained by a study of the shape of the cone of depression, which is indicated by the water levels in the surrounding observation wells. The shape of the cone of depression depends on the pumping rate; the duration of pumping; the nature of the ground; the existence, or otherwise, of intermediate or other boundaries; the shape of the ground water table; and nature of recharge. From the data obtained from the test, the coefficients of permeability, transmissivity and storage can be determined.

The results obtained are “averages” for the entire mass of ground which has been influenced by the pumping test. In permeable ground, pumping from the well may have a significant effect on piezometric pressures to a distance of 100 m or more. Flows from a well pumping test are accumulated from contributions coming from strata that may have very different values of permeability individually and may not be of consistent thickness within the radius of influence of the well. The overall result could be dominated by the flow from one highly permeable layer or discontinuity, which represents only a few per cent of the total thickness of the strata under test.

Down-the-hole velocity profiling, which can be done while pumping at a constant outflow, can often be useful when information is required on the permeability profile of the ground. Alternatively, it may be necessary to use borehole permeability tests, as described in clause **25**. It should be noted that a borehole test measures the permeability for a mass of ground extending only 1 to 2 m from the test position. An appreciable number of boreholes and a considerable number of borehole tests may therefore be required to investigate an extensive site with considerable variations in permeability.

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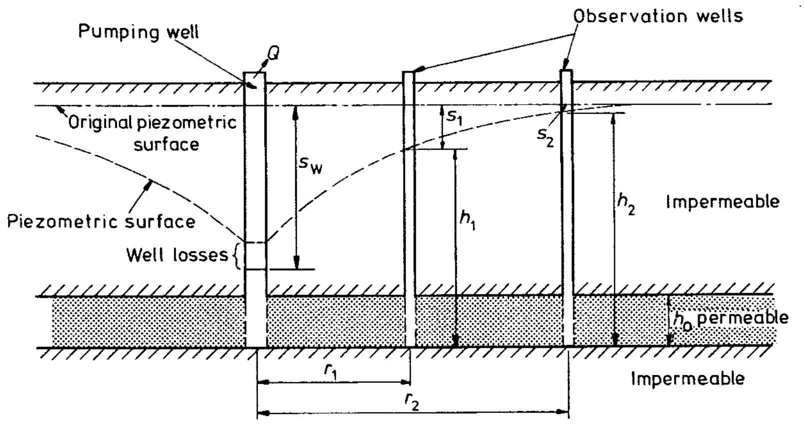
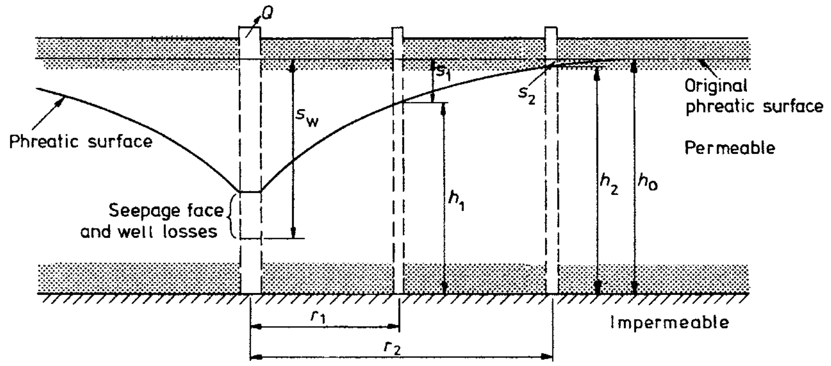
Case 1. Confined conditions



Permeability is given by:

$$k = \frac{2.3Q}{2\pi\, h_0\, (s_1 - s_2)} \times \log_{10} \frac{r_2}{r_1}$$

Case 2. Unconfined conditions



Permeability is given by:

$$k = \frac{2.3Q}{\pi\, (h_2^2 - h_1^2)} \times \log_{10} \frac{r_2}{r_1}$$

where

k	is the permeability in m/s;
Q	is the rate of flow of pump in m^3/s;
s_1, h_1, r_1 etc.	are distances as shown in m.

Figure 10 — Calculation of permeability from pumping tests

Before attempting to carry out a pumping test, reliable data should be obtained on the ground profile, if necessary by means of boreholes specially sunk for the purpose. The maximum advantage should be taken of boreholes used for the well and piezometers. All of these should be carefully logged and borehole tests should be undertaken as appropriate.

The natural groundwater conditions should be determined by careful monitoring over a sufficient period before the pumping test. Ideally, the conditions should be stable during the test. If they are not stable, the fluctuations, which can be caused by tidal and barometric effects, and interference from nearby pumping installations, have to be recorded. This is particularly important in aquifers with low storage coefficients.

The results of a pumping test yield suitable information for the design of large-scale ground water lowering schemes and for the assessment of seepage and drainage. On the other hand, borehole permeability tests are essential for projects requiring a detailed knowledge of the distribution of the permeability of the ground, e.g. cut-offs for dams, tunnels shaft sinking etc.

The interpretation of the data from a pumping test can be complicated and is much affected by the inferred ground conditions and the influence of any boundaries. Where necessary expert advice should be sought.

Pumping tests can be expensive, requiring adequately screened and developed pumping and observation wells, suitable pumping and support equipment, and personnel. Care should therefore be taken to design a suitable test programme.

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27.2 Groundwater conditions

There are two main types of groundwater conditions: confined and unconfined; both of these should be recognized for analytical and design purposes.

a) **Confined**

If the ground under investigation is fully saturated and the water is confined under pressure between two impermeable layers, then confined conditions are said to exist (see Figure 10).

b) **Unconfined**

If the original phreatic level is everywhere below the upper surface of the aquifer, then unconfined conditions are said to exist (see Figure 10).

Intermediate between the above two groundwater conditions is a third called the semi-confined condition. In this case fully saturated ground is overlaid by material of significant but lower permeability, and significant leakage takes place across the boundary in response to pumping. Analysis of data from semi-confined conditions is possible, but the condition is often not identified. The three types of groundwater conditions may be recognized from the plots during testing by the way in which water or piezometric levels fall with respect to time in response to pumping (see Figure 11).

27.3 Test site

Although the choice of test site may be dictated by practical considerations such as access and availability of existing boreholes, the site should be representative of the area of interest. The hydrological conditions should not change appreciably over the site. It is essential that discharged water is not allowed to return to the ground under test.

27.4 Pumped wells

Pumped wells should be of sufficient diameter to permit the insertion of a rising main and pump of a suitable type and capacity, together with a standpipe and velocity meter, if required. They should be provided with an adequate well screen, and filter pack where necessary, to prevent the withdrawal of fine particles from the surrounding soil; 300 mm is often the minimum borehole diameter for this purpose. It is desirable that the wells penetrate the full depth of the water-bearing strata being tested, although this is often not practical, particularly where the aquifer is thick, mixed or layered.

Where the ground is composed of two or more independent horizons, each should be tested separately. Where fully penetrating conditions do not exist, the data has to be corrected before analysis (see **28.4**). In all cases, the screen intake area should be large enough to ensure that the maximum velocity of water entering the well is no greater than about 30 mm/s; which means that hydraulic well losses are of an acceptable level.

If, during the test, changes occur in the shape of the cone of depression because of extraneous causes, and these are a significant fraction of those due to pumping, then the resulting estimate of permeability may become unacceptable, unless such influences can be corrected by monitoring [88], both before and during testing. Where possible, and within the limitations set by the permeability, the pumping rate should be chosen so that resulting changes in water levels are much greater than those due to extraneous causes, thus minimizing their effects on the results.

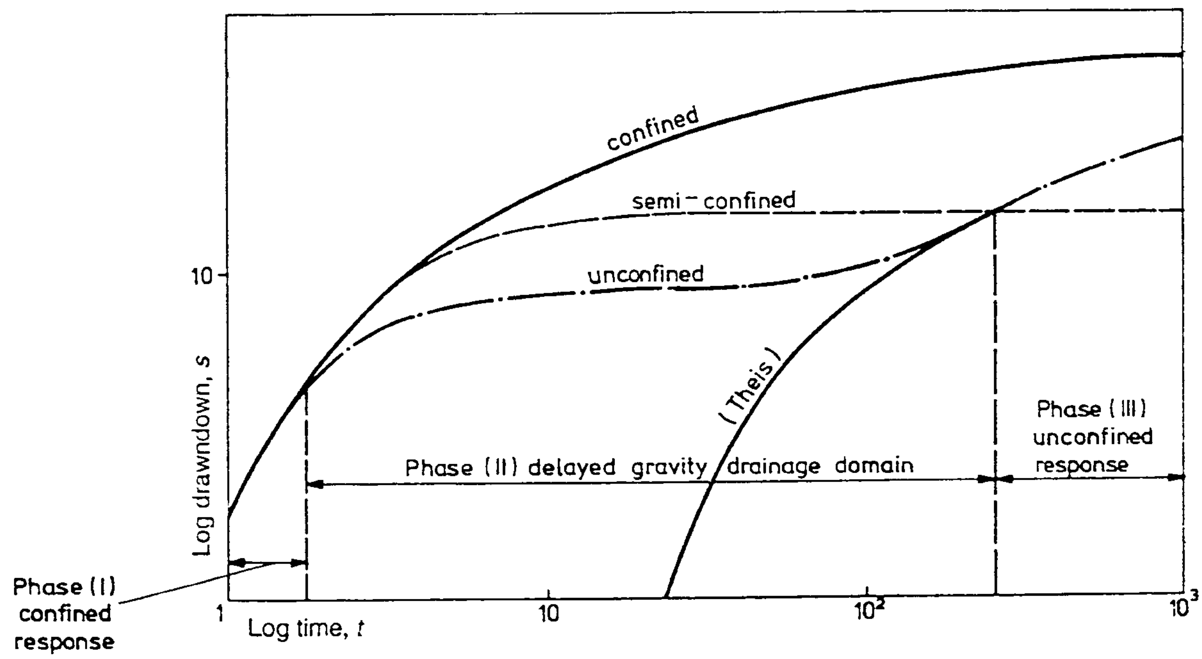

Figure 11 — Typical drawdown time pumping responses (after Theis [87])

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Suction pumps can be used where the ground water does not have to be depressed by more than about 5 m below the intake chamber of the pump, and setting the pump in a pit can increase drawdown. For greater depths, submersible pumps are preferable. Alternatively, air lifting can be used inside a riser pipe. The more permeable the ground, the greater the pump capacity required to produce measurable drawdowns in the observation wells.

Normally, the discharge is kept constant for the duration of the test and all the water level observations are related to a timescale connected to the onset of pumping.

It is particularly important to maintain a constant pumping rate when vertical flow velocities in the pumping well are being measured for the purposes of assessing the relative permeabilities of specific horizons in the ground under test. The pumping rate may be controlled by a gate valve in the discharge line or by varying the speed of the pump, or both. The rate of flow from the pump may be measured by a flow or orifice meter, or by a notch tank with automatic recording.

Step tests at varying rates up to the maximum yield of the well can be used to assess the efficiency of the well [89] [90].

It is important that pumped wells should be adequately developed. Development of a well is the process by which particles surrounding the screen are rearranged, so that more coarse grade particles and those more uniform particles are closer to the screen. This can be achieved in a number of ways [91]. Maximum development is indicated when the ratio of pumping rate to fall in water level in the pumping well reaches a maximum. Fine particles from the ground are removed during development, resulting in a stable porous and permeable medium, surrounding the well. This results in reduced hydraulic head losses (well losses), which are not accounted for in the following analyses, so modifications need to be applied (see **28.2.2.5**).

27.5 Observation wells

The observation wells should have an internal diameter sufficient to permit the insertion of a suitable water level measuring device or transducer. They should penetrate the same ground as the pumping well and should permit entry of water from the full depth of ground being tested. If there is any risk that fine soil particles may clog the observation wells, a suitably graded filter material should surround them.

Although the permeability of the ground may be estimated from the pumping well drawdown data alone, more reliable values are obtained using data from one or more observation wells. The minimum number of these is two, but this may limit the interpretation, particularly in complex or poorly understood hydrogeological conditions. The recommended minimum, for general application, is four observation wells, arranged in two rows at right angles to each other. Their distances from the pumping well should approximate to a geometrical series. It may be necessary to add more wells if the initial ones yield anomalous data. If linear boundary conditions are associated with the site, e.g. river, canal or an impermeable subsurface bedrock, scarp, fault or dyke, the two rows of observation wells are best arranged parallel and normal to the boundary.

The minimum distance between observation wells and the pumping wells should be ten times the pumping well radius, and at least one of the observation wells in each row should be at a radial distance greater than twice the thickness of the ground being tested. However, unless the pumping rate is very high, and the duration of pumping long, particularly low permeability ground under unconfined conditions, falls of water levels may be small at such distances. Preliminary calculations using assumed permeabilities and storage coefficients estimated from borehole data help to indicate the likely response in observation wells to pumping, and from this their spacing from the pumping well and timing of observations can be worked out.

In addition to the observation wells described above it is necessary to have an additional standpipe inside the pumping well in order to obtain a reliable record of the drawdown of the well itself.

Depths to water levels should be measured to within ±5 mm. It is preferable to use recording pressure transducers, which are checked against manual dippings at appropriate intervals.

27.6 Test procedures

Pumping should commence once the character of fluctuations and other extraneous influences has been established, the test programme has been designed and the wells developed. A variety of test procedures is possible. A typical test may comprise a series of stages of a day or two each and consisting of a constant rate test and a recovery test. Shorter, longer or more complicated programmes may be appropriate depending on the ground and the purpose of the test. Frequent water level readings in all observation wells are advisable, and continuous data logging of electric readings provides higher quality data as well as significant manpower savings. Further details on test procedures and reading frequency are given in BS 6316.

The data should be plotted on a log-log or linear-log graph during the course of pumping to evaluate the quality of data and the nature of the response by the ground. The nature of this response for the three ground conditions previously referred to in **27.2** is indicated in figure 11. Such curves can be used to confirm the duration for the pumping period.

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In case of an unconfined aquifer where the steady state analysis is to be used, pumping and reading should be continued until the water levels cease to fall. On a log-log graph, the field curve typically flattens out before returning to a Theis curve after several hundred to several thousand minutes. It is most important that pumping continues for some time into the third phase, and that the water levels fall very slowly before they can be said to have stabilized. A non-steady state (non-equilibrium) analysis is preferable, though, and pumping should continue until the field curves plotted on a log-log graph return to the Theis curve in the third phase as shown in Figure 11.

For confined conditions where the steady state (equilibrium) analysis is to be used, pumping and readings should be continued until equilibrium is reached. Non-steady state analysis is preferred, in which case pumping may be terminated within a few hundred minutes or less for more permeable strata as the field curves generally approximate to the Theis curve from the first few minutes of pumping. It may be desirable to check the results by repeating the test at a different flow rate.

In all cases, water levels should continue to be monitored with respect to time from cessation of pumping until recovery of levels to the original values is complete (see BS 6316). The data may then be analysed in a similar way to that described in **28.3.2.3** [91].

28 Analysis of pumping tests

28.1 Introduction

The principles of data collection are the same for all methods of analysis.

Plotting and evaluation of data should be carried out during the test to ensure adequate duration of pumping. There are two forms of analysis:

a) **Steady state**

If pumping continues long enough, water levels cease to fall, and the hydraulic condition of the ground is said to be in a steady state with respect to time.

b) **Non-steady state**

Until equilibrium is achieved, water levels fall at changing rate with respect to time and the hydraulic condition of the ground is said to be in a non-steady state.

The methods of analysis are normally based on the following simplifying assumptions:

1) the aquifer is infinite in lateral extent;

2) the ground is homogeneous, isotropic and of constant thickness;

3) the ground water level gradient is very small before pumping;

4) the ground is pumped at a constant flow rate;

5) the pumping well penetrates and is screened to full depth of ground horizon under test;

6) the ground water flow to the well is laminar.

28.2 Steady state analysis

The simplest form of analysis is the steady state analysis, but the necessary duration of pumping can be many days longer than that necessary for the non-steady analysis. The approach is difficult to use and has led to many problems in the design of dewatering systems. For these reasons, the non-steady state analysis is preferred.

28.3 Non-steady state analysis

28.3.1 *Introduction*

Several methods on non-steady state analysis are possible; use of these methods can in certain cases save days of pumping time. Although analyses given below are commonly applied to drawdown data, similar analyses may also be used with recovery data because on cessation of pumping, water and piezometer levels should ultimately recover to their initial levels. Analysis of recovery data can provide a useful check on drawdown analyses as it is not subject to well-losses occurring during the pumping phase. It should therefore always be observed.

28.3.2 *Confined conditions*

28.3.2.1 *Assumptions*

In addition to the assumptions listed in **28.1**, the following assumptions are made for confined conditions using non-steady state analysis:

a) the ground is confined;

b) piezometric levels have not stabilized;

c) water removed from storage is discharged instantaneously with change in head;

d) storage water in the well is negligible.

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28.3.2.2 *Theis analysis [87]*

The common method of analysis consists in plotting changes in piezometer levels for each piezometer against time since pumping commenced.

By Theis, drawdown, s, in an observation well is given by:

$$s = \frac{Q}{4\pi T} W(u)$$

where

$$u = \frac{r^2 S}{4Tt}$$

and where:

s	is the drawdown in metres (m);
Q	is the constant discharge of the pumping well in cubic metres per second (m^3/s);
T	is the coefficient of transmissivity of the ground in square metres per second (m^2/s) (equal to kh_o);
$W(u)$	is an exponential function referred to as the "well function" (non-dimensional);
r	is the radial distance of the observation well from the test well in metres (m);
S	is the storage coefficient (non-dimensional);
t	is the time of pumping in seconds (s);
h_o	is the thickness of the aquifer in metres (m).

The analysis comprises the following steps:

a) a Theis "type curve" is prepared on logarithmic paper of $W(u)$ against u (see Figure 12 and Table 5);

b) values of drawdown, s, are plotted against values of r^2/t on logarithmic paper of the same size as the "type curve";

c) the observed data are superimposed on the "type curve" keeping the co-ordinate axes of the two curves parallel, and adjusted until a position is found by trial whereby most of the plotted points of the observed data fall on a segment of the type curve;

d) an arbitrary point is selected on the coincident segment and the co-ordinates of this matching point are recorded. The values of $W(u)$, u, s, and r^2/t so determined are substituted in the equations given above.

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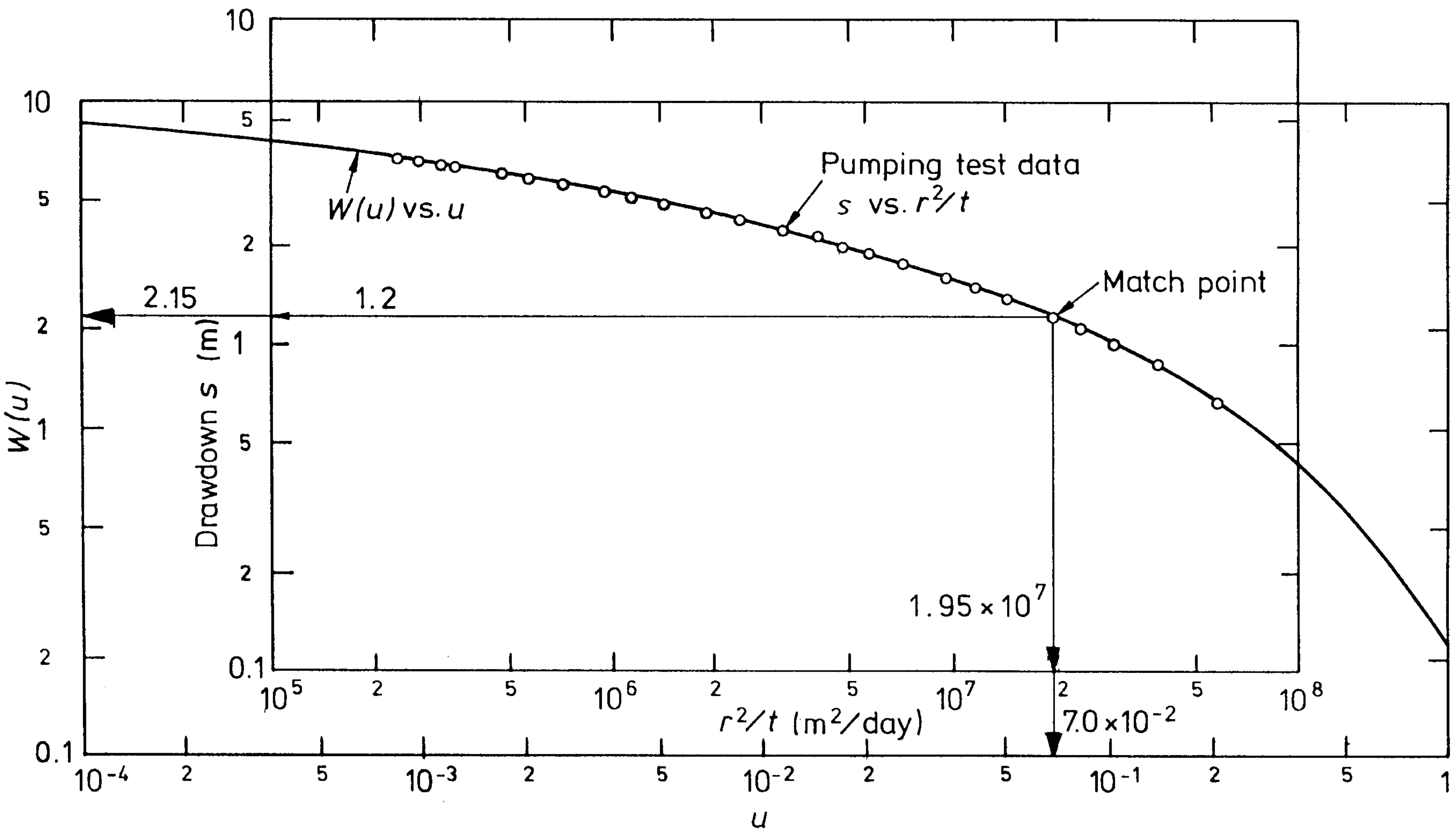

Figure 12 — Theis method of superposition for solution of the non-equilibrium equation

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Table 5 — Values of $W(u)$ for values of u [92]

u	1.0	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0
$\times 1$	0.219	0.049	0.013	0.0038	0.0011	0.00036	0.00012	0.000038	0.000012
$\times 10^{-1}$	1.82	1.22	0.91	0.70	0.56	0.45	0.37	0.31	0.25
$\times 10^{-2}$	4.04	3.35	2.96	2.68	2.47	2.30	2.15	2.03	1.92
$\times 10^{-3}$	6.33	5.64	5.23	4.95	4.73	4.54	4.39	4.26	4.14
$\times 10^{-4}$	8.63	7.94	7.53	7.25	7.02	6.84	6.69	6.55	6.44
$\times 10^{-5}$	10.94	10.24	9.84	9.55	9.33	9.14	8.99	8.86	8.74
$\times 10^{-6}$	13.24	12.55	12.14	11.85	11.63	11.45	11.29	11.16	11.04
$\times 10^{-7}$	15.54	14.85	14.44	14.15	13.93	13.75	13.60	13.46	13.34
$\times 10^{-8}$	17.84	17.15	16.74	16.46	16.23	16.05	15.90	15.76	15.65
$\times 10^{-9}$	20.15	19.45	19.05	18.76	18.54	18.35	18.20	18.07	17.95
$\times 10^{-10}$	22.45	21.76	21.35	21.06	20.84	20.66	20.50	20.37	20.25
$\times 10^{-11}$	24.75	24.06	23.65	23.36	23.14	22.96	22.81	22.67	22.55
$\times 10^{-12}$	27.05	26.36	25.96	25.67	25.44	25.26	25.11	24.97	24.86
$\times 10^{-13}$	29.36	28.66	28.26	27.97	27.75	27.56	27.41	27.28	27.16
$\times 10^{-14}$	31.66	30.97	30.56	30.27	30.05	29.87	29.71	29.58	29.46
$\times 10^{-15}$	33.96	33.27	32.86	32.58	32.35	32.17	32.02	31.88	31.76

28.3.2.3 *Cooper–Jacob analysis [93]*

When $u = (r^2S)/(4\ Tt) \leq 0.01$, the analysis may be simplified. The assumption that u is small may be satisified for confined conditions. The analysis comprises the following steps using either the time drawdown data or th distance drawdown data.

a) Time drawdown data:

1) drawdown data with respect to time at each piezometer is plotted on a semi-logarithmic diagram;

2) the field plot should be approximated to a straight line; read off the change in drawdown, Δs, over one time log cycle from the straight-line part of the curve;

3) substitute values into the following equations (see Figure 13);

$$k = \frac{2.3Q}{4\pi\ \Delta s\ h_o}$$

$$S = \frac{2.25\ k\ h_o\ t_o}{r^2}$$

where

- k is the permeability in metres per second (in m/s);
- Q is the constant discharge of the pumping well in cubic metres per second (in m^3/s);
- s is the drawdown in metres (m);
- h_o is the thickness of the aquifer in metres (m);
- S is the storage coefficient (non-dimensional);
- t_o is the extrapolated time at zero drawdown in seconds (s);
- r is the radial distance of the observation well from the test well in metres (m).

NOTE 1 Although t_o is in seconds in the above equation (to give k in m/s), it is convenient to plot t in minutes (see Figure 13).

b) Distance drawdown data:

1) the variation of drawdown with respect to distance at a particular time is plotted on a linear-logarithmc diagram for two or more piezometers;

2) the field plot should be a straight line; read off the change in drawdown, Δs, over one distance log cycle;

3) substitute values into the following equations (see Figure 13):

$$k = \frac{2.3Q}{2\pi\ \Delta s\ h_o}$$

$$S = \frac{2.25\ k\ h_0\ t}{r_0^2}$$

where r_o is the extrapolated distance at zero drawdown.

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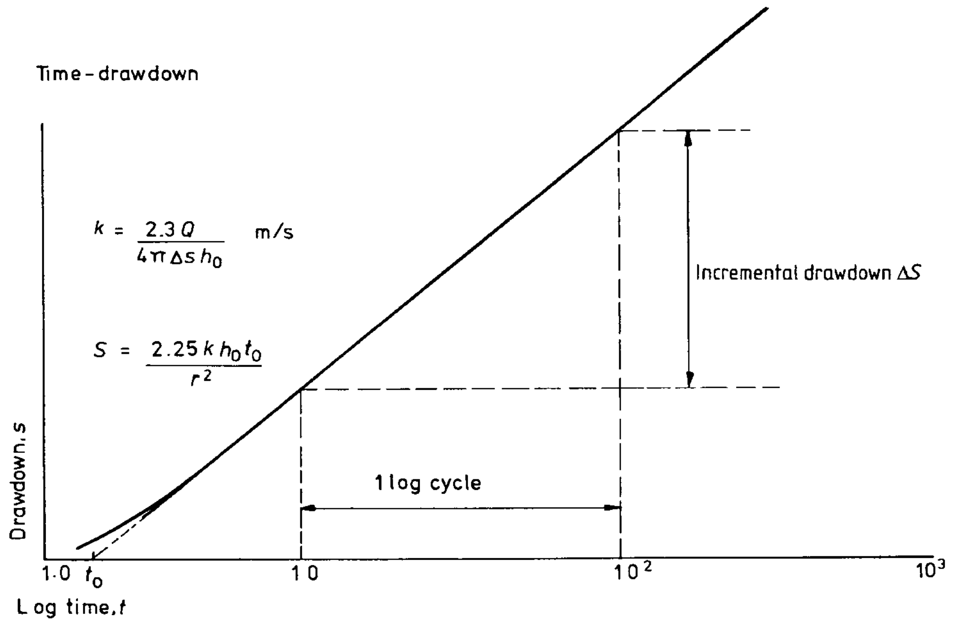
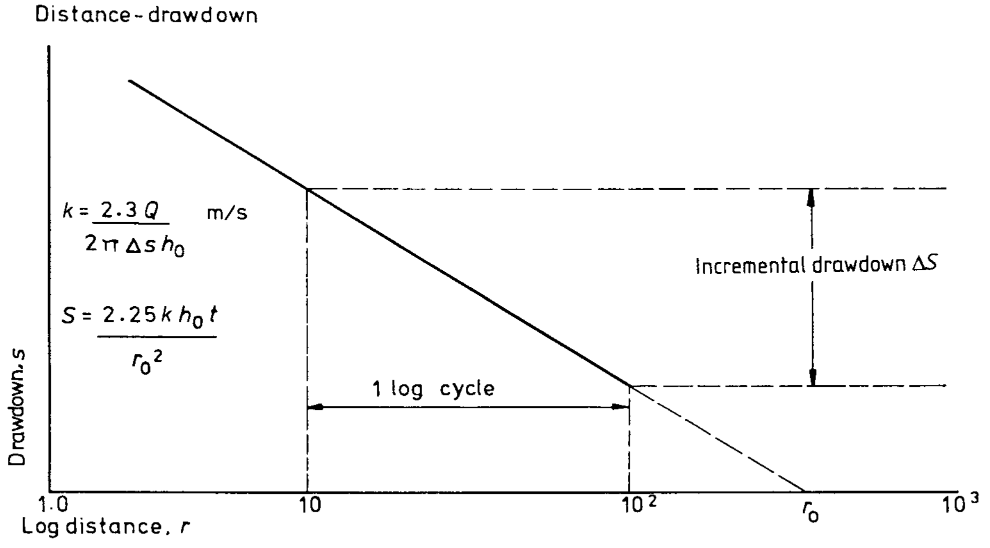


a) Time against drawdown



b) Distance against drawdown

NOTE 1 $u \leq 0.01$

NOTE 2 Valid for analysis of unconfined conditions only when delayed gravity drainage complete [see phase (3), Figure 11] and drawdowns have been corrected (see **28.2.3**).

Figure 13 — Cooper–Jacob straight-line analysis

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28.3.3 ***Unconfined conditions***

28.3.3.1 *Assumptions*

In addition to the assumptions listed in **28.1**, the following assumptions are made for the unconfined condition using the non-steady state of analysis:

a) the ground is unconfined, showing delayed gravity drainage on field plot (see Figure 11);

b) water levels have not stabilized;

c) stored water in the well is negligible.

28.3.3.2 *Theis analysis and Cooper–Jacob analysis*

The methods as described in **28.3.2** may be used provided that drawdowns are corrected as given in **28.2.3** and a double logarithmic plot drawdown against time shows that the field curve has encountered the third phase and is approximately the same as the Theis curve.

28.3.3.3 *Prickett analysis*

Procedures for analysing the non-steady state unconfined test data are described by Prickett [94]. The method is similar but slightly more complicated than a Theis analysis, but permits the use of field data from onset of pumping. Recent analytical work, e.g. see [95] on the hydraulics of wells in unconfined conditions, has considerably expanded and improved on the methods given by Prickett.

28.4 Validity of drawdown data

The drawdown data collected in the field often differs from the idealized results shown in **28.3.2.2** and **28.3.2.3**, Figure 11. One reason for this is often that the aquifer itself does not have the idealized properties ascribed to it in the analysis. Examples are anisotropy (most aquifers are more permeable along the bedding) and inhomogeneity (particularly open fissures). Another reason is that the layout of the pumping test may differ from the ideal conditions. Examples are the presence of a lateral impermeable barrier, e.g. a dyke or fault; a nearby body of surface water, e.g. a river or the sea; recharge of an unconfined aquifer by rainfall; and the well not fully penetrating the aquifer.

The effects of any variation in pumping rate should be taken into consideration, and it should also be noted that in some cases high flow velocity around the well may invalidate the use of Darcy's law, upon which the methods of analysis are based. Well losses should be calculated from the results of step drawdown tests.

It is important that the existence of such conditions is fully appreciated, so that the analysis can be suitably amended. Analytical solutions are available for some of these cases and in complex conditions the normal methods of analysis may become unworkable, although numerical modelling techniques can sometimes be used.

28.5 Numerical methods

The radial flow method [96] can be used to model complex ground conditions including leaky aquifers, well storage and anisotropy. Numerical solution of the Theis equation can be used to generate curves for comparison with the drawdown data for variable rate tests or for tests carried out with more than one pumping well. A suite of groundwater programs has been developed by the United States Geological Survey for analysis of pumping tests.

28.6 Pressure depletion and recovery tests

Where groundwater problems need to be explored in deep boreholes in the rock, preparatory to construction of, e.g. mine shafts, tunnels, or the exploitation of mineral resources, it is not economic to employ an array of observation wells to establish the cone of depression, as described in **26.2**. In such a case it is necessary to extract the required information from a single borehole.

Cores of the rock are extracted, from which a geological log may be obtained, and this is supplemented by a suite of downhole well-logging routines. Analyses of the resulting data demonstrate strata where there may be problems with groundwater in future construction works. The logs also indicate which parts of water-bearing strata are suitable for the setting of packers to isolate sections of the hole containing the strata of interest.

These are packed off and subjected to water pressure depletion by pumping at a steady rate, usually for a period of a few hours. The pump is suddenly cut off and the flow valve closed, so that pressure within the section isolated by the packers begins to rise towards its original piezometric level. The pressure changes with time are recorded automatically by pressure transducers and data logging. The results can be used to derive the aquifer transmissivity T from the slope m and the constant rate of withdrawal Q during the initial flow period of the test [97].

$$\mathrm{T} = \frac{2.303Q}{4\pi\, m}$$

m is expressed in terms of piezometric head per log cycle. The pressure converges towards the natural piezometric head in the aquifer, tested as the abscissa term tends to zero and representing pressure recovery at infinite time.

Changes in the property of the aquifer are reflected in changes of slope of the plot. Local effects close to the borehole such as damage during boring, effects of drill mud, and opening of fissures close to the bore may produce slight curvature of the plot at short times after cessation of flow; these are not representative of the aquifer as whole. Accordingly attention is directed in analysis to later times in the pressure recovery. This method is more fully described with examples from civil engineering and mining use in [98].

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It provides data for the strata tested on transmissivity, permeability, storage capacity, and a static piezometric head. It also indicates the distance to any aquifer feature that causes a change in properties; the nature of such a feature may be interpreted by reference to known geological information. For greater definition the test may be repeated using different rates of initial steady flow.

29 Field density

29.1 General principles

In essence, most of the available methods depend upon the removal of a representative sample of soil from the site and then determining its mass and the volume it occupied before being removed. The determination of mass is common to all methods and is straightforward. The variations lie in the several procedures used for measuring the volume and these depend upon the nature of the soil being tested. The following methods of test are in general use and are not described in detail here because they are fully covered by clause **2** of BS 1377-9:1990 and the ASTM standards quoted. All methods require physical access to the soil in-situ, and are therefore normally restricted to soil within 2 m or so of the surface, although, of course, they can equally well be used in deep shafts or headings.

The tests determine bulk density. The moisture content of the sample to be weighed has to be representative. Ideally, the weighing should be done on-site; if this is not possible, the entire sample has to be preserved in an airtight container until it can be weighed. The test can be of limited accuracy and it may be necessary to take the average of at least three determinations to obtain a significant result.

29.2 Sand replacement method

BS 1377-9:1990 describes two variations on the sand replacement method (**2.1** and **2.2**). The first, employing a small pouring cylinder, is used for fine- and medium-grained soils, as defined in BS 1377. The second, using a large pouring cylinder, is suitable for fine, medium and coarse-grained soils. These methods are unsuited to soils containing a high proportion of very coarse gravel or larger particles because the apparatus is not large enough to cope with a hole of sufficient size to obtain a representative sample; the sand also tends to run into the interstices of the material, thus leading to inaccurate results. The method cannot be used in soils where the volume of the hole cannot be maintained constantly. It also loses accuracy in soils where it is difficult to excavate a smooth hole because the sand cannot easily occupy the full volume.

29.3 Core cutter methods

The core cutter method is described in BS 1377-9:1990, **2.4**. The method depends upon being able to drive a cylindrical cutter into the soil without a significant change of density and retaining the sample inside it so that the known internal volume of the cylinder is completely filled. It is therefore restricted to fine soils that are sufficiently cohesive for the sample not to fall out, and to chalk soils or completely weathered rock free of stones. It may be preferable in cohesive and sensitive soils to trim and push the cutter rather than drive it.

29.4 Water replacement method

The water replacement method is described in BS 1377-9:1990, **2.3**. The method is normally used in coarse and very coarse soils (including rockfill) when the other methods for determining the field density are unsuitable because the volume excavated would be unrepresentative. In principle, it consists in excavating a hole large enough to obtain a representative sample, lining the hole with flexible polyethylene or similar sheeting and then determining the volume of water required to fill the hole.

The accuracy of the results of this test can be enhanced by attention to the following details:

a) the hole should be made as large as possible;

b) the sides of the hole should be made as smooth as possible;

c) as thin a gauge of polyethylene as possible should be used, consistent with it not fracturing or puncturing too easily.

29.5 Rubber balloon method

A description of the rubber balloon method is found in ASTM D 2167 [99]. In essence it is a water replacement test, with a rubber membrane to retain the water. It is an alternative to the sand replacement method, with the limitation that it is not suitable for very soft soil which deforms under slight pressure or in which the volume of the hole cannot be maintained constantly. The ASTM standard does not lay down the nature of the apparatus in precise terms, so the method could be used for soils coarser than the sand replacement method if a sufficiently large apparatus were constructed; for practical purposes, however, the limitation in this respect is likely to be similar.

29.6 Nuclear methods

Nuclear methods are described in BS 1377-9:1990, **2.5**. These do not measure density directly; calibration curves have to be established for each soil type, which involves measuring the densities of representative samples of the soils concerned by conventional methods. Once this has been done and provided there are no significant changes in soil type, the method is very much faster than the others. It is therefore most suited to jobs where there is a

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continuous need for density determinations over a long period of time and where the soil types do not vary to any significant extent. It should be noted that the density determined by these methods is not necessarily the average density within the volume involved in the measurement. The equipment utilizes radioactive materials, and appropriate safety precautions should be taken.

29.7 Uses of the field density test

The major use of this test is in the control of the compaction of earthworks. It is also used in connection with the design of road and airfield pavements and in the control of the compaction of sub-grades on which they rest. It can be used for the determination of natural in-situ density, where it is difficult or impossible to take undisturbed samples. It forms the "field" element of the relative density test, the other two elements often being carried out in the laboratory.

30 In-situ stress measurements

30.1 General

The stresses existing in a ground mass before changes caused by the application of loads or the formation of a cavity within the mass are referred to as the initial in-situ state of stress. These stresses are the result of gravitational stress and residual stresses related to the geological history of the mass.

Data on the initial in-situ state of stress in rock and soil masses before the execution of works are important in design. The most favourable orientation, shape, execution sequence and support of large and complex underground cavities, and the prediction of the final state of stress existing around the completed works, are all dependent on knowing the initial in-situ state of stress. Measurements of in-situ stress have shown that in many areas the horizontal stresses exceed the vertical stress, which in turn often exceeds that calculated, assuming that only gravity is acting on the ground mass.

Measurement of in-situ stress in soils may be made although the equipment used normally provides an estimate of horizontal stress only. To enable both total and effective stresses to be estimated, it is usual to measure the pore water pressure in addition to the total stress.

The interpretation of in-situ stress measurements requires special experience.

30.2 Stress measurements in rock

30.2.1 *General principles*

With the exception of the static equilibrium method (see **30.2.5**), the methods described are based on stress changes, achieved in some cases by over-coring or slotting a previously instrumented test area. Over-coring is used for measurement within the rock mass, whereas slotting is used for surface stress measurements. Measurements taken have to be adjusted to take account of the redistribution of stresses as a result of formation of the borehole or slot and when the measurement is made in the zone of influence of the main access, such as an adit. Stress measurements may also be determined from the measurement of displacements of the walls of a tunnel, or of an exploratory adit, close to the working face.

The techniques often require that the material in which the measurements are made behaves in a near elastic, homogeneous and isotropic manner and that it is not prone to swelling as a result of drilling water, or excessively fractured. Analyses are available that evaluate measurements made in anisotropic material but are not widely used. For the over-coring methods, the elastic behaviour is assumed to be reversible, the elastic constants being obtained from field or laboratory tests.

Stress measurements may be made using electrical strain gauges, photoelastic discs, solid inclusions and systems for measuring the diametrical change of a borehole. Some equipment is designed to measure stress change with time, or stress change due to an advancing excavation, whereas other equipment is designed to obtain an instantaneous measurement of stress. The technique selected has to be chosen in relation to the rock material, mass quality and water conditions.

The determination of the triaxial state of stress at a given point requires measurement in at least six independent directions. It is, however, desirable to have redundant data for better evaluation by statistical methods of error distribution.

The report on the results of in-situ stress measurement should include the following:

a) location of test and direction and depth of the drill holes, method of drilling and diameters of cores;

b) depth below ground level of the point of measurement;

c) geological description of the rock mass;

d) strain readings to the nearest 10 micro strain;

e) the modulus of elasticity, E, and Poisson's ratio, v, of the rock determined from static laboratory testing of core preserved at in-situ moisture content, over the appropriate stress path, from each stress measurement area;

f) the six components of stress (σ_x, σ_y, σ_z, τ_{xy}, τ_{yx}, τ_{zx}) at each point to the nearest 100 kPa;

g) the three principal stresses and the directions (to the nearest degree), related to both a borehole or adit axis system and a global axis system;

h) colour photographs of the cores or test location;

i) date of measurement and data at which excavation passes the point of measurement.

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30.2.2 ***Determination of the in-situ triaxial state of stress in rock***

The most widely adopted method for the determination of the in-situ triaxial state of stress in one set of measurements uses the CSIRO hollow inclusion stress cell. The method is one of over-coring a cell containing nine or twelve electrical strain gauges installed on the walls of a pilot drillhole. The test is relatively cheap and quick to perform. A full description of the equipment and the test procedure is given in the standard instruction manual [100] and the International Society of Rock Mechanics (ISRM) Suggested Methods for rock stress determination [101]. The strains are measured over relatively small gauge lengths, approximately 10 mm, on a small test area, so it is considered advisable to correlate and cross-check with data obtained from other tests involving a larger test area, such as a flat jack test in the side walls of a suitably shaped and oriented adit (**30.2.4**). This is not always possible, however.

The stress cell, shown in Figure 14a), contains three or four oriented rosettes each of which has three gauges and a temperature compensating gauge or thermistor. Nine or twelve independent strains are recorded, of which six are necessary to determine the total state of stress, and the three redundant measurements are used as a check and for estimating errors.

A second instrument that can be used in deep water-filled boreholes drilled from the surface is the Borre Probe. This has been used in the UK to depths of up to 250 m [102]. The probe shown in Figure 14b) contains three oriented rosettes, a temperature gauge and a dummy gauge. Strain changes during overcoring are recorded by a downhole data logger without connection to the surface.

It is usual when carrying out measurements from underground openings to carry out several tests in each borehole at increasing depths in order to investigate the change in stress distribution as a result of excavation. If possible, two holes should be drilled at orthogonal directions to take account of any anisotropic characteristics of the rock.

All methods require a knowledge of the modulus of elasticity and Poisson's ratio of the rock. These can be obtained by biaxial testing of the overcore sample in a Hoek cell in the field, or laboratory testing of the core preserved at in-situ moisture content, under the appropriate stress path.

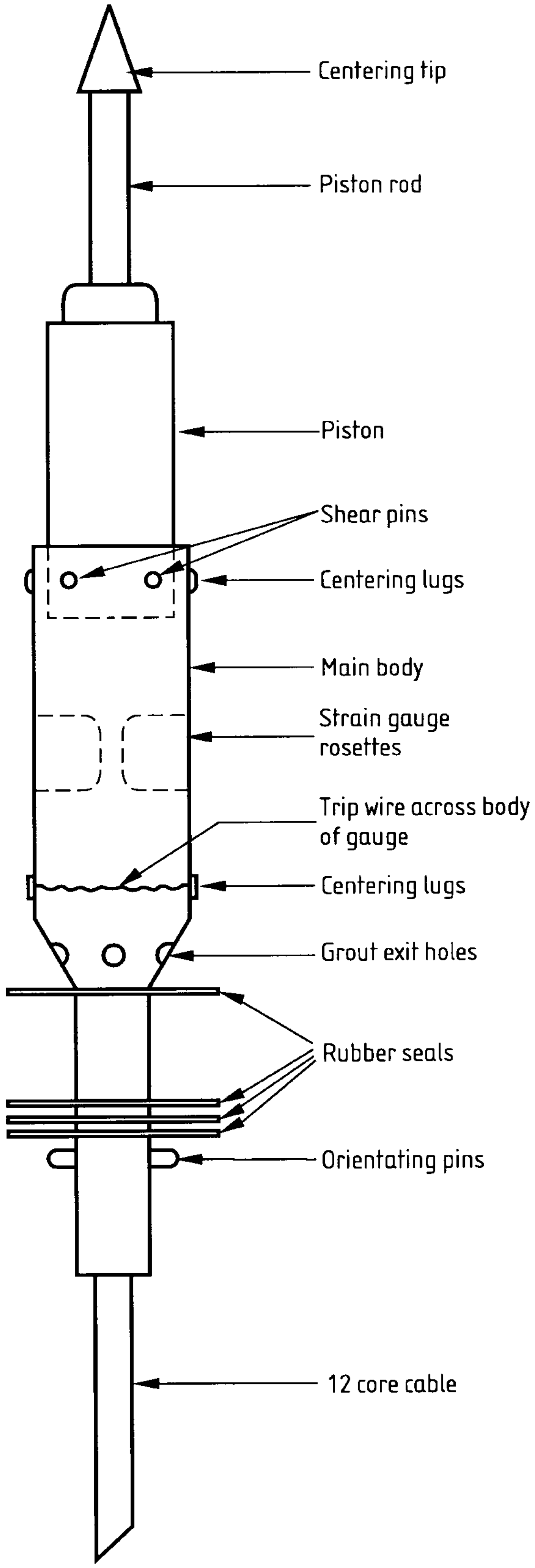

Figure 14a) — Measurement of in-situ stress — CSIRO cell

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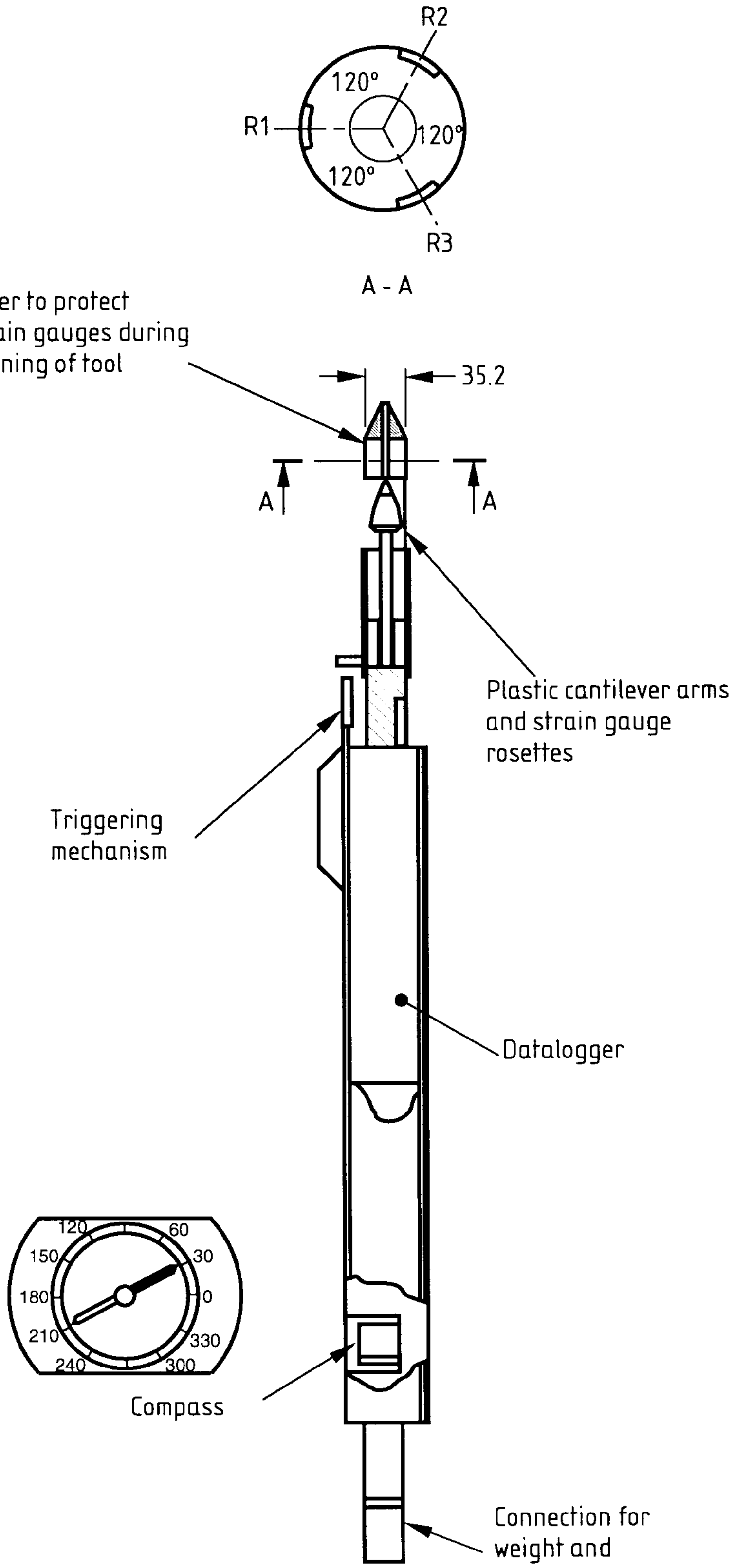

Figure 14b) — Measurement of in-situ stress — Borre probe

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30.2.3 ***Biaxial stress measurements in rock***

The USBM (United States Bureau of Mines) borehole deformation gauge comprises three strain gauges bonded to cantilevers that measure the changes in pilot hole diameter during overcoring. Stress components in the plane perpendicular to the borehole can be evaluated from the results. To enable the resolution of the total state of stress, this technique requires measurements to be made in three mutually perpendicular holes drilled into one area. The gauge can be used in water-filled boreholes provided the water pressure is less than 60 kPa.

Details of installation and test procedure may be found in the standard instruction manual and the ISRM suggested methods for rock stress determination [101].

30.2.4 ***Uniaxial stress measurements near a surface using a flat jack***

By means of a saw or overlapping holes, a slot is cut into a rock surface provided by an adit or prototype underground excavation. The stresses previously acting across the slot are relieved as the rock moves into the slotted void. This movement is measured by the convergence of marked points that are fixed on either side of the slot before it is formed. A suitable hydraulic flat jack is embedded in the slot and the pressure in the jack increased until the convergence of the datum points is cancelled. If creep can be ignored, the cancellation pressure is related to the stresses that were acting normally on the plane of the slot before it was formed.

All three methods require knowledge of the elastic properties. These can be obtained by biaxial testing of the over-core sample in a Hoek cell.

The stresses measured by this means are those parallel and near to the rock surface in which the slot is cut. The location of the adit should be such that its axis is driven in a direction nearly parallel to that of the proposed prototype excavation and as close as possible into the zone of interest. Excavation of the test zone in the adit has to be carefully carried out, preferably by hand excavation or using smooth blasting techniques, and the period between excavation and measurement should be as short as possible. A typical section of a flat jack test is shown in Figure 14c).

The equipment and procedures are fully described in the ISRM suggested methods for rock stress determination [101]. The technique measures only tangential stress near the surface of an excavation. As the flat jack stresses a greater mass of rock, the results tend to give a better average measurement than can be obtained using a smaller gauge length. This method can be used in ground where other methods are not suitable. To estimate the triaxial state of in-situ stress, at least six flat jack tests have to be carried out in independent directions.

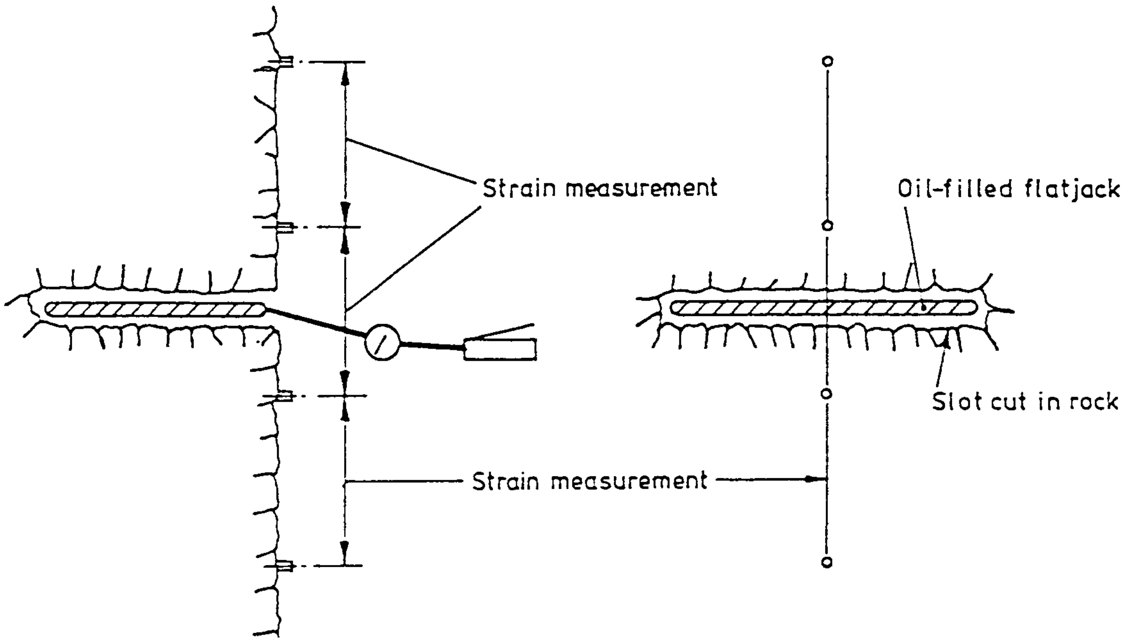

Figure 14c) — Measurement of in-situ stress — Flat jack equipment — Typical layout

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30.2.5 ***Static equilibrium method***

The static equilibrium method of in-situ stress determination [101] is based on the static equilibrium requirement that the total load on a sufficiently large area remains constant, even after an opening, such as mine drift, is made in the area. It does not require that the rock be elastic, homogeneous or isotropic.

30.2.6 ***Hydraulic fracturing technique***

Hydraulic fracturing [103] provides a determination of the maximum and minimum stresses in a plane perpendicular to the drillhole. The particular application of the technique has been in deep drillholes and requires no knowledge of the elastic properties of the rock. The technique is fully described in the ISRM suggested methods for rock stress determination [101].

30.3 Stress measurements in soils

30.3.1 ***General principles***

The analysis of the response of soil masses to applied loads requires reliable data on their strength and deformation characteristics, and, as these are stress-dependent, a knowledge of the in-situ state of stress assists in their evaluation by laboratory testing.

Direct in-situ measurements of the initial state of stress in soils is difficult because the disturbance created by gaining access to the ground mass is usually non-reversible, as well as being several times that produced by a stress-relieving technique. Most techniques that have been developed suffer from the disturbance that their instruments create in the ground on insertion.

It is usual to measure horizontal stress only and to make assumptions concerning the level of vertical stress from overburden depth. Total stress only may be measured, so to determine the effective stress conditions the pore water pressure at the test level has to be measured or assumed. Methods of determining pore water pressure in the field are discussed in **20.2**.

30.3.2 ***Hydraulic push-in pressure cells***

Measurement of total stress in soft to stiff clays has been achieved using thin rectangular hydraulic cells, "push-in spade shaped cells", carefully jacked into the ground [104]. The insertion of the cell into the ground generates a pressure in the cell which decays with time to a constant value equal to the total stress in the ground plus an over-read caused by the disturbance resulting from the insertion of the cell into the ground. The cell reading is adjusted for the over-read by a correction factor related to the undrained shear strength of the soil. Measurement of vertical and horizontal total stresses can also be achieved from vertical boreholes in soft clays using the BRE miniature push-in earth pressure cells [105].

30.3.3 ***Contact stress measurement***

The self-bored pressuremeter (see **25.7.2.2**) and Camkometer are instruments that reduce disturbance on insertion to a minimum by fully supporting the ground they penetrate. As pressure is applied to jack the cell into the ground, a cutting tool slowly rotates and gentle water flush removes surplus materials. Once installation has been completed the Camkometer electrical load cells measure the contact pressure, from which an estimate of total horizontal in-situ stress is obtained. The pressuremeter gives an estimate of the horizontal stress from the lift-off pressures of the membrane [71]. Facilities are available to measure pore water pressure with the same instruments. Ground conditions may limit the use of this technique.

30.3.4 Hydraulic fracturing

Hydraulic fracturing [106] may be used to estimate minimum total horizontal stresses in a deposit of normally consolidated clay. A length of borehole is sealed and a pumping-in test carried out. Pressure in the test zone is increased in increments until a sudden increase in water flow occurs, at which time it is assumed that tensile failure has occurred in the ground. The pressure at which failure takes place is related to the minimum in-situ stress by soil properties. Hydraulic fracturing is also used in rocks [103].

31 Bearing tests

31.1 Vertical loading tests

31.1.1 ***General principles***

In-situ vertical loading tests involve measuring the applied load and penetration of a plate being pushed into a soil or rock mass. The test can be carried out in shallow pits or trenches or at depth in the bottom of a borehole, pit or adit (see Figure 15). In soils, the test is carried out to determine the shear strength and deformation characteristics of the material beneath the loaded plate. The ultimate load is often unattainable in rocks, where the test is more frequently used to determine the deformation characteristics. The details of the test method for soils is given in BS 1377-9:1990, **4.1**, and that for rocks in the ISRM suggested method for determining in-situ deformability of rock [107].

The test is usually carried out either under a series of maintained loads or at a constant rate of penetration [108] [109]. In the former, the ground is allowed to consolidate under such a load before a further increment is applied; this yields the drained deformation characteristics and, if the test is continued to failure, also the strength characteristics. In the latter, the rate of penetration is often such that little or no drainage occurs, and the test gives the corresponding undrained deformation and strength characteristics.

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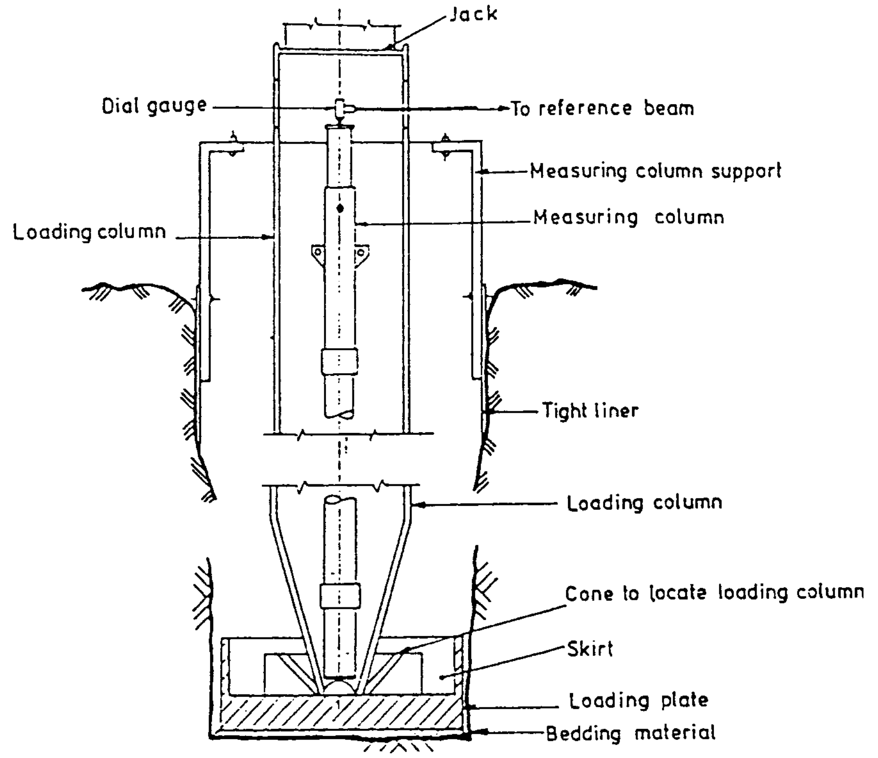
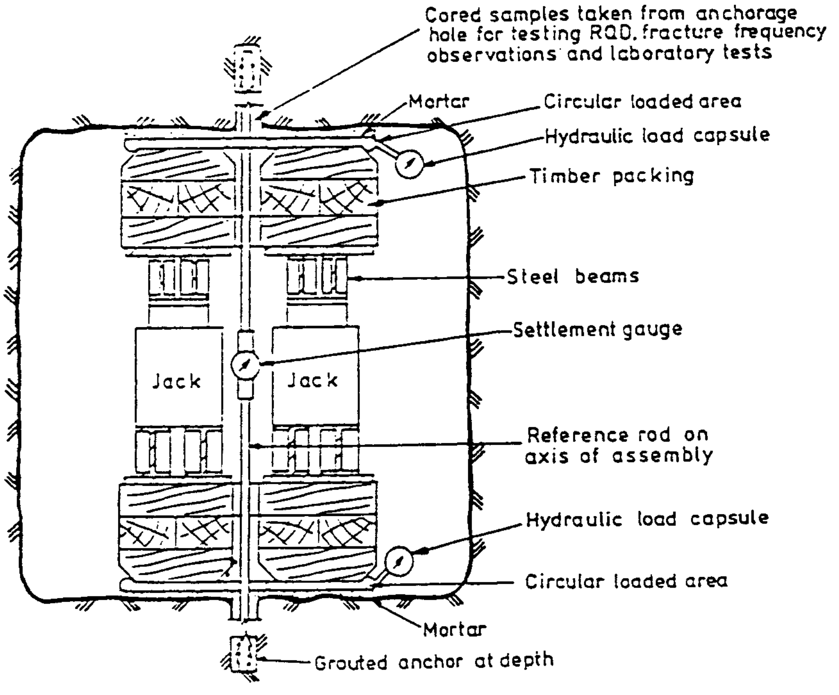


a) Plate test equipment for 864 mm diameter.



b) Jacking in adit-type of loading equipment

Figure 15 — Types of bearing test equipment

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It should be emphasized that the results of a single loading test apply only to the ground that is significantly stressed by the plate; this is typically a depth of about one and a half times the diameter or width of the plate. The depth of ground stressed by a structural foundation is usually far greater than that stressed by the loading test and, for this reason, the results of loading tests carried out at single elevation do not normally give a direct indication of the allowable bearing capacity and settlement characteristics of full-scale structural foundation. To determine the variation of ground properties with depth, it is often necessary to carry out a series of plate tests at different depths. These should be carried out such that each test subjects the ground to the same effective stress level it would receive at working load.

Where tests are carried out in rock, blasting for rock excavation may seriously affect the rock to be tested. This effect can be minimized by using small charges, and by finishing the excavation by hand methods.

31.1.2 ***Limitations of the test***

The main limitations of the test are the possibility of ground disturbance in the course of the excavation to gain access to the test position, plus the significant expense of the excavation.

Unavoidable changes in the ground stresses are caused by an excavation, which may produce irreversible effects on the properties the test is intended to study. For example, in stiff, fissured, over-consolidated clay, some swelling and expansion of the clay inevitably occurs during the setting-up process, due to the opening of fissures and other discontinuities; this can considerably reduce the values of the deformation moduli [110]. The moduli determined from plate tests are still more reliable, however, and often many times higher than those obtained from standard laboratory tests. In a project that involves a large excavation, e.g. a building with a deep and extensive basement, the excavation may cause disturbance to the ground beneath, with a consequent effect on the deformation characteristics. In such a case, it is necessary to allow for this unavoidable disturbance when interpreting the results of loading tests.

When carrying out the test below the prevailing ground water table, the seepage forces associated with dewatering may affect the properties to be measured. This effect is most severe for tests carried out at significant depths below the water table in soils and weak rocks. It may therefore be necessary to lower the water table by a system of wells set outside and below the test position.

The test is not frequently used to measure the ultimate bearing capacity. In cases where settlements and elastic deformation characteristics of the ground need to be determined, as in rock foundations, care should be exercised to work at stresses that are relevant. The observation of deformation, particularly at low stress levels, requires the utmost care in surface preparation and setting-up to achieve meaningful results. The errors that can be introduced by sample disturbance and inaccuracies of measurement can often be similar in size to the data sought or indeed larger.

The effect of sample disturbance can be reduced. To some extent, by carrying out preliminary cycles of loading and unloading. The maximum load in these cycles should not exceed the intended load. The rate of loading should be sufficiently rapid to prevent any significant consolidation or creep. After two or three cycles, the stress/settlement graph tends to become repeatable, and the test can then be extended to the main testing programme. The data from the preliminary load-cycles gives an indication of the effect of the sampling disturbance. The undrained deformation moduli, as measured after preliminary load-cycling, are normally a more reliable indication of the true properties of the undisturbed ground. Alternatively, displacement measuring systems can be installed in the ground beneath the plate, thus eliminating the effects of shallow disturbance [68] [108].

31.1.3 ***Interpretation of results***

The correct interpretation of the behaviour of the mass of ground under investigation requires a careful examination of the results, not only of the loading tests, but also of other data pertaining to the ground. Depending on the objects under investigation, such data might include the geological structure, the nature and distribution of discontinuities, lithology and the variability of the ground.

Several deformation moduli can be obtained from these tests, depending on the method used and the application. The results reflect the effects of the width and frequency of the discontinuities, and give an indication of the mass material behaviour under loading. The stress level at which these parameters should be examined depends on the working stress levels. In the case of tests on rock in adits, it may be necessary to consider the in-situ stresses in the test sample.

The moduli to be used for design purposes should be those relating to the ground both at the time of construction and after it has been affected by the construction procedures; e.g. a deep excavation might affect the deformation moduli of a soil, and blasting may affect the properties of a rock. Sometimes, the effect of a construction procedure may be sufficiently severe to justify the examination of alternative methods of construction.

On completion of the testing, full identification of the material (see section 6) beneath the loaded area should be carried out by sampling and testing in the laboratory (see sections 3 and 5). In many cases results obtained from these tests assist in extrapolating the test results to other areas on the site.

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31.2 Lateral and inclined loading tests

Lateral and inclined loading tests are essentially the same as vertical loading tests, and are carried out and analysed in a comparable way. Particular characteristics of the ground can be investigated by loading tests at a preferred orientation. These can be carried out in rock for investigations concerning tunnels and underground excavations. A simple lateral loading test, using an hydraulic jack between the opposite sides of a trial pit, forms a very convenient means of measuring in-situ the shear strength of soils. It is often used in soils that are not suitable for undisturbed sampling, e.g. clays containing stones.

31.3 Pressurized chamber tests

The test is carried out in an underground excavation or length of tunnel and gives the deformation moduli of the surrounding ground. It consists of charging the chamber with water under various pressures. The test is normally carried out for projects involving tunnels carrying water under pressure.

The test site should preferably form part of the actual excavation, or be of the same size and parallel to the axis in representative ground. The length of a test section should be at least five times the excavated diameter, unless allowance can be made for the end effects, and the method of excavation used selected to obtain a ground surface of similar quality to the actual excavation. In order to know whether the type of modulus is drained, partially drained or undrained, it is necessary to know which drainage conditions applied during the test.

31.4 In-situ California Bearing Ratio (CBR) test

31.4.1 *General*

The CBR method of flexible pavement design is essentially an empirical method in which design curves are used to estimate pavement thickness appropriate to the CBR of the soil. There is no unique CBR of a soil and the value obtained in any test depends very much on the manner in which the test is conducted. The design curves are usually based on one carefully specified method of measuring the CBR, which is usually a laboratory method. The parameter required for the design of flexible pavements is the CBR attained by the soil at formation level after all necessary compaction has been carried out, the pavement has been laid, and sufficient time has been allowed to elapse for equilibrium moisture content to become established. Before embarking on in-situ CBR tests, it is therefore necessary to consider carefully how relevant these are to the design method to be used and whether the condition of equilibrium moisture content is likely to pertain.

31.4.2 *Method of carrying out test*

The test is carried out by the method described in test 16 of BS 1377-9:1990, **4.3**.

31.4.3 *Limitations and uses of test*

The test is unsuitable for any soil containing particles of longest dimension greater than 20 mm, because the seating of the plunger on a large stone may lead to an unrepresentative result. The test is of dubious value with sands, because it tends to give results much lower than the laboratory tests on which the design charts are based. This is because of the confining effect of the mould in the laboratory tests. The test is most suited to clay soils, subject always to the soil under test being at equilibrium moisture content. The moisture content at a depth of 1 m to 2 m below ground surface, where the soil is normally unaffected by seasonal changes in moisture content, often gives a good indication of the equilibrium moisture content, provided that there is no change of strata. An alternative is to carry out the test directly beneath an existing pavement that has identical soil conditions to those of the proposed construction; this method has been used with some success for the design of airfield pavements. In-situ CBR tests have sometimes been carried out in conjunction with in-situ density and moisture content tests and then linked with laboratory compaction tests. A judicious study of all the resulting data leads to a reasonable design parameter on suitable soils. Attempts have sometimes been made to use the test as a means of controlling the compaction of fill or natural formations, but they have not usually been successful and the procedure cannot be recommended.

32 In-situ shear tests

32.1 General principles of the direct shear test

For this test, a sample of ground, prepared and tested in-situ, is subjected to direct shear, using a stress system similar to that of the laboratory shear box tests (see Figure 16). The samples are selected to include one or more discontinuities, if this is what is to be tested. The orientation of the discontinuities should be selected as relevant to the stress conditions being considered. The maximum sample size is often limited by practical considerations of loading and accessibility. The test is normally designed to measure the peak shear strength of the in-situ material as a function of the stress normal to the sheared plane [111]. More than one test is normally required to obtain a sensible design value. The measurement of residual shear strength presents major practical problems in arranging for a sufficiently long travel, but a useful indication of residual strength may be obtained by continuing the test to the limits of travel of the apparatus. In certain applications, the test may be designed to establish the strength of the interface between concrete and rock or soil.

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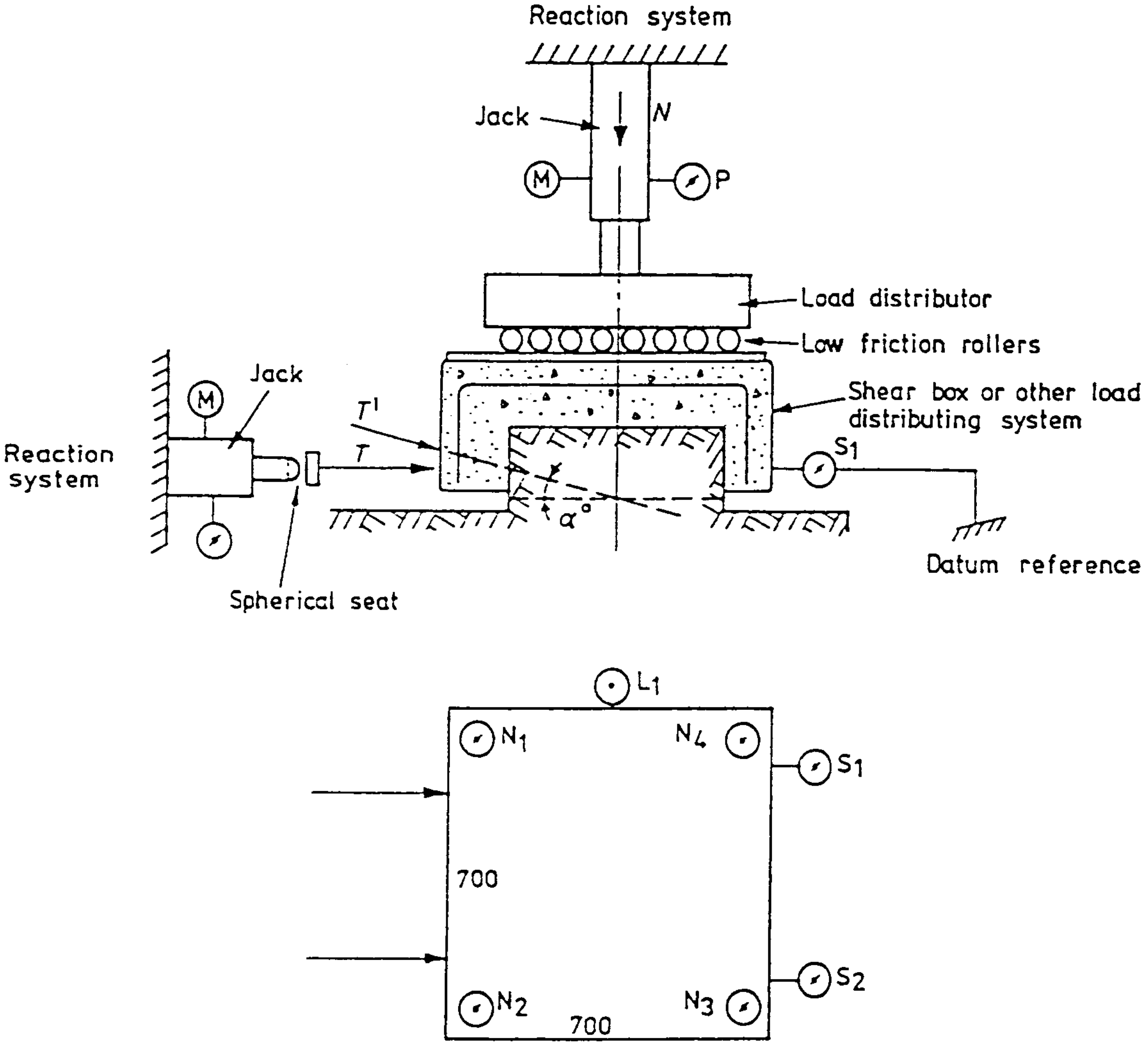
The orientation of the sample and the forces applied to it are usually governed by the direction of the forces that become effective during and on completion of the works, but modified to take account of the orientation of significant discontinuities. In many cases, however, to facilitate the setting-up of the test, the sample is prepared with the shear plane horizontal. The normal and shearing stresses are imposed as forces applied normally and along the shear plane. However, an inclined shear force passing through the centre of the shear plane may be preferred as this tends to produce a more uniform distribution of stress on the shear surface [111].

Field shear testing of intact soil may be necessary sometimes. Although it is theoretically possible to carry these out in the consolidated, unconsolidated drained or undrained state, in practice it is not usually possible to prevent some drainage. Such testing is not likely to be necessary on intact rocks, with the possible exception of weak rocks.



N_1 Normal deflection gauges
S_1 Shear deflection gauges
L_1 Lateral deflection gauges
P Pressure gauges
T Shearing force
N Normal force
M Load maintenance

Figure 16 — Equipment layout for shear and sliding friction test on rock or soil samples

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32.2 Comments on in-situ shear test

32.2.1 *General*

The in-situ general test in rock is described fully in the International Society of Rock Mechanics (ISRM) suggested method [111]. In-situ shear tests in soil are described in [112]. In addition to these references the following comments are pertinent. Samples are normally prepared at the bottom of pits or trenches in soil. Adits are more common for rock testing. The excavation permits access to the material at the zone of interest and in many cases provides a suitable means of setting up the reaction for the applied forces.

As a rough guide, the sample dimension should be at least ten times that of the largest particle; in rock, the sample size should reflect the roughness of the rock discontinuity being tested. For stronger rocks, the sample can be rendered with suitably strong cement and reinforced concrete to ensure adequate load distribution. The equipment should be of robust construction. Samples between 600 mm and 1 500 mm square have been used for testing soil and weak rocks. Larger samples may be required in ground containing boulders or in compacted fill material.

Great care has to be exercised in preserving the environmental conditions when carrying out the excavation. Excavation techniques that give rise to crumbling, fracturing or excessive dynamic shock loading, which would affect the discontinuities in the sample test area, should be avoided. Hand sawing, cutting and diamond drilling should be used to prepare and trim the sample. Adequate protection from the elements should be provided. Final exposure and trimming of the sample to fit the loading frame and the testing should all be completed with the minimum of delay to avoid possible significant changes in the moisture and stress conditions of the sample. Where tests are carried out below the water table, precautions should be taken to avoid the effects of water pressure and seepage (see **31.1.2**).

Where it is intended to test one discontinuity only, care has to be taken to avoid disturbance to the surface of the discontinuity and to prepare the sample so that the forces are applied in the plane of the discontinuity in the manner intended. The spatial orientation of the discontinuity should be defined by its dip and strike.

Where drained conditions are required, suitable drainage layers can be inserted around the sample and on the loaded upper surface.

32.2.2 *Test arrangement*

An example of a typical test layout for determining the peak shear strength is shown in Figure 16. In addition to the comments made elsewhere, a porous piston or other suitable medium can be used to distribute the load where drained conditions are required. The alignment of the force needs to be maintained during the test.

If a constant normal load is required for this type or test, a suitable reduction has to be made from the applied normal load during testing, to compensate for the increase in the vertical component with increasing shear force. The shear force application can be developed by similar means to the normal loading. In both cases, care has to be taken to ensure that the ground reaction does not extend to the sample. The reaction system can frequently be provided by the excavation sidewalls. In certain cases, it may be necessary to provide the shear force by traction on a system anchored by piles or anchored cable. Sufficient travel should be provided to run the complete test.

32.2.3 *Method of carrying out the test*

The forces used for the testing programme should be in the range of the working stresses to be applied by the structure. The peak force and an estimate of the residual direct shear strengths should also be determined to establish a factor of safety.

When testing a single fissure or joint, care should be taken to establish the initial slope of the fitted line at the lower normal stresses. There is sometimes a tendency for low values to be operative for the lower normal stresses until the asperities on the joint surface interlock. Photographs of the shear surface form a useful record.

Extrapolation of results obtained on single joints should not be attempted without due confirmation that the joint surface tested is representative of the planeness and roughness of the joints in the mass (see section 6). If not, appropriate calculations for extrapolation can be used [113] [114] [115].

Where failure occurs in a plane that dips at an angle to the applied shearing force, the analysis given in [116] may be used.

On completion of the test, full identification of the material and that immediately surrounding the sample should be carried out by sampling, visual examination, and laboratory testing. Tests on relatively small samples of single joints may give useful values of angle of shearing resistance, but the cohesion parameter tends to be size dependent.

33 Large-scale field trials

33.1 Introduction

Large-scale field trials are carried out in such a manner that the ground is tested on a scale and under conditions comparable with those prevailing in the project under investigation. Such trials, however, are likely to be costly in terms of instrumentation, technical support for the co-ordination of the results and requirements of purpose-made equipment. Some of the methods and types of instrumentation available for monitoring field tests are outlined briefly in the following clauses, together with some of the more common large-scale field trials used to obtain geotechnical data for design and construction.

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Several of the trials would include the use of tests described in this section 4 and in section 3, and involve the principles of site investigation embodied in the code. Large-scale field trials are not standard tests, and should be designed to suit the individual requirements of the proposed works and the particular ground on which or within which these are to be performed. On large projects, field trials can provide the necessary design parameters and valuable construction data on excavation, handling and placing, resulting in considerable savings and enhanced safety. Such methods and trials can usefully be extended to the construction stage and to monitoring the interrelated response of the ground and structure after completion under the working conditions.

33.2 Methods of instrumentation

Numerous techniques are now available to be used in ground investigations to monitor movements and strains, total stresses and pore water pressures associated with known or suspected ground mass movements. Such movements can result from construction processes, potential stability failures, tunnelling, subsidence and ground response in large scale field trials. The types, advantages, limitations and appropriateness of the various techniques are discussed in detail elsewhere [117] [118] [119] [120].

Ground movements are normally associated with stress redistribution and pore pressure changes which are characteristic of the particular ground.

Total stress can be monitored using:

— hydraulic cells;

— vibrating-wire cells;

— push-in type cells (see **30.3.2**);

— interface pressure measurements.

The techniques for the measurement of pore pressure response are covered in **23.2.3** to **23.2.6**.

Ground movements are normally measured in terms of the displacement of points which can be positioned on the surface of the ground or within the ground mass. The absolute movement of a point has to be referred to a stable datum, and sufficient measurements taken to define movement in three dimensions if this is required. This relative movement between displacement of points can be used to obtain strain.

Surface movements can be measured using:

— precise levelling [121];

— surveying;

— Electronic Distance Measurement (EDM);

— photogrammetric methods;

— global positioning systems.

An accuracy of ±0.5 mm can be achieved with precise levelling and ±3 ppm for distance measurements over 2 000 m can be achieved using EDM instruments. Care has to be taken to position datum points away from the effects of movements due to load and water changes.

Internal movements or displacements can be measured in boreholes or by direct placement of instruments within fill. Techniques for vertical movements include:

a) the use of extensometers and settlement gauges:

— magnet;

— plate;

— rod;

— liquid settlement gauges.

b) lateral or horizontal movements measured by:

— inclinometers;

— electrolevels [122];

— inverted pendulums;

— magnet plate gauges.

Automatic data-logging of many types of instruments is now possible but it is essential for great care to be exercised when maintaining and assessing the performance of the instruments and the quality of the data recorded.

The limitations of any instrumentation should always be kept in mind, particularly for long term measurements. Electrical instruments are more likely to fail than mechanical ones, which are simple and robust. All measurements, however, are usually discontinuous in space and time. Data from instrumentation is not comprehensive and may not record the most adverse circumstances. Instrumentation may malfunction and give no measurements or, worse, give erroneous results. The installation of instrumentation may affect soil behaviour, particularly with respect to earth pressure measurements.

33.3 Trial embankments and excavations

The construction of trial embankments may serve a threefold purpose: the quality and compaction characteristics of available borrow material can be determined on the field scale and compared with laboratory test results; the characteristics and performance of placing and compacting equipment can be investigated; and the strength and settlement characteristics of the ground on which the embankment is placed can be examined. A trial bank may be constructed in such a way that, where failure is of no consequence, it can be induced deliberately, either in the embankment alone or in the embankment and the foundations. Such failures sometimes occur in an unexpected manner, and the engineer should take precautions to ensure that no injury to persons or unexpected damage is caused; even so, some installed instrumentation may be destroyed. The value of such a failure is that back analysis (see clause **34**), can be used to check strength parameters.

Compaction trials can include experiments using differing borrow pit materials, layer thickness, amounts of watering and amounts of work

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performed in compaction. Measurements should be taken of in-situ density and water content and comparisons made both with laboratory compaction tests to obtain a specification standard, and with in-situ borrow pit densities, so that the degree of bulking or volume reduction can be estimated for given quantities (see BS 6031). Trials of equipment can also be undertaken. Care should be taken not to vary too many factors at the same time, otherwise the effects of variation cannot be estimated.

Trial excavations yield information on the material excavated and the performance of excavating equipment, and they also permit more detailed examination of the ground than is possible from borehole samples. Excavations can sometimes be used to test the short-term stability of excavated slopes. Trial excavations may be constructed deliberately to fail. However, failure in excavation, especially if deep, is correspondingly more dangerous than failure of fills, and increased vigilance is needed. Trial excavations also enable the response of the ground and groundwater to excavation to be measured.

Adequate instrumentation to trial embankments or excavations is essential, together with continuous observation (see **33.2**), if the maximum information is to be gained. The scale of trial embankments or excavations needs careful consideration. Clearly, the more closely the size of the trial approaches that of the prototype, the more directly applicable are the results obtained from the trial.

33.4 Construction trials

In many projects, considerable value can be derived from trials carried out before the commencement of the permanent works. Such trials permit the evaluation of the procedures to be adopted and the effectiveness of the various expedients. As with all large-scale testing, a prior knowledge of the characteristics of the ground is essential. The results of the trial give an assessment of the properties of the ground and often enable the results to be correlated with those obtained from routine ground investigation methods.

A wide range of expedients is commonly tested in trials. Examples include: construction methods, such as pile tests; ground anchor tests; compaction tests for earthworks, experimental shafts and adits for tunnels; and construction methods such as grouting trials, trial blasts for explosives and dewatering trials.

34 Back analysis of full-scale performance

Natural or man-made conditions on a site sometimes produce phenomena that can be used to assess parameters which are otherwise difficult to assess, or that can be used to check the validity of parameters measured in the laboratory. Examples of such phenomena are slope failure and settlement of a structure. It may be possible, starting from the observed phenomena, to perform a back analysis and, in the case of a slope failure, to arrive at shear strength parameters that fit the observed facts [123]. Back analysis of settlements is also possible, but care is required in assessing actual loadings and the times when they have taken place. For a back analysis to be effective, it should be accompanied by a full investigation to determine the ground and groundwater conditions. The development of finite element techniques has greatly improved the ability to back analyse more complicated geotechnical structures.

35 Geophysical surveying

35.1 Introduction

There are many different geophysical techniques, each based on different theoretical principles, such as seismic velocity or electrical resistivity, and consequently producing different sets of information relating to the properties of subsurface materials. For any given geophysical technique the variations in the information obtained can give an indication of the geological structure. Invariably interpretation of geophysical survey data involves some degree of prior knowledge of the underlying geological structure derived from the preliminary reconnaissance and from boreholes. For optimum interpretation of the data from a geophysical survey it is essential that adequate direct control is available, such as boreholes or trial pits.

In comparison with borehole investigations, geophysical surveys can offer considerable savings in both time and money. In order to assist the subsequent borehole investigation it may be advantageous to undertake a reconnaissance geophysical survey at an early stage in a site investigation. This is done by identifying areas of the site where anomalous geophysical data are obtained, and which should be investigated by drilling. On sites where contamination is suspected, a geophysical survey may form part of a preliminary risk assessment prior to drilling or sampling. During the drilling programme on the site, geophysical surveys may be used to check the interpretation of the geological structure between the boreholes. Later in the site investigation further geophysical surveys may be carried out within and between the boreholes and on the ground surface; these are to determine the geological, hydrogeological and geotechnical properties of the ground mass in which the engineering construction is taking place.

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The application of geophysical techniques to the solution of engineering problems has sometimes been disappointing, either because a method was used that lacked the precision required in a particular site investigation or because a method was specified that was inappropriate to the problem under consideration. In some cases these problems could have been avoided by taking expert advice before initiating the survey, but in other cases it should be emphasized that the geological conditions at the site are far more complex than anticipated at the planning stage of the geophysical survey and therefore interpretation of the geophysical data by the geophysicist does not yield the information expected by the engineer. It is often advisable to undertake a feasibility study at the field site to assess the suitability of the proposed geophysical techniques for the investigation of the geological problem. A detailed consideration of the use of geophysical techniques in civil engineering applications is given in [124].

The implementation and interpretation of a geophysical survey requires expert personnel and would normally be entrusted to an organization specializing in this work. The consulting engineer, however, should also have access to an independent geophysical advisor who, in addition to advising on the suitability of the various geophysical techniques available, can design and oversee the geophysical survey for his client. Early consideration should be given to the applicability of geophysical methods to the various aspects of the site investigation programme, for example, to assist in the siting of boreholes, trial pits, etc. It is important to realise that the choice of the correct geophysical method, or combination of methods is absolutely essential, both from the point of view of maximizing the success of the survey and obtaining the desired geological information on a cost-effective basis.

Once the geophysical data have been obtained it is possible to compute a model of the geological structure, which gives a realistic correlation with the data. The best overall model is synthesized from all available geological information from boreholes and field mapping. Without this input of precise information the model cannot be constrained or evaluated in practical terms. It follows that close collaboration between the site geologists, engineers and geophysicists in the interpretation of the geophysical data is essential.

The performance of all geophysical methods used in site investigation is influenced by four fundamental controlling factors:

a) depth penetration;

b) vertical and lateral resolution;

c) signal-to-noise ratio;

d) contrast in physical properties.

All four factors are intimately linked and are particularly relevant in the field of engineering geophysics, where the small-scale nature of the engineering site puts increased demands on the accuracy of the final interpretation. For example, with the current range of operational equipment some geophysical methods, such as ground probing radar, have limited penetration into the ground, but can achieve excellent resolution of the near-surface geological structures when significant penetration is achieved. Excessive environmental noise can be a problem when carrying out seismic and electromagnetic surveys, for instance, and if the signal-to-noise ratio is too low the required signal may not be observed within the ambient noise, although the use of signal enhancement techniques has considerably reduced this problem. Most geophysical methods used in site investigation relate to the location of a contrast in the physical properties of the materials under investigation, such as the distinct change in seismic velocity at the boundary between two geological strata, or a difference between the magnetic properties of the material in a disused mineshaft and those of the surrounding ground.

In the study of geological structures, the best results are obtained when the geological conditions are uniform and simple with large clear-cut contrasts in the relevant physical properties of the formations, for example, strata on either side of a fault. It should be noted that there is often a transition zone at a boundary, which can lead to a margin of uncertainty in the interpretation of the geophysical data when related to the engineering or geological boundary; for example, in a survey to determine the depth to competent rock, the results may be complicated by the presence of a weathered layer, or overlying boulders. In the location of geological hazards, such as natural cavities or buried mineshafts, geophysical surveying techniques are at their most productive when the survey lines are laid out on a rectilinear grid, or when there is constant line separation, so that the area of anomalous ground conditions can be rapidly identified by comparison with a survey line passing over a known geological structure in the survey area.

There has been a steady improvement in the performance of the basic geophysical surveying techniques over the past decade. It is now possible to carry out on-site interpretation of geophysical data in the field and to adjust the survey layout to improve the final product as a result. Because of the accelerated development of computer technology, this has led to the development of high quality equipment, accompanied by new capabilities in the acquisition, recording and processing of geophysical data. The increased capability for numerical modelling should be balanced by an awareness that the quality of raw data has to be such as to warrant subsequent manipulations. There is a role for an independent skilled person overseeing the acquisition phase who can make some kind of quality assessment, especially if the data are to be processed further prior to interpretation. This may help to prevent a time-consuming and expensive investigation of "system artefacts".

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35.2 Objectives

Geophysical techniques can be used on land, at sea, and in the air. In each case basic techniques are modified, but the same physical properties are involved irrespective of the environment.

The four primary objectives in the use of engineering geophysical surveys are listed in Table 6. These are as follows.

a) Geological investigation: geophysical methods have a major role to play in mapping geological boundaries between layers; determining the thickness of superficial deposits and depth to "engineering rockhead"; establishing weathering profiles; and the study of particular erosional and structural features, such as the location of buried channels, faults, dykes, etc.

b) Resource assessment: location of aquifers and determination of water quality; exploration of sand and gravel deposits and rocks for aggregate; identification of clay deposits, etc.

c) Hazard assessment: detection of voids and buried artefacts; location of buried mineshafts and adits, natural cavities, old foundations, pipelines etc.; detection of leaks in barriers; pollution plumes on landfill sites.

d) Determination of engineering properties of the ground, such as dynamic elastic moduli, rock rippability and rock quality; soil corrosivity for pipeline protection studies etc.

All of the engineering geophysical applications mentioned in Table 6 are considered in some detail in [124]. Table 7 is also published in [124] and gives a usefulness rating to the geophysical techniques considered below in various geotechnical applications. Further details of the various geophysical methods, to which reference is made in this section, can be found in standard textbooks, including Telford et al. [125], Dobrin and Savit [126], Keary and Brooks [127], Milsom [128], Parasnis [129] and Reynolds [130].

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Table 6 — Geophysical methods in ground investigation

Problem		Example	Methods and remarks
Geological	Stratigraphical	Sediments over bedrock:	*Land*
		i) Sands and gravel over bedrock, water table low in sands and gravels	Seismic refraction
		ii) Sands and gravels overlying clay, water table high in sands and gravels	Resistivity
		iii) Clay over bedrock	Resistivity or seismic refraction
		Sediments over bedrock normally	*Marine*. Continuous seismic reflection profiling
	Erosional (for caverns, see "Shafts..." below)	Buried channel	Seismic refraction Resistivity for feature wider than depth of cover
		Buried karstic surface	Resistivity contouring
	Structural	Buried faults, dykes	Resistivity contouring Seismic reflection or refraction Magnetic and gravimetric (large faults)
Resources buried	Water	Location of aquifer Location of saline/potable interface	Resistivity and seismic refraction
	Sand and gravel	Sand, gravel over clay	*Land*. Resistivity
		Gravel banks	*Marine*. Continuous seismic profiling, side scan sonar, echo sounding
	Rock	Intrusive in sedimentary rocks	Magnetic
	Clay	Clay pockets	Resistivity (weathering may give low resistivity)
Engineering parameters	Modulus of elasticity, density and porosity	Dynamic deformation modulus	Seismic velocity at surface, or with single or multiple boreholes. (Cross hole transmission.) Borehole geophysics
	Depths of piles	Check on effects of ground treatment	
	Rock rippability	Choice of excavation method	Seismic
	Corrosivity of soils	Pipeline surveys	Surface resistivity. Redox potential
Buried artefacts	Cables	Trenches on land	Magnetometer Electromagnetic field detectors
	Pipes	Submarine trenches	Echo sounding, side scan sonar
		Submarine pipelines	Side scan sonar, magnetic, continuous seismic profiling (especially if thought to be partially buried) with high frequency pinger
	Shafts, adits and caverns	Shaft, sink holes, mine workings	Resistivity. Magnetics, electromagnetic, radar; infra-red air photography on clear areas. Cross hole seismic measurements. Detailed gravity for large systems
	Archaeological remains	Foundations, buried wall, crypts	Magnetic, electromagnetic resistivity and radar

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Table 7 — Usefulness of engineering geophysical methods

Geophysical method	Geotechnical applications									
	Depth to bedrock	Stratigraphy	Lithology	Fractured zones	Fault displacements	Dynamic elastic moduli	Density	Rippability	Cavity detection	Buried artefacts
Seismic										
— Refraction	4	4	3	3	4	3	2	4	1	1
— Reflection – land	2	2	2	1	2	0	0	0	2	1
— Reflection – marine	4	4	2	2	4	0	0	1	0	2
— Cross-hole	2	2	3	3	1	4	2	2	3	2
Electrical										
— Resistivity sounding	4	3	3	2	2	0	0	1	2	1
— Induced polarization (IP)	2	2	3	1	0	0	0	0	0	0
— Electromagnetic (EM) and resistivity profiling	3	2	2	4	1	0	0	0	3	3
Other										
— Ground probing radar	2	3	1	2	3	0	0	0	3	4
— Gravity	1	0	0	0	2	0	2	0	2	1
— Magnetic	0	0	0	0	2	0	0	0	2	3
Borehole										
— Self-potential	2	4	4	1	1	0	0	0	1	1
— Single point resistance	2	4	4	0	0	0	0	0	0	0
— Long and short normal, and lateral resistivity	2	4	4	0	0	0	0	0	0	0
— Natural gamma	2	4	4	0	0	0	0	0	0	0
— Gamma-gamma	3A	4	4	0	0	0	3A	0	0	0
— Neutron	2A	4	4	0	0	0	3A	0	0	0
— Fluid conductivity	0	1	0	0	0	0	1	0	2	0
— Fluid temperature	0	0	0	1	0	0	0	0	1	0
— Sonce (velocity)	3	4	2	3	0	3	2	1	2	0

KEY

0 = Not considered applicable; 1 = limited use; 2 = used, or could be used, but not best approach, or has limitations; 3 = excellent potential but not fully developed; 4 = generally considered an excellent approach, techniques well developed;

A = in conjunction with other electrical or nuclear logs.

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Table 7 — Usefulness of engineering geophysical methods *(continued)*

Geophysical method	Geotechnical applications									
	Ground water exploration	**Water quality**	**Porosity**	**Permeability**	**Tempratur e**	**Flow rate and/or direcdtion**	**Buried channels**	**Clay pockets in limestone**	**Sand and gravel**	**Basic igneous dykes**
Seismic										
— Refraction	2	0	0	0	0	0	4	1	2	1
— Reflection - land	2	0	0	0	0	0	1	0	0	1
— Reflection - marine	0	0	0	0	0	0	4	0	0	0
— Cross-hole	0	0	0	0	0	0	2	0	1	2
Electrical										
— Resistivity sounding	4	4	3	1	0	0	3	0	3	0
— Induced polarization (IP)	3	1	3	2	0	0	2	1	1	1
— Electromagnetic (EM) and resistivity profiling	4	4	1	0	0	0	3	4	3	3
Other										
— Ground probing radar	2	2	1	0	0	0	2	2	1	2
— Gravity	1	0	0	0	0	0	2	1	1	2
— Magnetic	0	0	0	0	0	0	1	3	0	4
Borehole										
— Self-potential	4	2	0	0	0	0	0	0	0	0
— Single point resistance	4	2	1	0	0	0	0	0	0	0
— Long and short normal, and lateral resistivity	4	2	4	0	0	0	0	0	0	0
— Natural gamma	2A	2	1A	3A	0	0	0	0	0	0
— Gamma-gamma	2A	0	3A	2A	0	0	0	0	0	0
— Neutron	3A	0	3A	2	0	0	0	0	0	0
— Fluid conductivity	4	4	4	1	0	0	0	0	0	0
— Fluid temperature	2	3	0	0	4	2	0	0	0	0
— Sonce (velocity)	1	0	1	0	0	0	0	0	0	0

KEY

0 = Not considered applicable; 1 = limited use; 2 = used, or could be used, but not best approach, or has limitations; 3 = excellent potential but not fully developed; 4 = generally considered an excellent approach, techniques well developed;

A = in conjunction with other electrical or nuclear logs.

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35.3 Land geophysics

35.3.1 *Electrical resistivity*

An electrical current is passed into the ground surface between a pair of electrodes and the potential difference between two similar electrodes is measured. From these two measurements a value of electrical resistivity can be calculated. By then changing the electrode spacing the variation of electrical resistivity with depth can be determined and related to changes in the geological strata. The Wenner or Schlumberger arrays are the most commonly used configurations for deploying the electrodes. The method may be used to provide information on the geo-electrical stratification of the ground (soundings or depth probe), lateral changes in resistivity (constant separation traversing) or local anomalous areas associated with cavities or buried mineshafts (equipotential surveys). Computer-controlled multiple electrode techniques are now being developed to produce an electrical image (or tomogram) of the geological strata beneath the array. Using standard iterative modelling techniques, the geological section can be adjusted until it gives rise to an electrical image similar to the one produced from the field data. Due attention should be given to the effects of lateral variations on sounding measurements. The best results are obtained if soundings are positioned parallel to the geological strike and away from services such as water mains, metal fences and power lines. Soundings measured along the sides of roads usually produce poor results.The interpretation of resistivity soundings does not always provide a definite solution, because the theoretical models used to fit the data assume horizontal layering of the geological strata, which may not always be a realistic assumption. There is also the inherent problem of ambiguity where a number of models fit the resistivity data obtained, so it is essential that the interpretation should be based on a model that is consistent with the known geology derived from borehole control.

35.3.2 *Gravity*

The gravity method involves the measurement of variations in the gravity field of the earth caused by local differences in the density of the subsurface rocks. The technique is normally associated with large scale regional geophysical surveys investigating the geological structure to considerable depth. In engineering surveys, the method has found some application in the location of large fault zones, or the rock face in infilled quarries. The technique is also suitable for studies of bedrock depths in excess of 50 m in areas of low topography, e.g. in the delineation of deep sediment-filled valleys. A more recent trend has been the use of microgravity surveys in the detection of natural cavities and mine shafts. While the gravity method is expensive because of the variety of corrections that have to be applied to the observed data, at times it may well be the only geophysical method that is applicable in an urban environment. It should be noted, however, that significant terrain corrections may have to be made for local anomalies in built-up areas.

35.3.3 *Magnetic*

The magnetic method involves the measurement of variations in the magnetic field of the earth caused by local differences in the magnetization of the subsurface rocks. A magnetic survey is rapid and easy to carry out and a site can be surveyed with a close grid spacing (often 1 m) at low cost. The only correction required to the observed data is the subtraction of the diurnal variation, which is usually continuously recorded throughout the survey period. This is not required if a magnetic gradiometer survey is carried out, since this measures the gradient of the vertical magnetic field using two magnetometer sensors separated by a fixed distance in the vertical plane and recording simultaneously. When a site has a long urban or industrial history it is invariably littered with ferrous debris which may prevent the location of the main magnetic anomaly. Some attention should be paid to the possible effects of magnetic objects carried by the operator on the local magnetic field.

The method currently finds its greatest application in the location of buried mineshafts and adits. It is also widely used in the location of buried metalliferous man-made objects, such as cables or pipelines and, in the marine environment, sunken vessels. In the geological field, the method may locate boundaries between rocks that display magnetic contrasts, such as faults or dykes.

35.3.4 *Seismic refraction*

The seismic refraction method involves the recording of the primary wave, alternatively known as the compressional or P wave travelling both directly through surface layers and refracted along underlying layers of higher seismic velocity. A variety of seismic sources is available, ranging from the hammer and falling weight to detonators or explosives. Interpretation of the data provides layer thicknesses and seismic velocities, but it should be noted that if a low velocity layer is overlaid by a high velocity layer in the geological sequence then a misleading interpretation and incorrect depth determination may result. The method is best used to provide detailed information along a line where the geology is not too complex or where the lithological or structural variation in the bedrock is of greater interest than variations in the overburden. In this situation the technique provides data efficiently, although it is relatively expensive if explosives have to be employed. As mentioned previously, the presence of significant ambient noise, such as that generated by a busy road, may inhibit the use of the method. A study of secondary or shear (S) wave refraction data is carried out if information on the in-situ dynamic elastic properties of the bedrock is required. The method is widely used to determine

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the depth to bedrock, particularly in site investigation for roads, dam sites and tunnels. It is also applicable to the assessment of rock mass rippability based on the published tables of the Caterpillar Tractor Company [131]. Calibration of seismic velocity data by laboratory tests on borehole core is also recommended.

35.3.5 ***Seismic reflection***

Seismic reflection sections in the depth range 10–50 m have been obtained using standard 12- and 24-channel signal enhancement seismographs and hammer or similar seismic sources. Good reflections can be obtained if the bedrock offers a marked contrast in acoustic impedance with the overlying superficial material, but this is an idealized case. Resolution, and hence the shallower limit to useful data is, at present, limited by dominant source frequencies of approximately 100 Hz. Although penetration is to an order of magnitude greater than can be easily obtained by the more conventional seismic refraction method, which uses a hammer source, the technique is not in routine use. One of the main reasons for this is the difficulty in identifying low energy reflection arrivals in the part of the seismic record that also often includes strong refraction and surface wave arrivals. Research is continuing, however, into the development of high frequency seismic sources and it seems likely that the shallow seismic reflection method will be in common use in site investigation in the future. It should be noted that deeper reflection techniques using surface vibrators and sophisticated correlation processing have already been used in conjunction with special engineering studies for deep radioactive waste repositories and for gas storage.

35.3.6 ***Electromagnetic***

There has been a rapid increase in the use of electromagnetic surveying techniques for site investigation over the past decade. These range from the very low frequency (VLF) system to ground probing radar:

a) Very Low Frequency (VLF)

The VLF method is based on the transmission of electromagnetic waves from distant, very low frequency radio stations (10 kHz to 30 kHz), which can be used to determine the electrical properties of the earth.

The VLF system can be operated both on the ground and from the air but has limited application in determining the electrical conductivity of near-surface materials. It may have some applications in cavity and mineshaft location where fairly low resistivity material overlies a resistive bedrock. The location of fluid-filled or mineralized fault zones is another area where this method has a useful application.

b) Ground electrical conductivity

In this method a transmitter coil is energized with an alternating current and is placed on or above the ground surface. The electromagnetic field from the transmitter coil induces very small currents in the earth. These currents generate a secondary magnetic field, which is sensed, together with the primary field by the receiver coil. The intercoil spacing and operating frequency are chosen so that the ratio of the secondary to primary magnetic field is linearly proportional to the apparent ground conductivity. This ratio is measured and a direct reading of apparent ground conductivity is obtained.

Using a fixed separation of 4 m between the transmitter and receiver coils the depth of penetration is limited to less than 6 m but the survey can be carried out in a rapid and economic manner by a single operator. Greater penetration to around 60 m is achieved by moving the two coils apart to a maximum distance of 40 m or by reorienting the coils.

It is important to realise that a ground conductivity survey does not supply the quantitative information on earth layering that can be obtained by resistivity sounding (**35.3.1**) or seismic refraction (**35.3.4**) surveys. As the technique is so cost effective, though, it can be considered for the provision of rapid reconnaissance surveys prior to drilling or for filling gaps between boreholes or resistivity soundings. The constant separation equipment is particularly effective in the location of cavities or buried mineshafts when used in conjunction with a magnetic survey. The measurements compare very closely with results obtained from conventional resistivity profiling and ground conductivity surveys should be carried out in preference to resistivity profiling if a similar depth of investigation is required.

c) Ground probing radar

This method has been introduced into site investigation since the 1980s. The system consists of a radar antenna transmitting electromagnetic energy in pulse form at frequencies between 50 MHz and 1 GHz. The pulses are partially reflected by the subsurface geological structures, picked up by the receiving antenna and plotted as a continuous two-way travel time record, a pseudo-geological record section. The vertical depth scale of this section can be calibrated from the measured two-way travel times of the reflected events, either by use of the appropriate velocity values of electromagnetic energy through the lithological units identified or by direct correlation with borehole logs.

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The depth of penetration achieved by the radar pulse is a function of both its frequency and the resistivity of the ground. For UK soils, the maximum depth of penetration is likely to be between 1 m and 4 m but useful penetration to greater depths can sometimes be achieved in more resistive geological environments. At the moment ground probing radar is of limited use in normal depth to bedrock determination but developments in hand on current equipment include higher output power from the transmitter and lower frequency antennae (25 MHz).

35.3.7 ***Development of other geophysical techniques in geotechnical applications***

This information is included to indicate likely future trends in the application to the civil engineering industry of geophysical techniques currently in common use in other fields, such as mineral exploration. The use of these methods should not be considered without reference to a geophysical advisor to assess their applicability in any particular situation.

a) Induced Polarization (IP)

When electrical current flows in the ground, some parts of the rock mass become electrically polarized. If the current ceases abruptly, then polarized cells discharge and produce currents, voltages, and magnetic fields, which may be detected on the ground. Although this method is currently used mainly in the mineral exploration field, it is considered that it may have application in the study of contaminated land sites. Work in this area is currently at the research stage.

b) Transient Electromagnetic (TEM)

Electromagnetic energy can also be applied to the ground using transient current pulses instead of the continuous waves mentioned above. The collapse of a steady state primary magnetic field induces eddy currents to flow in a conductive earth, and these in turn give rise to a transient secondary magnetic field, which may be detected in a receiver coil as a time-dependent decaying voltage. The characteristics of this transient decay can be related to the conductivity and geometry of the subsurface geology. Typical TEM systems can provide rapid geoelectric depth scans from a few metres down to several hundred metres and therefore present an attractive alternative to electrical resistivity sounding. TEM is a well-established technique for mineral exploration and is increasingly being applied to hydrogeological mapping (especially saline intrusion problems) and to shallow engineering site investigation studies.

c) Magnetotelluric (MT)

Time-varying electromagnetic fields above the Earth's surface induce currents in the subsurface. The induced electric field decays with depth according to a skin-depth rule, which depends on the frequency. The largest fields are natural and occur at low frequencies (<1 Hz). They are the result of solar particles (e.g. flares) interacting with plasma in the Earth's near-space environment. The currents extend many kilometres into the subsurface. At higher frequencies, natural fields due to thunderstorms (sferics) exist which induce current systems in the shallow subsurface. By using sensors to measure the time variations of the electric (E) and magnetic fields (B) at the surface, the impedance (E/B) obtained provides a measure of the subsurface resistivity structure for geological interpretation. High frequency measurements have potential for both engineering and hydrogeological investigations.

d) Self Potential (SP)

Where an electrochemical gradient exists between two zones of groundwater then electrochemical or diffusion potentials may form. The method has been used in the study of water leakage from dam sites and the location of the boundaries of landfill sites.

e) Surface Wave Methods

Exploration of the velocity depth profile using surface waves, continuous or transient, is increasing in popularity. In the USA and Europe the technique is known as the Spectral-Analysis-of-Surface-Waves(SASW). Dispersion curves (variation of phase of group velocity with frequency) are constructed and interpreted by semi-empirical procedures or by numerical inversion as the shear wave velocity versus depth. The increasing capacity of field computers and the power of new numerical procedures enables layered systems of greater complexity than before to be characterized.

f) Microseismic Networks

Large installations, such as dams or radioactive waste disposal sites, often require the installation of networks of seismographs designed to pick up small seismic events (0 M_l to 2.0 M_l). These installations monitor potentially threatening faults and the effects of induced seismicity related to reservoir operations. In the extractive industry such microseismic monitoring is a key element in the monitoring and control of rock bursts and has been used to monitor both evaporite solution cavities and the stability of large and potentially threatening cavities. The data from such installations can be used to estimate the stress state at depth. In areas where strong motion seismic data are not available, microseismic data can be used for seismic hazard assessment.

35.4 Marine geophysics

35.4.1 ***General***

With suitable instrumental modification, the land survey techniques, such as seismic refraction, magnetic and gravimetric methods, may be extended to the marine environment. Of greater use are the three seismic methods described next, which were specifically developed for the marine environment. Modern positioning systems, such as the Global Positioning System (GPS) and the Differential Global Positioning System (DGPS) are now in common use and accurate position-fixing of the survey vessel can

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be achieved. For proper interpretation and reduction to an appropriate datum level, it is necessary to apply tidal corrections to the data obtained, particularly in the near-shore environment. Standard tide tables for the area of interest are normally used but in some cases a tide gauge may be installed and continuously monitored. Surveys using the marine geophysical surveying systems described below can be carried out very rapidly and are extremely cost-effective.

35.4.2 ***Echo-sounding***

A continuous water depth profile along the track of a survey vessel is obtained by using an instrument that measures the time taken for a short pulse of high frequency sound to travel from a transducer, which is attached to the survey vessel, down to the sea-floor and back again. Such profiles are combined to produce a bathymetric chart. Additional control may be required to ascertain whether the sounding is reproducing reflections from soft surface sediments or higher density material underneath. Compensation may be required to correct the vertical motion of the survey vessel. Swathe systems, where multiple echosounders collect depth data across a swathe (up to 6 times the water depth) are increasingly being used.

35.4.3 ***Continuous seismic reflection profiling***

The use of continuous seismic reflection profiling should always be considered as an essential aid to exploratory borings in major offshore site investigations. An instrumental extension of the echo-sounding principle is used to provide information on sub-seabed acoustic reflectors, which usually correspond to lithological and geological horizons. The instrumentation required, especially the acoustic source type, depends on the local geological conditions and its choice should be left to a geophysical advisor who has suitable experience. As a guide, the higher frequency sources, such as the “pinger” and “high resolution boomer” are suitable for resolving near-surface layering, whereas the “sparker” or “air-gun” is more suited for coarser and thicker overburden and for the acquisition of data from deeper levels beyond the penetration of boomers.

Signal processing can enhance the resolution, penetration, and signal-to-noise ratio of the resultant record. Important techniques are: swell filtering, to compensate for short period source-detector motion; time variant gain correction; and time variant frequency filtering. The results may visually reproduce geological features, but quantitative data on depths to interfaces can only be determined if characteristic velocities of the seabed materials are known. Close spacing of the seismic profiling survey lines allows 3D images of the geological structure to be created.

Two limitations of the technique are: first, that it usually cannot delineate the boundary between two dissimilar materials with similar geophysical characteristics, e.g. a very coarse glacial till and a heavily weathered and fractured rock; and secondly, that in shallow water depths, particularly in areas of “acoustically hard” seabed, near-surface reflectors may be obscured by multiple reflections originating from the seabed itself. Single and multichannel digital recording systems are available, which allow multiple suppression, filtering, and other signal enhancement operations to be carried out on the recorded data.

35.4.4 ***Sidescan sonar***

This is an underwater acoustic technique, analagous to oblique aerial photography and used for imaging the sea floor. It is based on the backscattered reflection of high frequency pulses of sound from the seabed, and provides a quantitative guide to the position and shape of seabed features and a qualitative guide to the type of seabed material. The system is particularly useful in surveys for rock outcrops, pipelines, sand waves, trenches and seabed obstructions, such as wrecks.

Seafloor mapping systems are available that apply digital scale corrections to produce a true isometric display of the seabed topography. Records from adjacent lines can be combined to produce a composite mosaic of the survey area.

35.5 Borehole methods

35.5.1 ***Borehole geophysical logging***

Borehole geophysical logging provides a series of profiles down the length of a borehole, each profile giving a different geophysical parameter measured with an appropriate sonde. The data are plotted against depth and a comparison of the data obtained from a number of sondes often yields useful information on the properties and characteristics of the subsurface. Experience of the use of these methods has increased considerably over the past decade and geophysical logging methods are used extensively on major site investigations to provide additional data from the site investigation boreholes. The subject of geophysical logging is broad and many different types of logging tools have been developed for a variety of engineering and hydrogeological applications. For further information reference should be made to specialist textbooks, such as Keyes [132], or standard guides, such as BS 7022:1989 (see Bibliography) and ASTM D5753-95 [133].

Logging tools may be divided into four main categories as follows:

a) formation logs, which provide information on the geological formations immediately surrounding the borehole;

b) fluid logs, which provide information on the fluid filling the borehole;

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c) physical properties logs, from which the geotechnical and hydrogeological parameters of the rock mass surrounding the borehole can be derived;

d) borehole geometry logs, which normally provide information on the borehole construction parameters.

For formation studies, the natural gamma and electrical logs are the most commonly used to provide stratigraphical correlation across a construction site or along the line of a tunnel or road excavation. The fluid logs are often used to indicate zones of water inflow and outflow from the borehole, while formation porosity can be derived from the neutron or sonic logs. Details of borehole construction can be obtained from logs, such as the cement bond log, which indicates the degree of coupling between the borehole casing and the surrounding formations.

Engineering design data, such as the dynamic elastic moduli, can be calculated from values of the compressional and shear wave velocities obtained from the full wave train sonic log and density values obtained from the formation density or gamma/gamma log. The degree of fracturing in the rock mass can be assessed from a number of the logs, particularly the sonic log, while the in-situ stress can be determined from the observation of borehole breakouts in the calliper log. It is essential that logs used to derive geotechnical and hydrogeological properties are calibrated against laboratory tests carried out on borehole core. It should be noted that the dynamic moduli obtained differ from the static values because of the different strain levels involved; hence, calibration with laboratory values can be difficult.

The performance of borehole geophysical logging methods is limited by the same controlling factors mentioned in **35.1**. It is also essential that each logging tool is carefully calibrated against known standards and the tool operated in the borehole at the correct operational speed. The application of borehole geophysical logging varies from site to site and it may be necessary to consult a geophysical advisor to achieve the optimum results. Logging, itself, is normally carried out by a specialist contractor.

Brief details of the principal logging methods are given in **35.5.1.1** to **35.5.1.7** and their main use in the construction industry is then summarized in Table 8.

35.5.1.1 *Caliper log*

The caliper log is used to obtain an accurate profile of the diameter of the borehole down its length, because correction for any changes in this parameter have to be applied to most other geophysical logs run in the borehole The log can be used, to a certain extent, for the identification of lithology and stratigraphic correlation but its most important use arises in the location of zones of fractured rock.

35.5.1.2 *Sonic log*

In its simplest form, the sonic log is a recording of the time taken by a compressional (P) wave to travel a defined length of formation plotted against depth. In the oil industry, this is expressed as microseconds per foot, but in civil engineering applications the more familiar units of microseconds per metre are used. The basic sonic log is used mainly to compute the formation porosity via the time average equation of Wyllie [134].

With the full wave train sonic log it is often possible to measure the velocities of both the compressional (P) and shear (S) wave velocities. Using values of the formation density computed from the gamma-gamma log, it is possible to calculate the dynamic elastic properties of the rock mass from the P and S wave velocities. It is not always possible to identify the S wave since there is often no distinct break at the onset of the shear wave pulse or the shear wave is highly attenuated. In the latter case this is extremely useful in picking out zones of highly fractured rock.

The presence of fractures in the rock mass interferes with the transmission of elastic wave energy along the wall of the borehole. In highly fractured rock both the velocity of propagation and amplitude of a compressional wave is considerably reduced and similar characteristics have been noted for shear waves. Laboratory and field values of V_P and V_S, the velocities of the compressional and shear waves respectively, can be used to compute a fracture index following the procedure suggested by Knill [135].

35.5.1.3 *Radioactivity logs*

a) Natural gamma

This log is a measurement of the natural radioactivity of a formation and is useful for lithological identification or stratigraphic correlation. The log also can be used for the identification of zones containing radioactive minerals such as potash or uranium-rich ores.

b) Gamma-gamma

This log measures the intensity of gamma radiation from a radioactive source in the sonde, such as cobalt 60 or caesium 127, after it is backscattered and attenuated within the borehole and the surrounding rock mass. Provided the necessary calibrations are applied to the sonde, the recorded count rate is directly proportional to the formation density. The effects of variation in the borehole diameter are offset by forcing the sonde against the wall of the borehole with a suitable locating device.

The main use of the gamma-gamma log in the oil industry is the determination of formation porosity, while in engineering investigations it is the formation density that is most applicable. The sonde can also be used for lithological correlation and identification of formations within a single geological area. Bulk density can be measured to an accuracy of ± 0.05 Mg/m^3 but this can be improved by careful calibration of the source, detectors, and instrumentation and considerable care in the preparation of the borehole.

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c) Neutron-neutron

The neutron-neutron sonde is used to measure formation porosity or moisture content, because it responds primarily to the amount of hydrogen present in the formation. A plutonium-beryllium or an americium-beryllium source is used to provide the neutrons. The best results are obtained with the neutron source and detector pressed against the borehole wall to minimize borehole effects. For accurate determination of moisture content or porosity very careful calibration of the sonde is required, particularly at low porosity values. The neutron sonde responds to the total water content of the formation and, as this would include adsorbed water associated with clay minerals, the porosity measured on a shale, for instance, is greater than the effective porosity of the formation. The log is usually very subdued in low porosity crystalline rocks, but the presence of a fracture zone artificially increases the effective porosity and a significant reduction in the neutron count is observed.

35.5.1.4 *Electrical methods*

a) Spontaneous potential log

The spontaneous potential log is a recording versus depth of the natural potential development between the borehole fluid and the surrounding rock mass. The log is used mainly for the detection of permeable beds, the location of their boundaries and their correlation between adjacent boreholes. In a low porosity rock mass, such as granite, variations in spontaneous potential are related to zones of fractured rock associated with joint patterns or fissures, but the measured values cannot be applied in a quantitative manner to obtain formation permeability.

b) Single point resistance log

This is the simplest of the electrical logging systems, in which a lead electrode is lowered down the borehole on an insulated cable, with an electrode buried at the surface to provide the return path for the current flow. The measured resistance is a function of the formation resistivity, so that variations are related directly to changes in lithology and the log has been used as a lithology tool for geological correlations between boreholes. Although the depth of investigation (into the borehole wall) is only between 50 mm and 100 mm, the log responds to fractures in the borehole wall and is often used in conjunction with the caliper log to investigate fracturing in the rock mass.

c) Normal devices

A more realistic value of the apparent resistivity is required to compute the in-situ porosity of the rock mass and this is achieved by use of the normal logging tool, which passes current deeper into the rock mass depending on the electrode separation. The normal tools are most effective in low to medium resistivity media, the short normal log proving particularly useful for geological correlation between boreholes. In low porosity rocks, such as granite, the resistivity of the rock mass is very high and the major factor influencing the resistivity log is joint patterns. The effects of fractures and joints can be observed in a qualitative manner in the rock mass by a comparison of the short and long normal logs.

d) Focused devices

The resistivity sondes described above are all relatively insensitive in high resistivity formations, because most of the current flow is in the borehole fluid and very little enters the formation. This results in poor resolution of the bed boundaries and the apparent resistivity measured is not representative of the formation resistivity. This limitation has been overcome in a high resistivity formation by the introduction of focused resistivity tools. The underlying concept is that if the measuring current is forced to flow radially as a thin sheet of current into the formation being logged, then the influence of the borehole and the surrounding formation on the resistivity measurement is minimized.

e) Inductive conductivity log

The borehole induction log measures the electrical conductivity of the surrounding rock mass within a distance range from 20 cm to 100 cm from the borehole axis. It is insensitive both to the conductivity of the borehole fluid and disturbed material in the wall of the borehole. Its mode of operation is similar to that of the ground conductivity meter described previously in **35.3.6**.

The principal advantage of the use of induced electromagnetic energy is that the conductivity of the rock mass can be measured in boreholes that have been cased with insulating PVC or Teflon tubing. Thus, although the induction log is normally used to locate zones of significant groundwater contamination in uncased boreholes to optimize the positioning of well screen, its real advantage is that it can be deployed in existing boreholes to confirm that the screening is correctly placed. The log can also be deployed for the monitoring of contamination levels outside cased boreholes to indicate changes in the plume composition with time.

f) Dipmeter

A dipmeter is usually a 4-arm (but can be 3-arm or 6-arm) side-wall micro-resistivity device, which measures fine variations of resistivity in the formation, with such a resolution that the relative vertical shift of characteristic patterns of variation on the traces can be used to derive the attitude of a plane intersecting the borehole. Such patterns can be caused by bedding planes, changes in lithology, or fractures and joints in the rock mass. The patterns are arranged across the traces as sinusoids. Computer analysis can identify the patterns by cross-correlation, and perform the trigonometrical solution, incorporating the relative shift of patterns, the orientation of the sonde and the borehole dimensions, to derive the attitude of the planes.

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35.5.1.5 *Formation scanning tool*

The basic principle of the formation micro-scanner is to map the resistivity of the borehole wall with a dense array of sensors. The sonde was originally a development of the dipmeter in which two of the four dipmeter pads carried an array of button electrodes. The sonde simply obtains a number of closely spaced microresistivity traces, which are processed to produce a very high resolution electrical image of two strips down the borehole wall. The data are produced in digitized form and can be processed by a computer-based interpretational package for direct comparison with the records from the borehole televiewer. A later development of the sonde equipped it with eight pads, which now provides in excess of 80 per cent of the borehole wall. The penetration of this form of electrical imaging is usually of a few centimetres into the formation and the resulting image bears a striking resemblance to recovered core. In addition, a wealth of data is produced on subsurface fracturing.

35.5.1.6 *Televiewer*

The Acoustic Borehole Televiewer produces an image of the borehole wall, based either on the measured amplitude or the transit time of a reflected acoustic pulse. The televiewer pulses ultrasonic energy from a piezoelectric transducer to the borehole wall via the fluid in the borehole, where some of the energy is reflected and detected by the transducer (now acting as a receiver). The transducer is rotated in the borehole at 3 rev/s and is oriented relative to the Earth's magnetic field by a downhole magnetometer within the sonde. The amplitude of the reflected signal is proportional to the reflected energy, which is a function of the acoustic impedance of the borehole wall. The raw amplitude and transit time values are processed using the digital techniques applied to the electrical data and again a flattened picture of the borehole wall with fractures appearing as sine wave is obtained.

35.5.1.7 *Television*

Borehole television logging involves the use of a low-light sensitive closed circuit television camera system, specially adapted for use under water. Illumination is from a variety of external peripheral lighting heads attached to the camera and selected to suit the individual borehole conditions. The camera can be packaged as small as 50 mm diameter and can operate in boreholes to a minimum diameter of 100 mm allowing for adequate clearance. For general usage, a simple axial view of the borehole is adequate, but for details of features on the borehole wall, mirror assemblies give a radial view, which can be scanned around the borehole by a motor-rotate unit fitted above the camera assembly. Focusing, light intensity, rotation and digital depth control on the image are from a surface control unit and the image is recorded on standard VHS format video tape.

Borehole CCTV logging allows for the condition (joints, leaks from corrosion perforation, damaged sections) and depth of casing in old boreholes to be determined where records may be poor or lost, prior to reinstatement. In open-hole sections, the inclination, azimuth (from a compass attachment) and frequency and aperture of fractures can be determined, together with any fluid ingress from above water level. The lithological nature and variation of the rocks of the borehole and the general borehole condition can be determined, i.e. zones of collapse or in water supply bores, lost pumps.

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Table 8 — Borehole logs and their applications and limitations

Application limitations	Log type															
	Formation micro	Televiewer	Spectral gamma	Diameter	Flow meter	Fluid conductivity	Fluid temperature	Televison	Caliper	Sonic	Neutron	Gamma - gamma	Natural gamma	Introduction	Electrical resistivity	Spontane potential
Lined hole			1		1	1	1	1	1	1	1	1	1			
Open hole	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1
Air-filled			1					1	1		1	1	1	1		
Water/mud filled	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1
Diameter									1							
Casing				1				1	1				1		1	
Fractures/joints	1	1		1	1	1	1	1	1	1		1			1	
Cement bend										1						
Bed boundaries	1	1	1	1				1	1	1	1	1	1	1	1	1
Bed thickness	1	1	1					1	1	1	1	1	1	1	1	1
Bed type			1							1	1	1	1	1	1	1
Porosity										1	1	1		1	1	
Density												1				
Permeable zones	1	1			1	1	1		1		1	1				
Borehole fluid quality						1										
Formation fluid quality														1	1	1
Fluid movement					1	1	1									
Direction of dip				1												
Shale/sand indication													1			1
NOTE A "l" against a particular log type means that it is suitable for the application listed.																

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35.5.2 ***Cross-hole methods***

The requirement for information on the ground mass outside the site investigation boreholes has resulted in a number of geophysical methods that operate between adjacent boreholes. The oldest of these methods is the cross-hole seismic technique, which provides a scan of the variation of the velocity of propagation and attenuation of both compressional and shear waves with depth. A further advance in this method has been the development of seismic tomography which can provide an image of the rock mass in terms of a seismic parameter, such as compressional wave velocity. This image can be related to the presence of geological discontinuities, such as fault and fracture zones, cavities, or dykes.

Further research is in progress into the development of the cross-hole electrical and radar techniques and again increased use of both methods is anticipated in the future.

35.5.3 ***Vertical seismic profiling (VSP)***

In this method an array of seismic detectors is deployed in a borehole and shots are fired from a seismic source located on the ground surface. Similar results to those obtained in standard seismic refractions and reflection surveys are observed, because each detector records both the direct seismic pulse as it propagates downwards and later pulses reflected from interfaces within the rock mass. By moving the seismic source away from the top of the borehole or moving the detector array within the borehole it is possible to determine the source of each pulse train observed at an individual detector and to produce a geological model that fits the recorded data. The VSP method can also be operated with the seismic source in the borehole and the receiving array on the ground. A shear wave source can also be used and the dynamic elastic moduli can be derived from the compressional and shear wave velocity data. The basic principles of the VSP method are described in Reynolds [130].

35.6 Penetration testing

The static cone penetration test involves the thrusting of a probe, normally 35.7 mm diameter and fitted with a conical point into the ground. Thrusts between 20 kN and 200 kN are normally used in the UK and useful penetration can be achieved in soils and some weak rocks. Probes can incorporate various sensors and have been developed to include a number of geophysical measurements, such as electrical resistivity, seismic wave velocity and nuclear density. The results are presented on plots versus depth and are comparable to borehole geophysical logging (**35.5.1**).

35.7 Corrosion testing

Electrical resistivity may be used to assess the corrosivity of soils towards ferrous materials. Conventional traverses with fixed intervals between electrodes enable rapid coverage of the ground and the location of areas of low resistivity. The spacing between electrodes should be appropriate to the depth of burial of the ferrous material. Conventional depth soundings may also be used to determine the variation in soil resistivity with depth. The likelihood of severe corrosion normally decreases as the resistivity rises (see appendix H of BS 5493:1977).

35.8 Specification and planning of a geophysical survey

35.8.1 ***General***

Darracott and McCann [136] have dealt with most aspects of planning surveys and have mentioned certain pitfalls involved in producing a specification. It is, for instance, inappropriate for the consulting engineer to specify the survey unless there is some understanding of the geophysical methods and their application in a site investigation programme. In some cases the wrong geophysical technique is specified and a rigid tender submission style called for, which usually utilizes an inappropriate Bill of Quantities to “measure” the amount of field work done.

There is a role for an independent experienced person overseeing the acquisition phase who can make some kind of quality grading, especially if the data are to be processed further prior to interpretation. This may help avoid time-consuming and expensive exploration of “system artefacts” and is in line with the recommendations of the report of the Site Investigation Steering Group [137]. A greater use of the geophysical advisor or consultant would dramatically improve the output obtained from a geophysical survey and its relevance to the geological problem under investigation. The geotechnical consultant would therefore have the benefit of the advice of a qualified geophysical advisor, who would specify the work required, supervise the work and evaluate the interpretation of the data obtained by the geophysical contractor.

It is suggested by McCann et al. [138] that the steps given in **35.8.2** to **35.8.11** should be followed where possible in designing a geophysical investigation.

35.8.2 ***Initial consultation***

The first consultation between the client and the geophysical advisor should cover the following points:

a) the exact nature of the geological or geotechnical problem;

b) whether the problem may be solved by geophysical methods;

c) what knowledge is available about the site.

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35.8.3 ***Preliminary site appraisal***

A site appraisal should be carried out covering the following aspects.

a) A site visit to assess its suitability for a geophysical survey. It is essential that surveys be located in the areas of prime interest and be extensive enough to provide information to a depth or horizontal extent sufficient for use in subsequent analysis or design efforts. The effect of existing man-made buried structural elements, such as sheet pile cut-offs, hydraulic conduits, pipes, electrical power cables (buried and overhead) or other items that might cause some interference need to be considered.

b) A health and safety assessment of the site and its proper documentation.

c) Any aerial photographs, geological maps, reports, borehole information and any other data should be studied to assist assessment of the suitability of the site for the use of geophysical methods. Geophysical methods provide information on the physical properties of subsurface materials (for example, seismic velocity), and in almost all cases the precision of measurement is very high. It is the task of the geophysical contractor to interpret these data to produce the most likely model that fits the observed data (see below). The accuracy, and indeed reality, of the interpretation and interferences drawn from the measurements depends very much on the experience of the interpreter and extent of other available geological control data. No purpose whatsoever is served by the practice, sometimes encountered, of the client withholding available control data in an attempt to “test” the geophysical contractor and his interpretation. All relevant information should be made available.

d) At this point an initial geophysical modelling should be carried out. It may be possible to predict the response of a particular geophysical method to the anticipated geological structure or anomaly using a suitable computer software package.

35.8.4 ***Pre-survey meeting***

When the geophysical adviser has considered the problem to be resolved, the data available and the nature of the site, a meeting should be held prior to the sending out of the requirements for a geophysical survey to a number of geophysical contractors for tender submission. A number of decisions need to be made as described in a) to e).

a) The selection of the most appropriate geophysical method or methods. A combination of methods may be more effective than a single method. It is essential that at this stage some agreement be reached about the operating parameters for the chosen technique(s) and the scope of work. For a seismic refraction survey, therefore, the geophone spacing should be selected to be consistent with the resolution required; the electrode configuration in a resistivity survey should be decided; and whether electrical soundings are required or constant electrode separation traverses. In a mineshaft location survey the measurement grid interval needs to be determined and agreed.

b) Cost-effectiveness has to be a consideration, and it is essential that a balanced programme be planned to yield the maximum amount of accurate information consistent with the overall objectives. It is usually possible to assess the likely costs of a geophysical survey at a given site having selected a particular technique and agreed a scope of work. It would be very difficult, however, and misleading to quote generalized unit cost rates for each type of operation here (for example, cost per kilometre of seismic refraction traverse). This is because so many parameters have to be considered in order to estimate a production rate. These include the size and nature of the site, the density of observations required, spacing between traverses and the required depth of investigation.

c) Adoption of agreed QA/QC procedures for the execution of the survey, the interpretation and presentation of the geophysical data and the format of the geophysical report.

d) Who is responsible for setting out and surveying? Ground elevations of geophone points and gravity stations, for example, are required. Allied to this is the question of access to the site, site clearance, if necessary, and responsibility for payment of damages, for example, to crops along a seismic traverse.

e) Arrangements for the safe storage of raw data for subsequent re-evaluation or reinterpretation should be agreed.

35.8.5 ***Geophysical trial survey***

If there is any doubt about the feasibility of the geophysical investigation, a test or trial survey should be conducted to determine the appropriate method and field techniques. A single geophysical method sometimes may not provide enough information about subsurface conditions to be useful on its own. Each method typically responds to several different physical characteristics of earth materials, and correlation of data from two or more geophysical methods may provide the most meaningful results. Some degree of redundancy and flexibility should be built into any well-planned site investigation. Certain methods may be useful under one set of subsurface conditions but offer little or no information in others. The results of the trial geophysical survey can be used to refine the specification of the main survey or, in some cases, lead to a decision not to proceed with any further geophysical work.

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35.8.6 ***Main geophysical survey***

This includes the following.

a) Setting out of geophysical traverse lines and sections, and topographic levelling.

b) Carrying out of the survey. Data quality is strongly enhanced by data reduction and analysis on-site. Modern geophysical digital equipment allows the downloading of all field data to a suitable portable PC and a preliminary interpretation can be carried out. It is good practice, if not imperative, for the geophysicist to reduce the data collected at the end of each day (or week, if more appropriate), make all check calculations possible, plot the data, and evaluate it for consistency with other data (geophysical, geological, borehole and pits) already to hand. In this way, major errors can be recognized and traverses re-run to obtain better data, and the survey programme can be modified to take advantage of newly obtained information.

c) It is relatively rare for the raw field records obtained during a geophysical survey to be included in an engineering geophysical report, but the records should be kept for any subsequent re-evaluation or re-interpretation. The records should be annotated and stored so that another geophysicist could receive them and proceed to an independent interpretation if necessary.

d) It is important to understand the distinction between the preliminary interpretation and the final results. Usually a geophysical field survey consists of a number of intersecting, or at least overlapping, traverses. Clearly the geological interpretation has to be the same at the common point(s) for the traverses concerned, but the initial geological interpretation of the traverses carried out earlier in the field programme often has to be revised. This is usually because the geophysical results are sensitive to parameters such as geological strike and topography. For example, in a seismic refraction survey, the data in one direction may appear to indicate a simple two-layer situation, whereas the data for a traverse crossing perpendicularly may be best represented by a three-layer case, thus hinting at the possibility of a thin, "hidden-layer" of variable thickness. The geophysical data set at a site should be treated as a whole and not piecemeal, and the premature use of data acquired early in the survey programme is probably one of the more frequent sources of disappointment with geophysical methods.

35.8.7 ***Correlation boreholes***

a) A borehole programme should then be designed both to check and/or "calibrate" the geophysical interpretation, and to obtain detailed information on specific problem areas. As additional subsurface information is obtained, the initial geophysical interpretation is refined, and plans for the remainder of the investigation may have to be revised to optimally fill in the gaps in the required subsurface information. Where it is possible the site investigation boreholes should be subjected to downhole geophysical logging in order to further refine the geophysical interpretation.

b) If the borehole and the geophysical survey programmes are carried out at sites where regional geological evidence indicates possible small-scale geological features, failure of the geophysical surveys to detect any such feature (e.g. a narrow zone of shearing in the rock mass), which could have an adverse effect on project safety or cost, does not mean that such features do not exist. While the combined use of geophysical and borehole investigations greatly improves the chances of finding these important small-scale features, the combination is not foolproof.

35.8.8 ***Submission of report***

The final report should incorporate all the geophysical results, presented in the agreed format and scales, together with any correlation drilling results, etc.

35.8.9 ***Feedback of subsequently acquired data***

The geophysicial advisor is not always appraised of the results of any subsequent corroborative boring, excavation or mapping that may have been recommended. This is unfortunate because it is therefore impossible to validate his interpretation or make a re-interpretation. Only by involving the geophysical advisor through all stages of a project, usually to completion, can the experience be built up which helps to ensure that the interpreter and fellow practitioners offer the best available service.

35.8.10 ***The commercial framework***

Although it is difficult to set out firm recommendations regarding the commercial aspects of the commissioning and execution of engineering geophysical surveys, Darracott and McCann [136] drew attention to certain factors that can influence the successful completion of an engineering geophysical investigation. The way that engineering geophysical surveys are specified and commissioned in practice often prevents the approach just described.

The following are some of the pitfalls listed by Darracott and McCann [136].

a) Specifications are often written by engineers or their assistants, who have little or no specialized understanding of geology and geophysics.

b) The inappropriate technique is sometimes specified and extreme cases have been known in which any attempt on the part of the geophysicist to suggest alternative approaches has led to penalization by rejection.

c) A rigid tender submission style is called for, usually utilizing an inappropriate Bill of Quantities to "measure" the amount of field work done.

35.8.11 ***Conclusion***

Darracott and McCann [136] concluded that on some projects the geophysicist can be so constrained that there is a danger of a conflict developing between the client and the geophysicist, and that the client may be more concerned with ensuring that contract conditions and procedures are followed, than with the value and success of the survey.

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Section 5. Laboratory tests on samples

36 General principles

36.1 Safety

Before samples are passed to the laboratory care should be taken to assess the possibility that some may be contaminated with harmful substances. If such a possibility exists, appropriate safety precautions should be implemented and preliminary tests done to determine the nature of any contamination. The results of these preliminary tests establish whether it is necessary to impose any special procedures to ensure the safety of the laboratory personnel.

36.2 Purposes of laboratory testing

The purposes of laboratory testing of samples of soil and rock may be summarized as follows:

a) to describe and classify the samples (see section 6);

b) to investigate the fundamental behaviour of the soils and rocks in order to determine the most appropriate method to be used in the analysis;

c) to obtain soil and rock parameters relevant to the technical objectives of the investigation (see section 2).

Analysis and interpretation of laboratory test results is not covered by this Code and reference should be made to the appropriate standards, design manuals and textbooks [139] [140] [141].

36.3 Selection of testing programme

The programme of laboratory testing and the precise specification of each test should be determined by the geotechnical advisor. Each test, or series of tests, should address one or more of the purposes listed in **36.2**.

A detailed description of the ground is essential before embarking on a programme of soil or rock testing. After this it is necessary to consider carefully to what use the data obtained from laboratory tests are to be put, whether the samples represent the conditions being investigated and whether the information can reasonably be expected to assist in the solution of the engineering problems concerned.

The principal factors which should be taken into account are:

a) the nature of the ground and the type of soil or rock being tested;

b) the quality of the sample and whether it represents the behaviour of the ground in-situ;

c) the method of analysis proposed: this might be, for example, an empirical method, a limit equilibrium calculation or a finite element analysis;

d) the requirements of the structure and the temporary works, including whether the designs are controlled by stability or by the need to limit deformations; and whether short-term or long-term conditions are most critical;

e) the capabilities of the laboratory where the testing is to be carried out.

Common laboratory tests on soil are given in Table 10. Many of the tests, together with references, are given in BS 1377:1990, which is published in nine parts, as listed in Table 9.

General comments on the scope and limitations of various laboratory tests on soil and rock are given in clauses **38;** comments on tests on soil are in clause **39** and its corresponding Table 10; and on rock in clause **40** and Table 11. Some tests describe and classify soils and rocks; some determine parameters used in empirical analyses and the design is based on a body of past experience, while other tests determine parameters used in more rigorous theoretical analyses.

36.4 Quality management

Tests should be carried out in a laboratory with the appropriate technical capabilities (i.e. goetechnical, mineralogical, chemical). The capabilities and quality of the testing laboratory should be approved by the geotechnical advisor. The laboratory should have an appropriate Quality Assurance system and, preferably, external accreditation (e.g. NAMAS, National Measurement Accreditation Service).

The laboratory should have a full-time supervisor. The supervisor should be a geotechnical specialist with experience in soil testing to the level required for the particular tests. Tests should be carried out by suitably qualified technicians with experience in the particular test, under the direction of the supervisor.

36.5 Relevance of test results

Some laboratory tests are suitable only for particular types of soil. Where, after a test has been carried out, it is found that the test was not relevant to the actual sample used, a suitable comment on the result should be made in the report of the laboratory tests.

All laboratory test results should be examined critically by the laboratory supervisor and by the geotechnical advisor. They should be examined to check the following:

a) that the results are reasonable, there are no gross errors and the results refer to the correct samples;

b) that the results from similar samples are consistent (i.e. results from samples with similar descriptions should be comparable);

c) that results from a particular sample are consistent (e.g. undrained strength should be consistent with the liquidity index);

d) that the results of the laboratory tests are consistent with the results of the in-situ tests;

e) that the test results are consistent with accepted data for similar soils and rocks.

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Whenever test results appear to be anomalous the reason should be established by re-examination of the original results or confirmed by further testing. If the reasons for a strange result cannot be established the anomaly should be clearly stated in the report on the laboratory tests.

37 Sample storage and inspection facilities

An important feature of a geotechnical laboratory is the provision of good facilities for handling, storing and inspecting samples. All samples entering the laboratory should be registered and receipts issued.

37.1 Handling and labelling

The general procedures for handling and labelling of samples in the field are given in **22.11**. Note should be taken of any warnings on the labels of likely contamination. Samples should be treated with care to prevent damage, both during transportation and on arrival in the laboratory.

37.2 Storage of samples

On arrival at the laboratory each sample should be registered. Samples should be stored so that they are protected from damage and deterioration, for example, from frost, excessive heat or change in humidity, and so they can readily be located when required for examination or testing. Disturbed samples should be placed in purpose-made carriers that prevent loss of moisture content. Tube samples and core samples should normally be stored on their sides in purpose-made racks with the possible exception of very soft marine sediment. The sample storage area should be of sufficient size to cater for the number of samples being handled without overcrowding. Ideally samples should be stored in a temperature and humidity controlled enclosure.

37.3 Inspection facilities

A specific area should be established in the laboratory for the inspection and description of samples. This should have sufficient space for the temporary stacking of the samples and an adequate area of bench space with good lighting, preferably daylight, for the actual inspection. In general, the following equipment should be provided:

a) an extruder for removing samples from the sample tubes or liners and a means of splitting plastic liners when extrusion is not desirable;

b) an adequate number of trays to enable disturbed samples of granular soils to be tipped out for inspection, and some means of returning them quickly to their containers afterwards;

c) spatulas and knives for splitting samples;

d) dilute hydrochloric acid to assist identification of carbonate soils and rocks;

e) a water supply to enable the fines to be washed out of samples of soils and facilitate description of the coarser particles; to clean rock cores and block samples; and to wet-up fine grained soils;

f) a balance suitable for checking the adequacy of bulk samples for testing;

g) means of resealing samples required for further use;

h) washing facilities for personnel inspecting the samples so that they can keep themselves and their notes clean;

i) hand lens, geological hammer, penknife, metre scale and protractor for logging cores;

j) a simple stereo microscope with magnification to times 30.

38 Visual examination and description of laboratory samples

38.1 Introduction

Description of samples of soil and rock tested in the laboratory forms an important part of the record of the test results. Such descriptions should be included on the laboratory work sheet(s). Descriptions of samples made in the laboratory should be compared with the equivalent field descriptions and any anomalies resolved.

Information about the grading and plasticity of soils can be estimated from inspection of bulk samples obtained during drilling, from tube samples not required for further intact testing or from trimmings from sample preparation operations. Information about the structure and fabric of soils and the jointing in rocks can be obtained from inspection of high quality samples, but is often best carried out by inspection of the ground in test pits and exposures. Variability, disturbance and representativeness of the samples are all important and appropriate comments should be made.

More detailed specialist examination of soil and rock samples may be valuable: these may include information of the palaeontology and examinations of the fabric, texture and petrography by optical or electron microscopy. Mineralogical analyses may also be helpful.

38.2 Description of samples

All samples to be tested should be described prior to testing and the information recorded on the laboratory worksheet for the test. It would be good practice to select other representative samples for detailed description during laboratory testing.

Procedures for the description and classification of soils and rocks are given in section 6.

38.3 Photographic records

Photographic records are a most valuable supplement to the written description or log. Photographs can be used to provide a continuous record of, for example, rock cores, typical features, such as in split tube samples, or a record of atypical features. Where a continuous record is sought, common magnification, lighting and format should be used. All photographs, including those taken in the field, should include a scale, colour chart, details of the provenance of the sample and the site location.

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Colour can be used to illustrate features clearly, but often monochrome can provide better definition of some features. Under normal circumstances 36 mm film and 35 mm lenses are a good choice, but wide angle lenses can offer better views of some features. Natural light is preferable to artificial light, but in the laboratory this is often difficult; when light is poor careful use of flash can give good results.

Photographs can be taken of samples in the "as received" state before testing and again after testing to show the mode of failure, or after splitting to show features of the fabric. Certain features may be seen more clearly after the sample has been allow to dry or partially dry.

The magnification required depends on the size of the features of interest. Rowe [42] used a magnification of times 30 to show fabric and structure in soils. Photomicrographs from electron microscopes are useful on occasions. Photographs of disturbed samples can be used to show details of the particle shape and grading.

The photographic record of laboratory samples can be supplemented with photographs from the field showing natural or excavated faces in quarries or test pits, spoil heaps and photographs of features observed during the walk over survey.

39 Tests on soil

39.1 General

Laboratory tests are used to determine design parameters and to complement field observations, field testing and back analysis of the behaviour of existing structures that have been monitored. In some cases a field test may give more realistic results because of reduced problems of sample disturbance. However, there is a large body of practical experience behind most of the more common tests and when the data derived from them are used with skill and experience, reliable predictions usually result. In assessing the quality and relevance of laboratory test results the following points **39.2** to **39.4** should be considered.

Table 9 provides a list of tests on soil contained in BS 1377:1990.

39.2 Sample quality

Classes of sample quality are defined in **22.2** and it is essential that the sample used is of sufficiently high quality for the test in question. Handling of samples in the field is described in **22.10**. When samples arrive in the laboratory, all necessary steps should be taken to ensure that they are preserved and stored at constant temperature, humidity and natural moisture content and that they suffer the minimum amount of shock and disturbance. Laboratory tests should be carried out at constant temperature, preferably in an air-conditioned laboratory. Laboratory tests should be carried out at a constant temperature and preferably in an air-conditioned laboratory.

Where the test is carried out on a nominally "undisturbed" sample, the sample may, in truth, be far from undisturbed: indeed the act of taking the sample releases the initial state of stress in it. In preparing the laboratory test specimen, there is further disturbance and unavoidable change in the stress conditions. As a result, the test is not usually carried out at the same state of stress and water content as exist in the natural ground unless attempts are made to return the specimen to its in-situ state (see **39.4**).

39.3 Sample size

For disturbed samples the quantity of soil required for any particular test is given in Table 3 in **22.2**. As the behaviour of the ground can be significantly affected by discontinuities, "undisturbed" samples should ideally be sufficiently large to include a representative pattern of these discontinuities. This may require use of large "undisturbed" samples.

39.4 Conditions of test

Where, as in the case of measurements of soil strength and compressibility or stiffness, tests can be carried out under several different sets of conditions, the particular test should be selected and carefully specified by a geotechnical specialist with relevant experience. The test carried out should be the one that determines the parameters required for the proposed design calculations. In many instances the test conditions and stress paths used should approximate to those that exist in the field at the time being considered in the design.

Table 9 — Categories of tests on soils specified in BS 1377:1990

Part 1	General requirements and sample preparation
Part 2	Classification tests
Part 3	Chemical and electro-chemical tests
Part 4	Compaction-related tests
Part 5	Compressibility, permeability and durability tests
Part 6	Consolidation and permeability tests in hydraulic cells with pore pressure measurement
Part 7	Shear strength tests (total stress)
Part 8	Shear strength tests (effective stress)
Part 9	In-situ tests

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Table 10 — Common laboratory tests for soil

Category of test	Name of test or parameter measured	Where details can be found	Remarks
Classification tests	Moisture content or water content	BS 1377-2	Frequently carried out as a part of other soil tests. Read in conjunction with liquid and plastic limits, it gives an indication of undrained strength.
	Soil suction	[142]	To assess negative pore pressures in soil samples; especially for desiccated soils.
	Liquid and plastic limits (Atterberg Limits)	BS 1377-2	To classify fine grained soil and the fine fraction of mixed soil.
	Volumetric shrinkage limit	BS 1377-2	To determine the moisture content below which a clay ceases to shrink.
	Linear shrinkage	BS 1377	To assess the magnitude of shrinkage on desiccation.
	Swelling clay content	BS 1377	Relevant to expansive materials and based on total cation exchange capacity of soil.
	Particle density	BS 1377-2	Values commonly range between 2.55 and 2.75 but a more accurate value is required for air voids determination. Only occasional checks are needed for most British soils, for which a value of 2.65 is assumed unless experience of similar soils shows otherwise.
	Mass density or unit weight	BS 1377-2	Used in the calculation of forces exerted by soil.
Classification tests	Particle size distribution (grading) a) sieving b) sedimentation	BS 1377 BS 1377-2 BS 1377-2	Sieving methods give the grading of soil coarser than silt and the proportion passing the finest sieve represents the combined silt/clay fraction. When the sample contains silt or clay the test should be done by wet sieving. The relative proportions of silt and clay can only be determined by means of sedimentation tests.
Chemical and electro-chemical tests	Dispersion	BS 1377-5	Qualitative tests to assess the erodibility of fine grained soils.
	Contaminants	See annex F	This is a rapidly developing field: check the most recent guidelines.
	Organic matter	BS 1377-3 BS 1924	Detects the presence of organic matter able to interfere with the hydration of Portland cement in soil: cement pastes.
	Mass loss on ignition	BS 1377-3	Measures the organic content in soils, particularly peats.
	Sulfate content of soil and ground water	BS 1377-3	Assesses the aggressiveness of soil or groundwater to buried concrete. (See remarks on test for pH value and chloride content.)

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Table 10 — Common laboratory tests for soil *(continued)*

Category of test	Name of test or parameter measured	Where details can be found	Remarks
Chemical and electro-chemical tests	Magnesium content	[143]	Supplements the sulfate content test to assess the aggressiveness of soil or groundwater to buried concrete.
	pH value	BS 1377-3	Measures the acidity or alkalinity of the soil or water. It is usually carried out in conjunction with sulfate content tests. This test and the two above should be performed as soon as possible after the samples have been taken.
	Carbonate content	BS 1377-3	Confirms the presence of carbonates, which often indicates cementing.
	Chloride content	BS 1377-3	Test recommended where pH of ground is less than 5.8. Results used in conjunction with those for sulfate, nitrate and pH to assess aggressiveness of ground, especially to concrete.
	Total dissolved solids in groundwater	BS 1377-3	A general measure of salinity indicative of aggressiveness of ground and related to electrical conductivity or soil resistivity
Soil corrosivity tests	a) bacteriological b) Redox pot c) Resistivity	BS 7361-1 BS 7361-1 BS 1377-3 BS 1377-3	Undisturbed specimens required in sterilized containers.
Compaction-related tests	Dry density (or dry unit weight)	BS 1377-9	Measures the mass (or weight) of solids per unit volume of soil. Often used as a quality control for compaction of fill.
	Standard compaction tests	BS 1377-4	Indicate the degree of compaction that can be achieved at different moisture contents with different compactive efforts.
	Maximum, minimum density and density index of coarse grained soil	BS 1377-4	Density index indicates the stiffness and peak strength of coarse grained soils. A number of different methods are available, so the method used should be clearly stated.
	Moisture condition value (MCV)	BS 1377-4	Determines compactive effort required to produce near-full compaction. Used for control of materials for earthworks.
Pavement design tests	California bearing ratio (CBR)	BS 1377-4	This is an empirical test used for design of flexible pavements. The test can be made either in-situ (see **31.4**) or in the laboratory.
	Chalk crushing value (CCV)	BS 1377-4	Similar in concept to the aggregate crushing value (ACV)
	Frost heave test	BS 812	Assesses susceptibility of compacted soil to frost heave.

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Table 10 — Common laboratory tests for soil *(continued)*

Category of test	Name of test or parameter measured	Where details can be found	Remarks
	Aggregate suitability	BS 812	Physical and chemical tests for aiding the selection and assessing the suitability of materials to act as bound and unbound aggregates.
Soil strength tests	Triaxial compression:	BS 1377 [141]	Triaxial tests are normally carried out on nominal 100 mm or 38 mm diameter samples with height to diameter ratio 2:1. If the height to diameter ratio is reduced to 1:1 the end platens should be lubricated. Undrained tests measure undrained strength s_u. Drained tests, or undrained tests with measurement of pore pressure, evaluate the Mohr Coulomb parameters c′and f′. Since soil strength depends on strain it is necessary to state whether the strength corresponds to the peak state, the critical state or the residual [144].
	a) Unconsolidated undrained	BS 1377-7	Prior to triaxial shearing, samples may be consolidated in the apparatus to some specified state: these are then known as consolidated undrained or consolidated drained tests as appropriate. Any drained or undrained test in which port pressures are measured should be consolidated before shearing.
	b) Undrained with measurement of pore water pressure	BS 1377-8	
	c) Drained with measurements of volume change	BS 1377-8	
	d) Multi-stage	BS 1377-7	Several techniques have been used for both drained and undrained multi-stage tests, details of which may be found in the references. The test may be useful where there is a shortage of specimens. Multi-stage tests are not recommended when single stage tests can be carried out.
	e) Stress path tests	[143]	Stress paths other than those used in a) to c) may be applied to reproduce the history of stress and strain in the ground before and during construction.
	Unconfined compression test	BS 1377-7	This simple test is a rapid substitute for the undrained triaxial test. It is suitable only for saturated non-fissured fine grained soil.
	Laboratory vane shear	BS 1377-7	For soft clay, as an alternative to the undrained triaxial test or the unconfined compression test.

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Table 10 — Common laboratory tests for soil *(continued)*

Category of test	Name of test or parameter measured	Where details can be found	Remarks
	Direct shear box	BS 1377-7	Direct shear tests are an alternative to triaxial tests although the latter are more versatile and more often used. Disadvantages are: drainage conditions cannot be controlled nor pore pressures measured and the plane of shear is predetermined by the nature of the test. An advantage is that samples of coarse grained soil can be more easily prepared than in the triaxial test. In general only drained tests should be undertaken. Shear boxes are normally square with sides 60 mm or 100 mm but may also be circular in plan. For very coarse grained soils shear boxes with sides 300 mm or larger should be used.
	Residual shear strength: a) Multiple reversal shear box b) Triaxial test with pre-formed shear surface c) Shear-box test with preformed shear surface d) Ring shear test	BS 1377-7	The residual shear strength of clay soil is relevant for slope stability problems where previous sliding has developed residual slip planes in-situ. The multiple reversal shear box test is the one that is most commonly used, although the ring shear test would be the more logical choice.
Soil deformation tests	One-dimensional compression and consolidation tests:	BS 1377	These tests measure soil parameters m_V and c_V for simple calculations of the magnitude and rate of settlement of foundations.
	a) Standard (incremental loading) oedometer test	BS 1377-5	The standard dead weight loading oedometer is the one in general use. The alternative is the hydraulic oedometer (Rowe cell) in which the vertical loading and the pore pressures can be independently controlled. Reasonable assessments of the magnitudes of foundation settlements can be made if: Class 1A samples are tested: For stiff clay a careful load —unload and reload sequence is applied using small increments and decrements. For soft clay reliable determinations of the yield shell are made. Estimates of settlement can be much improved if small strain triaxial and pressure meter tests are used. Estimates of the rate of settlement have been found to be highly inaccurate with certain types of soil.

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Table 10 — Common laboratory tests for soil *(continued)*

Category of test	Name of test or parameter measured	Where details can be found	Remarks
	b) Continuous loading oedometer tests	[141] [145]	Instead of applying the loads in discrete increments, as in the standard test, stresses, strains or pore pressures may be varied continuously.
	c) Swelling and collapse on wetting	BS 1377-5	Additional tests are carried out to determine the swelling pressure and the swelling or settlement on saturation.
	Shear and bulk modulus	[146]	Stress/strain relations for soils are highly non-linear and the bulk modulus and shear modulus both vary with loading. For the relatively small loadings, appropriate to most engineering applications soil strains are relatively small (typically less than 0.1 %). Measurement of these small strains requires use of special apparatus and procedures. These include use of local strain gauges attached to the sample and application of stress paths closely resembling the field stress paths.
Soil permeability tests	Tests in permeameters	BS 1377	The constant head test is suited only to soils of permeability normally within the range 10^{-4} m/s to 10^{-2} m/s. For soils of lower permeability,
	a) Constant head test	BS 1377-5	the falling head test is applicable. For various reasons laboratory permeability tests often yield results of limited value and in-situ tests are generally thought to yield more reliable data.
	b) Falling head test	[140]	
	Triaxial permeability test	BS 1377-6	The triaxial cell and the Rowe consolidation cell allow the direct measurement of permeability under constant head with a back pressure and confining pressures more closely consistent with the field state. The Rowe cell allows either vertical or radial flow.
	Rowe consolidation cell	BS 1377-6	

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39.5 Reporting test results

Results of laboratory tests should be reported in such a way that the accuracy of the observations is clear, as well as how the test was conducted and how the raw data obtained from the test were analysed and interpreted. Although results may be re-typed onto standard result sheets for presentation, the original laboratory worksheets should be kept for future reference. Data from tests should be provided in a form compatible with any Quality Assurance system that may have been chosen for the work.

The origin of the sample, its size and class and all procedures carried out prior to testing should be reported. In some cases the test results are reported factually: in other cases the test results may be interpreted and values for design parameters given.

The appearance of samples tested for strength, deformation or permeability, should always be sketched and/or photographed after they have been tested. Visible surfaces of failure should be noted and their angle to the direction of externally applied direct loads recorded. Samples should be broken open to assess their uniformity and to identify any obvious reason for their behaviour.

40 Tests on rock

Common laboratory tests for rock, together with references, are given in Table 11. Most of the comments on testing soils given in clause **39** also apply to tests on rock.

Because the behaviour of rock masses is so frequently controlled by the nature of the discontinuities present and their orientation to the stresses created by the works or during their construction, laboratory tests on rock require a clear distinction to be made between tests that relate to the behaviour of the discontinuities within the rock [147] and those that are relevant to the rock material.

Rock discontinuities and regions of weak rock are usually the critical elements in a rock mass and their shear strength and deformability are often useful parameters to obtain in laboratory tests. The influence of changes in moisture content on the strength and deformation characteristics of rock may not be immediately apparent from visual inspection alone, but this factor should not be disregarded.

Weak rock is frequently highly fractured, which makes it difficult to sample and test in the laboratory. It should be recognized that sometimes only the stronger sections of a rock core are tested and the weaker and missing sections are not tested at all.

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Table 11 — Common laboratory tests for rock

Category of test	Name of test	Where details can be found	Remarks
Rock classification tests	Saturation moisture content (alteration index) Bulk density Moisture content Porosity	BS 812-2 [148]	The parameters from these tests may be related to other parameters such as compressive strength, modulus of elasticity, seismic wave velocity, resistance to weathering and degree of weathering.
	Petrographic analysis	[149]	Useful to identify rock type and degree of weathering and gives an indication of stress history.
	Slake-durability	[148]	Useful quality index for testing clay-bearing rocks proposed for construction materials.
	Hardness and abrasiveness	[150]	Gives some indication of potential wear and tear of machinery involved in rock cutting and breaking.
	Carbonate test	BS 1881-6	Reference describes a method using Collins' calcimeter. Useful for the identification of chalk and calcareous mudrocks.
	Swelling test	BS 1377, [148] [151]	Gives some indication of moisture sensitivity of rock and possible ranges of induced pressures on tunnel linings.
Dynamic tests	Seismic velocity Dynamic modulus	[152] [150] [152]	Results may sometimes be useful in extrapolating laboratory and field tests to rock mass behaviour.
Rock strength tests	Point load test	[153]	Simple laboratory and field strength test. Useful aid to core logging.
	Uniaxial compression	[154]	Carried out on intact samples with no discontinuities and yields data on the rock material properties. The length to diameter ratio of 2:1 is a minimum for cylinders.
	Direct tension test Indirect tensile strength test (Brazil test)	[155] [155]	See remarks on uniaxial compression test.
	Triaxial compression: a) Undrained b) Undrained with measurement of pore water pressure c) Drained	[156] [157] [158] [159] [160] [156]	Usually carried out on intact samples with no discontinuities and yield data on rock material properties. Larger samples may contain one or more discontinuities, in which case the data relates to the properties of the rock mass.
Discontinuity strength tests	Direct shear box	[161] [162] [163]	Of considerable importance for study of friction on discontinuities. References also cover residual shear strength. Where joints are filled with gouge the properties of the combined gouge and joint should be determined under conditions closely simulating those existing in-situ.

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Table 11 — Common laboratory tests for rock *(continued)*

Category of test	Name of test	Where details can be found	Remarks
Rock deformation tests	Static elastic modulus	[154] [156]	See remarks on triaxial compression test.
Rock deformation tests	Creep tests: a) Undrained b) Constant load c) Triaxial	[164] [165]	Most meaningful when carried out under multi-stress conditions.
	Consolidation of rock mass containing gouge material		Treat the gouge material as a soil (see clause **37**).
Rock permeability tests	Triaxial cell test	[166]	Makes use of a modified Hoek-Franklin cell.
	Centrifugal test	[167]	Considerably faster than other methods.
	Radial test	[162] [168] [169]	A measure of the degree of fracturing of the rock material.

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Section 6. Description of soils and rocks

41 Description of soils

41.1 The scope of soil description

Detailed descriptions of samples of soil and rock are an important aspect of ground investigation. The results of a ground investigation may be required long after the disposal of the samples when the descriptions are, in many cases, the only evidence remaining of what was discovered. In addition, designers often make use of past experience for materials of similar age, origin or condition based on the descriptions. For this reason, classification (see clause **42**) may provide additional guidance on the behaviour of a particular soil.

Soil descriptions are made on samples recovered from boreholes and excavations and/or from examination of the in-situ materials. The reliability of sample descriptions in reflecting the in-situ characteristics depends greatly on the quality of the samples and the level of detail in the description should reflect this quality. It is essential that any doubts as to the representativeness or reliability of the sample be stated. An accompanying report should give the origin, type and quality of each sample. Table 12 summarizes the descriptive process in flowchart form and Table 13 summarizes the recommendations that are developed in more detail in the text.

The words cohesive and non-cohesive, or granular, are often used to distinguish soils that contain a significant proportion of fine grains and behave in a cohesive manner when subjected to quick loading, from coarse-grained soils, which have no apparent cohesion. The word cohesive usually describes a soil which has an undrained strength measured in an unconfined compression test. In this test the strength arises from a combination of friction and negative pore pressure; cohesion therefore describes the ability of a soil to sustain a pore water suction when unconfined.

All soils can be considered granular so, in terms of effective stress, true cohesion in any uncemented soils is very small (most uncemented soils slake when immersed in water). Moreover any uncemented soil can behave in either a cohesive manner or in a non-cohesive manner depending on the response of the pore pressure to loading. For example, saturated clean sand is non-cohesive and can be poured, but unsaturated sand can behave in a cohesive manner and a cohesive strength can be measured. Under long term, fully drained conditions, clays behave in a frictional, non-cohesive manner and the long term stability of clay slopes is governed by the friction angle of the soil.

To avoid these contradictions soils are described as coarse or fine. Coarse soils are gravels and sands. When saturated and unconfined, they cannot sustain negative pore pressures, so these soils do not have any undrained strength or apparent cohesion. Fine soils are clays and silts. When saturated they can sustain suctions in unconfined tests and so have an apparent cohesion. If well-graded soils contain sufficient fine grains to fill the spaces between the coarse grains, they are described as fine soils; if well-graded soils contain insufficient fine grains to fill the spaces between the coarse grains, they are described as coarse soils. These distinctions based on grain size apply equally to natural and compacted soils. In the following clauses, are reference is made to “material” and “mass” characteristics; these can be divided into nature, state and structure as follows:

— *Nature of the soil grains*: their particle size grading, shape and texture or, if appropriate, their plasticity, together with special features such as organic or carbonate content. The nature of a soil does not usually change during civil engineering works. Nature can be described on disturbed samples.

— *State of the soil grains*: their packing, water content, strength or relative density, and stiffness. The state of a soil usually changes during civil engineering works; the description of the soil requires undisturbed samples or exposures.

— *Structure*: describes all the features of a soil that are removed by reconstitution, i.e. fabric or microfabric features, such as layering, fissuring or cementing. Structure is often destroyed by large distortions, and so can be observed only in the field on natural or artificial exposures or, to some extent, in an undisturbed (Quality Class 1) sample.

The geological formation, age and type of a deposit may also be named where known, but these may not be readily determinable without a detailed geological study of the area. A material formed by man (made ground or fill) can be highlighted here.

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Table 12 — General identification and description of soils

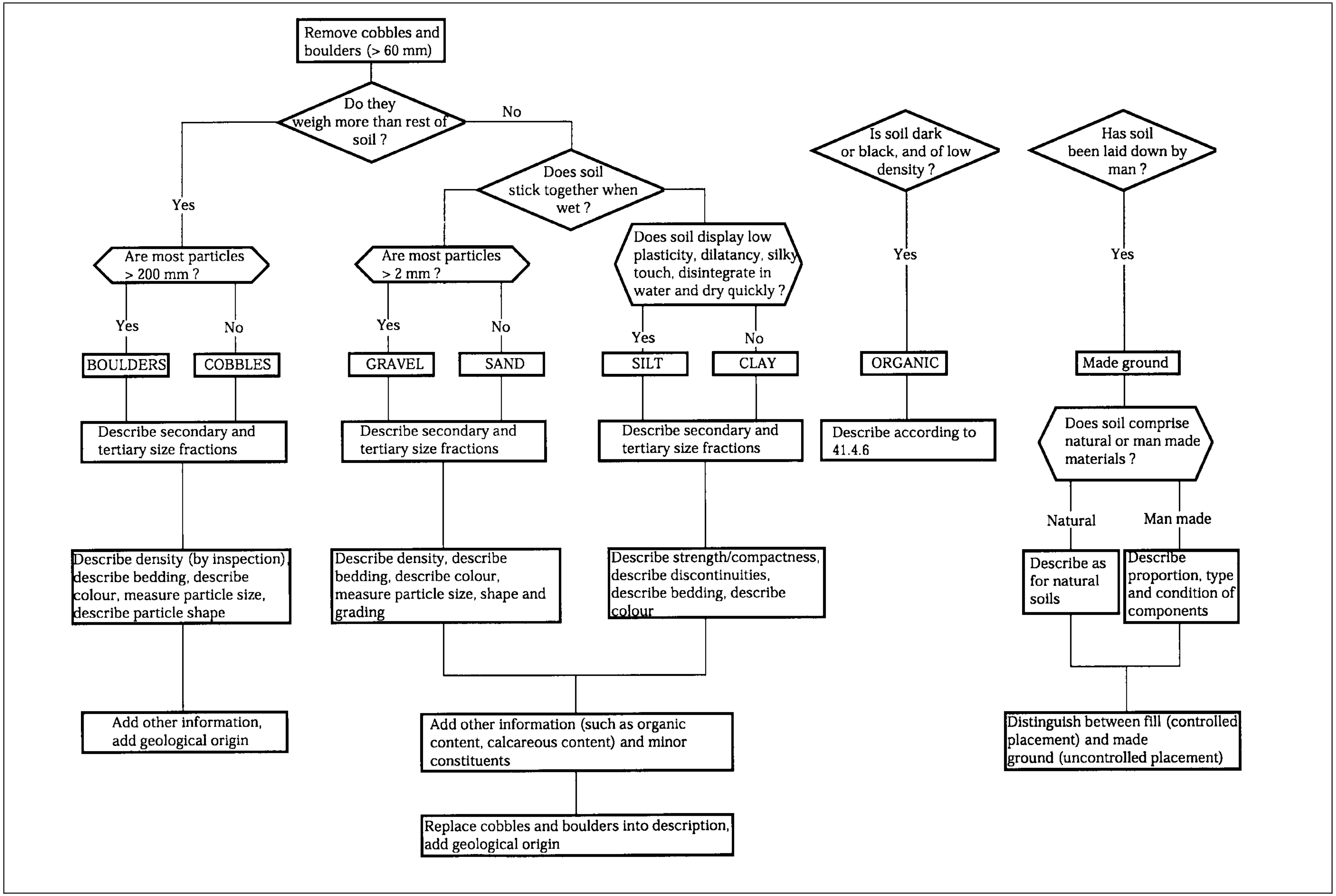

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Table 13 — Identification and description of soils

Soil group	Density/compactness/strength: Term	Field test	Discontinuities: Term	Mean spacing mm	Bedding: Term	Mean thickness mm	Colour	Composite soil types (mixtures of basic soil types): Term	Approx. % secondary[c]	Particle shape	Particle size	PRINCIPAL SOIL TYPE
Very coarse soils	Loose	By inspection of voids and particle packing	Scale of spacing of discontinuities		Scale of bedding thickness			For mixtures involving very coarse soils, see 41.4.4.3			— 200	BOULDERS
	Dense						Red Orange Yellow			Angular Sub angular	— 60	COBBLES
Coarse soils (over about 65% sand and gravel sizes)	Borehole with SPT N-value		Very widely	Over 2 000	Very thickly bedded	Over 2 000	Brown Green	Slightly (sandy[d])	< 5	Sub rounded	Coarse	GRAVEL
	Very loose	0 - 4	Widely	2 000 to 600	Thickly bedded	2 000 to 600	Blue			Rounded	— 20	
	Loose	4 - 10	Medium	600 to 200	Medium bedded	600 to 200	White	(sandy[d])	5 to 20[b]	Flat	Medium	
	Medium dense	10 - 30	Closely	200 to 60	Thinly bedded	200 to 60	Cream			Tabular	— 6	
	Dense	30 - 50	Very closely	60 to 20	Very thinly bedded	60 to 20	Grey	Very (sandy[d])	> 20[b]	Elongated	Fine	
	Very dense	> 50	Extremely closely	Under 20	Thickly laminated	20 to 6	Black				— 2	
			Fissured	Breaks into blocks along unpolished discontinuities	Thinly laminated	Under 6	etc.	SAND AND GRAVEL	about 50[b]	Minor constituent type	Coarse — 0.6	SAND
	Slightly cemented	Visual examination: pick removes soil in lumps which can be abraded								Calcareous, shelly, glauconitic, micaceous etc. using terms such as	Medium — 0.2 Fine — 0.06	
Fine soils (over about 35% silt and clay sizes)	Un-compact	Easily moulded or crushed in the fingers	Sheared	Breaks into blocks along polished discontinuities	Inter-bedded	Alternating layers of different types Prequalified by thickness term if in equal proportions. Otherwise thickness of and spacing between subordinate layers defined	Light	Term	Approx. % secondary[c]	Slightly calcareous,	Coarse — 0.02	SILT
	Compact	Can be moulded or crushed by strong pressure in the fingers					Dark	Slightly (sandy[e])	< 35	calcareous,	Medium — 0.006	
	Very soft 0 - 20	Finger easily pushed in up to 25 mm	Spacing terms also used for distance between partings, isolated beds or laminae, dessication cracks, rootlets etc.		Inter-laminated		Mottled			very calcareous.	Fine — 0.002	CLAY/SILT
	Soft 20 - 40	Finger pushed in up to 10 mm						(sandy[e])	35 to 65[a]	% defined on a site or material specific basis or subjective		CLAY
	Firm 40 - 75	Thumb makes impression easily										
	Stiff 75 - 150	Can be indented slightly by thumb										
	Very stiff 150 - 300	Can be indented by thumb nail						Very (sandy[f])	> 65[a]			
	Hard (or very weak mudstone) Cu > 300 kPa	Can be scratched by thumbnail see 41.2.2										

Soil group	Term	Field test	Term	Description	Transported mixtures	Colour	Description
Organic soils	Firm	Fibres already compressed together	Fibrous	Plant remains recognizable and retains some strength	Slightly organic clay or silt Slightly organic sand	Grey as mineral	Contains finely divided or discrete particles of organic matter, often with distinctive smell, may oxidize rapidly. Describe as for inorganic soils using terminology above.
					Organic clay or silt Organic sand	Dark grey Dark grey	
	Spongy	Very compressible and open structure	Pseudo-fibrous	Plant remains recognizable, strength lost	Very organic clay or silt Very organic sand	Black Black	
					Accumulated in situ		
	Plastic	Can be moulded in hand and smears fingers	Amor-phous	Recognizable plant remains absent	Peat		Predominantly plant remains, usually dark brown or black in colour, distinctive smell, low bulk density. Can contain disseminated or discrete mineral soils

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Table 13 — Identification and description of soils *(continued)*

<table>
<tr><th>PRINCIPAL SOIL TYPE</th><th>Visual identification</th><th>Minor constituents</th><th>Stratum name</th><th>Example descriptions</th></tr>
<tr><td>BOULDERS</td><td>Only seen complete in pits or exposures</td><td rowspan="7">Shell fragments, pockets of peat, gypsum crystals, flint gravel, fragments of brick, rootlets, plastic bags etc.

using terms such as;

with rare

with occasional

with abundant/frequent/ numerous

% defined on a site or material specific basis or subjective</td><td rowspan="7">RECENT DEPOSITS,

ALLUVIUM,

WEATHERED BRACKLESHAM CLAY,

LIAS CLAY,

EMBANKMENT FILL,

TOPSOIL,

MADE GROUND OR GLACIAL DEPOSITS ? etc.</td><td rowspan="7">Loose brown very sandy sub-angular fine to coarse flint GRAVEL with small pockets (up to 30 mm) of clay.
(TERRACE GRAVELS)

Medium dense light brown gravelly clayey fine SAND.
Gravel is fine
(GLACIAL DEPOSITS)

Stiff very closely sheared orange mottled brown slightly gravelly CLAY.
Gravel is fine and medium of rounded quartzite.
(REWORKED WEATHERED LONDON CLAY)

Firm thinly laminated grey CLAY with closely spaced thick laminae of sand.
(ALLUVIUM)

Plastic brown clayey amorphous PEAT.
(RECENT DEPOSITS)</td></tr>
<tr><td>COBBLES</td><td>Often difficult to recover whole from boreholes</td></tr>
<tr><td>GRAVEL</td><td>Easily visible to naked eye;
particle shape can be described;
grading can be described.</td></tr>
<tr><td>SAND</td><td>Visible to naked eye;
no cohesion when dry;
grading can be described.</td></tr>
<tr><td>SILT</td><td>Only coarse silt visible with hand lens;
exhibits little plasticity and marked dilatancy;
slightly granular or silky to the touch:
disintegrates in water;
lumps dry quickly;
possesses cohesion but can be powdered easily between fingers</td></tr>
<tr><td>CLAY/ SILT</td><td>Intermediate in behaviour between clay and silt.
Slightly dilatant</td></tr>
<tr><td>CLAY</td><td>Dry lumps can be broken but not powdered between the fingers;
they also disintegrate under water but more slowly than silt;
smooth to the touch;
exhibits plasticity but no dilatancy;
sticks to the fingers and dries slowly;
shrinks appreciably on drying usually showing cracks.</td></tr>
</table>

NOTES

[a] Or described as coarse soil depending on mass behaviour

[b] Or described as fine soil depending on mass behaviour

[c] % coarse or fine soil type assessed excluding cobbles and boulders

[d] Gravelly or sandy and/or silty or clayey

[e] Gravelly and/or sandy

[f] Gravelly or sandy

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41.2 The basis of soil description

A soil's characteristics are based on the particle size grading of the coarser particles and the plasticity of the finer particles; these play a major role in determining the engineering properties of the soil and form the basis of the soil's description. A first appraisal of the engineering properties should be made from the visual description of the soil's nature and composition, assisted by a few simple hand tests (see clause **43**). Soils that stick together when wet and can be rolled into a thread that supports the soil's own weight (i.e. have cohesion and plasticity), contain sufficient silt and/or clay in them to be described as fine soils. Soils that do not exhibit these properties behave and should be described as coarse soils. In addition the soil may be classified (see clause **42**). When accurately applied, description supplies valuable information that may allow correlation of materials occurring on different parts of the site or on other sites. In some cases, this may be sufficient to allow the most effective solution of an engineering problem to be judged.

The process of description first requires a decision on the principal soil type, followed by a description of the secondary and minor fractions and other features such as bedding, colour and particle shape. This descriptive process is summarized as a flowchart in Table 12. The decision-making process shown there is in a different sequence to the order of words in the written description.

In a soil description, the main characteristics should be given using the following standard headings, as applicable (these are in the same order as the column headings in Table 13). The main description can be followed, where appropriate, by further details for clarity.

a) mass characteristics comprising state and structure (see **41.3**):

1) density/compactness/field strength (see column 2 of Table 13);

2) discontinuities (see column 3 of Table 13);

3) bedding (see column 4 of Table 13);

b) material characteristics comprising nature and state (see **41.4**);

1) colour (see column 5 of Table 13);

2) composite soil types: particle grading and composition; shape and size (see columns 6, 7 and 8 of Table 13);

3) principal soil type (name in capitals, e.g. SAND), based on grading and plasticity shape (see column 9 of Table 13);

c) stratum name: geological formation, age and type of deposit (see **41.5**); classification (optional) (see clause **42**). (See column 12 of Table 13.)

EXAMPLES:

"Firm closely-fissured yellowish-brown CLAY (LONDON CLAY FORMATION). Loose brown sub-angular fine and medium flint GRAVEL (TERRACE GRAVELS)."

Materials in interstratified beds may be described as follows:

"Thinly interbedded dense yellow fine SAND and soft grey CLAY (ALLUVIUM)."

Any additional information or minor details should be placed at the end of the main description after a full stop, in order to keep the standard main description concise.

41.3 Mass characteristics of soils

41.3.1 *Range of application*

Under the heading "mass characteristics of soils" are descriptions of those characteristics that depend on structure and which can therefore only be observed in the field or in some undisturbed samples. Reference should be made to Table 13. Further explanations of certain aspects are given in the following clauses.

41.3.2 *A scale of strength and relative density*

The scale for strength of clays estimated in the field is given in column 2 (Density/Compactness/Strength) of Table 13; a scale in terms of undrained shear strength is as follows.

Term	Undrained shear strength kN/m^2
Very soft	less than 20
Soft	20 to 40
Firm	40 to 75
Stiff	75 to 150
Very stiff	150 to 300
Hard (or very weak mudstone)	Greater than 300

The assessment of undrained strength is affected by a number of factors including fabric, sample disturbance and moisture content and stress changes. Where such assessment is critical, appropriate testing should be carried out. This may be particularly marked in very soft or very stiff clays, but may occur throughout the strength range. Any doubts as to the representativeness of the sample seen should be indicated e.g. "probably firm, brown CLAY". Clays with undrained strength greater than about 300 kN/m^2 can be described as very weak mudstone or as hard clay [170]. The field assessment of 300 kN/m^2 is not easy, being beyond the range of hand penetrometers and thumb nails, but such clays, in their saturated condition, break in a brittle manner.

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Silts are described as fine soil (see **41.4**) but, depending on their grading, may behave as a coarse soil. The compactness terms given in column 2 (Density/Compactness/Strength) of Table 13 should be used; these terms for silts are unquantified, and may not be applicable to disturbed samples.

If a silt or clay includes fissures or other discontinuities the field description shall apply to the intact material between discontinuities. If a mass strength can also be assessed in the field this should be separately and clearly reported. These discontinuities are so important that the term should be included as part of the strength descriptor, e.g. "stiff fissured", "firm sheared". Description of the discontinuities is given at the end of the main description, after a full stop for clarity. If a mineral cement appears to be present the nature and degree of cementing should be noted e.g. "slightly iron cemented sand". It is also useful to note whether slaking occurs on immersing the air dry material in water.

It is desirable that operators check their strength descriptions against test results from time to time to ensure that their judgement is sound; pocket penetrometers or hand vanes are useful and immediate for this purpose. A variety of hand tools is available and their use is recommended to improve reliability; they should be used in accordance with the manufacturer's instructions. The results should only be used for guidance or comparison as the volumes tested are small and each tool tests the soil in a different way. The inclusion of the test results on the log is recommended.

The use of empirical relationships between standard penetration test "N" values and strength of fine soils may also be useful [171, 172]. Laboratory results should also be used, as the effects of sample disturbance on laboratory test results can lead to significant underestimation of field strength. Discrepancies between strength assessment should be reported, apart from operator error, which should be corrected, bearing in mind that the different tests involve different sample sizes, failure modes and strain rates. Laboratory tests on fissured materials usually give lower results than the intact material strength assessed in the field.

The relative density of sands and gravels only may be determined by the standard penetration test. A scale in terms of N-values (see BS 1377:1990) is as follows.

Term	**SPT N-values, blows/300 mm penetration**
Very loose	0 to 4
Loose	4 to 10
Medium dense	10 to 30
Dense	30 to 50
Very dense	over 50

The numerical values on the log should be uncorrected [53]. The field N-value can be corrected, at least for rod energy and overburden pressure, before applying the above descriptor. If any correction to SPT N-values is made prior to assessing the equivalent descriptive term, this should be stated on the borehole log. Particular care in applying these descriptors is required in coarse gravels; they should not be used for very coarse soils. Relative density terms should not normally be determined, except from the results of standard penetration tests.

41.3.3 *Discontinuities*

Discontinuities in soils should be described; types include fissures and shear planes. The spacing of discontinuities should be described using the spacing scale given in column 3 (Discontinuities) of Table 13. Their surface texture, e.g. rough, smooth, polished, striated should be described (see column 5 of Table 15). Where possible in exposure, the orientation or trend of discontinuities should be given by stating direction of dip and angle of dip (e.g. 180°/40°); their persistence and openness should also be stated. Alternatively, the apparent dip with respect to a specified exposed surface may be given where the previous measurements cannot be made.

41.3.4 *Bedding*

The thickness of bedding units should be described using the terms in column 4 (Bedding) of Table 13; in a homogeneous soil this is marked by bedding planes or, possibly, colour changes, and not necessarily discontinuities.

Interstratified deposits are those in which there are layers of different types of material, which may be of constant thickness, or may thin out locally or occur as lenses.

If beds of alternating or different soil types are too thin to be described as individual strata, the soil may be described as interbedded or interlaminated, using the terms in column 4 (Bedding) of Table 13, as appropriate. Where the soil types are approximately equal, "thinly interlaminated SAND and CLAY" would, for example, be appropriate. Where one material is dominant, the subordinate material should be described with a bed thickness and a bed spacing (using the bedding and discontinuity spacing terms respectively), e.g. "SAND with closely spaced thick laminae of clay".

Partings are bedding surfaces that separate easily, e.g. a layer of silt of no appreciable thickness in a more cohesive material. The nature of any parting material should be noted. The spacing of sedimentary features, such as shell bands, and of minor structures, such as root holes in soils, may also be described using the spacing terms for discontinuities. There are descriptive terms that have no size connotation (e.g. pocket, lens, inclusion); where such terms are used their size should be defined or reported.

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Any special bedding characteristics, e.g. cross-bedding, graded bedding, should be described, besides disturbed bedding structures, including slump bedding or convoluted bedding. Where two or more soils types are present in a deposit, arranged in an irregular manner, the soils may be described as intermixed, e.g. "intermixed SAND and gravel size pockets of CLAY".

41.4 Material characteristics of soils

41.4.1 *Range of application*

Material characteristics refer to those characteristics that can be described from visual and manual examination of either disturbed or undisturbed samples, and include soil name, colour, particle shape, particle grading and particle composition.

41.4.2 *Colour*

Details are given in column 5 (Colour) of Table 13. The colour given should be the overall impression of the stratum. Strata with more than one distinct colour may be described, for example, as mottled, but strata with more than three distinct colours should normally be described as multicoloured. Colour changes due to oxidation or desiccation, for example, should be noted. For more detailed descriptions, colour charts such as those based on the system of Munsell may be used [173] [174]. Consistency of colour description is usually more important than absolute accuracy, and the use of reference samples or commercially available colour samples and charts is useful in this regard.

41.4.3 *Particle shape, grading and composition*

Where appropriate, particle shape may be described by reference to the general form of the particles, their angularity (which indicates the degree of rounding at edges and corners) and their surface characteristics. Some recommended terms are as follows. The angularity and form terms are illustrated in Figure 17.

Angularity	angular subangular subrounded rounded
Form	flat or tabular elongated
Surface texture	rough smooth

Angularity terms are normally only applied to particles of gravel size or larger; form would usually only be described for extreme particle shapes. Occasionally, the use of these terms may be of greater importance and used more fully, such as in an aggregate assessment study. "Flat" is preferred to the term "flaky" used in BS 812 where the shapes are illustrated. The surfaces of particles may be described, for example, as etched, pitted, honeycombed or polished.

The distribution of particle sizes within sands and gravels should be described using the terms in columns 8 (Particle size) and 9 (Principal soil type) of Table 13, stating the predominant size fractions present, e.g. "fine and medium GRAVEL" or "fine to coarse SAND". The absence of these adjectives means that fine, medium and coarse fractions are all present in roughly equal proportions.

The composition of particles visible to the naked eye or with a hand lens may be described. Gravel particles are usually rock fragments, e.g. sandstone, limestone, flint. Sand and finer particles are normally individual mineral grains, e.g. quartz, mica, feldspar. Gravel and sand particles may be coated with mineral matter, including calcite, limonite and other iron oxides. Crystals, for example gypsum in clay, may be present. Particles may be weathered, showing, for instance, cracking, concentric layering or discoloration. These conditions should be described, where appropriate.

41.4.4 *Soil name (Principal soil type and secondary constituents)*

41.4.4.1 *Introduction*

The soil name is based on particle size distribution of the coarse fraction and/or the plasticity of the fine fraction as determined by the Atterberg Limits. These characteristics are used because they can be measured readily with reasonable precision, and estimated with sufficient accuracy for descriptive purposes. They give a general indication of the probable engineering characteristics of the soil at any particular moisture content. Table 13 is a key to the naming and description of soils by hand and eye. It will be seen that where a soil (omitting any boulders or cobbles) contains about 35 % or more of fine material, it is described as a fine soil ("CLAY" or "SILT" dependent on its plasticity). With less than about 35 % of fine material, it is usually described as a coarse soil ("SAND" or "GRAVEL" dependent on its particle size grading). The description of soil containing boulders or cobbles is discussed in **41.4.4.2**.

The 35 % boundary between fine and coarse soils is approximate being dependent, primarily, on the plasticity of the fine fraction and the grading of the coarse fraction. Although the 35 % limit is used in other countries and is often reasonably appropriate, soils with the boundary as low as 15 % are not unknown (see **41.2**). All soils should be described in terms of their likely engineering behaviour, the descriptions being supplemented with and checked against laboratory tests as required.

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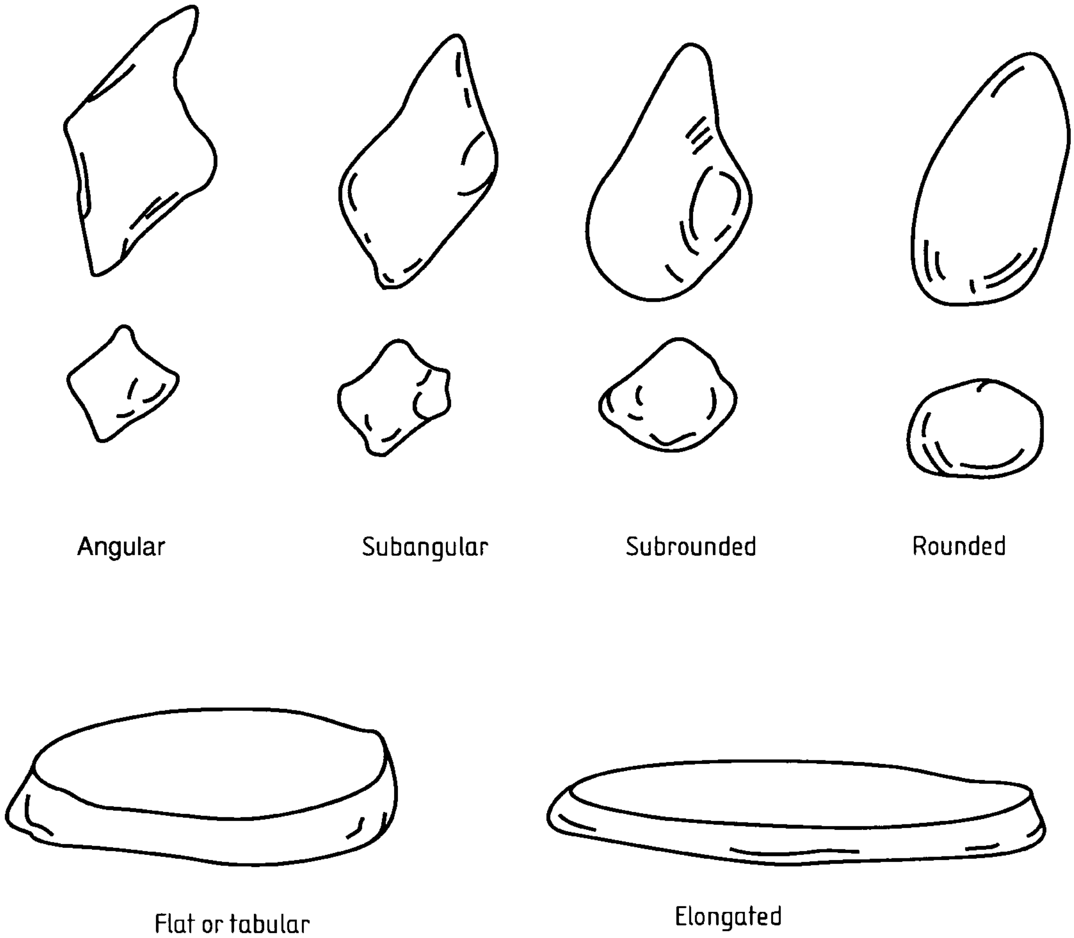

Figure 17 — Angularity and form terms

Table 13 sets out the basis for the identification of soils and should be studied in detail. The description of soils on the basis of particle size distribution and/or plasticity is described in the following clauses. In addition, soils may be classified (see clause **42**).

The basic soil types and their sub-divisions are defined by the range of their particle sizes as shown in Table 13 (see, in particular, column 8, Particle size). The naming of soils falling entirely within sands, gravels, cobbles or boulders is straightforward, as the particles are visible to the naked (see **43.2**).

A common difficulty arises in that the proportions of each soil fraction are percentages by weight, requiring adjustment from the percentage by volume seen by the eye. The naming of soils falling entirely within either clays or silts is less straightforward, relying on simple hand tests (see **43.3**). Most natural soils are composed of more than one soil type (see **41.4.4.2** to **41.4.4.5**). In composite soil types, cobbles and boulders are treated separately and are discounted in assessing proportions of the other components.

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41.4.4.2 *Deposits containing boulder-size and cobble-size particles*

When the "sample" is considered representative, such as may be the case in very large bulk samples from excavations or in excavated or exposed faces, then these deposits are described as follows:

	Main name	Estimated boulder or cobble content of very coarse fraction
Over 50 % of material is very coarse (> 60 mm)	BOULDERS	Over 50 % is of boulder size (> 200 mm)
	COBBLES	Over 50 % is of cobble size (200 mm to 60 mm)

The term boulder does not have an upper size limit, so dimensions should be given wherever available.

The proportion of cobbles in a boulder deposit (or vice versa) may be quantified using the terms "occasional", "some" or "many" as shown in the following table.

Mixtures of very coarse and finer material may be described by combining the terms for the very coarse constituent with those for the finer constituents as follows; percentages are approximate visual estimates in a field description and should only be taken as a subjective guide.

Representative sampling of soil mixtures containing very coarse soils is not possible in normal boreholes, and is very difficult even in conventional trial pits; a representative sample of a soil including boulders would need to weigh more than a tonne (BS 1377:1990). Cobbles or boulders are often noted only in passing on the driller's records, but frequently have a much greater significance to the engineering works (particularly piling or excavation). The location of individual cobbles and boulders should be noted on the log, even when it is considered appropriate to include the very coarse soil as part of the main description. Where possible, the characteristics of such cobbles and boulders should be described using the terms in clause **44**.

41.4.4.3 *Deposits containing gravel-size and sand-size particles*

As noted in **41.4.4.1**, a coarse soil (omitting any boulders or cobbles) contains about 65 % or more of coarse material and is described as a "SAND" or "GRAVEL", depending on which of the constituents predominates. The following terms may be used to describe the composition of the coarse fraction; percentages are by weight of the whole material less boulders and cobbles, and are approximate estimates in a field description.

The appropriate terms are used before the principal soil type. Further details should be provided at the end of the main description, after a full stop for clarity, e.g. "Medium dense brown very gravelly coarse SAND. Gravel is subangular fine and medium of sandstone and mudstone".

Term	Composition
BOULDERS (or COBBLES) with a little finer material[a]	up to 5 % finer material
BOULDERS (or COBBLES) with some finer material[a]	5 % to 20 % finer material
BOULDERS (or COBBLES) with much finer material[a]	20 % to 50 % finer material
FINER MATERIAL[a] with many boulders (or cobbles)	50 % to 20 % boulders (or cobbles)
FINER MATERIAL[a] with some boulders (or cobbles)	20 % to 5 % boulders(or cobbles)
FINER MATERIAL[a] with occasional boulders (or cobbles)	up to 5 % boulders (or cobbles)

[a] The description of "finer material" is made in accordance with **41.4.2** to **41.4.6**, ignoring the very coarse fraction; the principal soil type name of the finer material may also be given in capital letters, e.g. sandy GRAVEL with occasional boulders; COBBLES with some sandy CLAY.

Term	Principal soil type	Approximate proportion of secondary constituent
slightly sandy or gravelly	SAND or GRAVEL	up to 5 %
sandy or gravelly		5 % to 20 %
very sandy or gravelly		over 20 %
—	SAND and GRAVEL	about equal proportions

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41.4.4.4 *Deposits containing silt-size and clay-size particles*

Most fine soils are mixtures of clay and silt size particles; these can include silt size aggregates of clay minerals and clay size particles such as quartz. Soils formed solely of coarse silt are rare; such soils may not demonstrate plasticity, but should still be described as silt rather than fine sand, if the grains cannot be seen with the naked eye. The distinction between clay and silt is often taken to be the A-line on the plasticity chart, with clays plotting above and silts below (see Figure 18); however, the reliability of the A-line in this regard is poor, as might be expected [175] [176]. The effects of clay mineralogy and organic content are also significant. Fine soil should be described as either a "SILT" or a "CLAY" depending on the plastic properties; these terms are to be mutually exclusive and so terms such as "silty CLAY" are unnecessary and are not to be used. The field distinction between CLAY and SILT should be made using the hand tests in **43.3**. Where these hand tests are genuinely indecisive or ambiguous, the hybrid term CLAY/SILT may be used, which may flag up the need for additional plasticity determinations in the laboratory to provide further information. The hybrid terms "CLAY/SILT" and "SILT/CLAY" should be taken to be synonymous. Soil classifications, as distinct from soil descriptions, may use the A-line.

41.4.4.5 *Deposits containing mixtures of fine soil and coarse soil*

The following terms may be used to describe those common soils that include a mixture of soil types. The appropriate quantified terms should be used before the principal soil type. It is recommended that the dominant secondary fraction comes immediately before the principal soil term. To avoid ambiguity, if any of the constituent sizes require qualifying adjectives, these should be added in separate sentences after the main description; this commonly applies to the gravel or, less frequently, the sand fractions. Additional detail can be added in these sentences as appropriate, such as the estimated percentage proportion or consistency, e.g. "Gravelly very clayey SAND. Gravel (10 %) is fine of rounded quartz. Clay is firm".

The terms "silty" and "clayey" are mutually exclusive (as in **41.4.4.4**) in a coarse soil; which term is used is based solely on the plastic properties of the fine fraction (see **41.4.4.4**), e.g. "clayey gravelly fine SAND. Gravel is fine of mudstone". On the other hand, the terms "sandy" and "gravelly" may both be used, in which case the percentages are assessed separately, e.g. "slightly gravelly slightly sandy CLAY. Gravel is rounded of quartzite" means that the soil contains up to 35 % sand and up to 35 % gravel.

Soils that exhibit cohesion but have a high proportion of coarse particles, such as many glacial deposits, are difficult to describe. Descriptions with cumulative proportions of the various fractions exceeding 100 % are incorrect.

Term	Principal soil type	Approximate proportion of secondary constituent	
		Coarse soil	Coarse and/or fine soil
slightly clayey or silty and/or sandy or gravelly	SAND and/or GRAVEL		> 5 %
clayey or silty and/or sandy or gravelly			5 % – 20 %[a]
very clayey or silty and/or sandy or gravelly			> 20 %[a]
very sandy or gravelly	SILT or CLAY	> 65 %[b]	
sandy and/or gravelly		35 % – 65 %	
slightly sandy and/or gravelly		< 35 %	

[a] or described as fine soil depending on assessed engineering behaviour.

[b] or described as coarse soil depending on assessed engineering behaviour.

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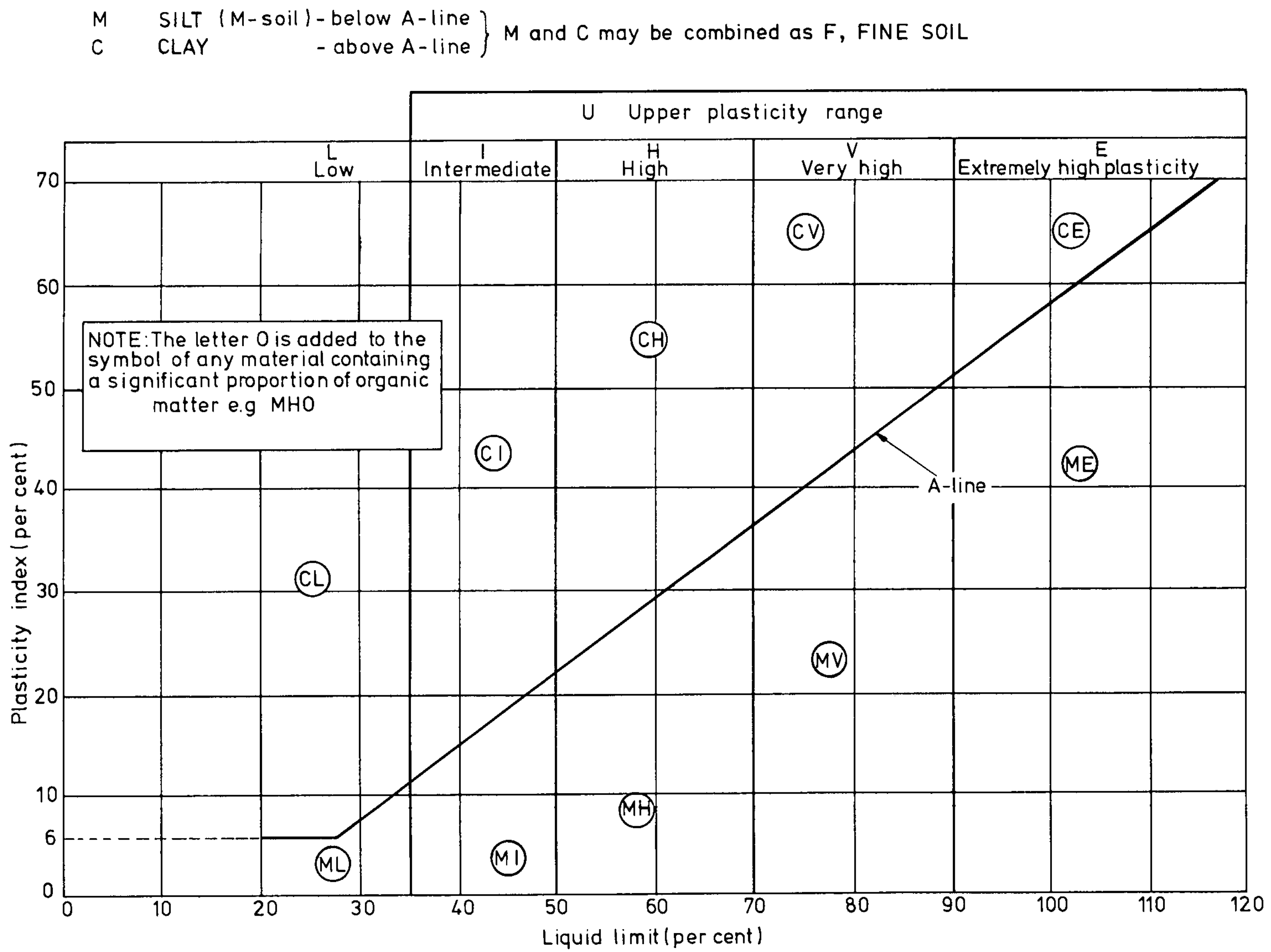

Figure 18 — Plasticity chart for the classification of fine soils and the finer part of coarse soils (measurements made on material passing a 425 μm sieve, in accordance with BS 410)

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41.4.4.6 *Minor constituents*

Where soils contain minor quantities of constituents, which are more likely to be relevant to the geology or practicalities of laboratory testing than to the engineering behaviour of the soil (for example less than about 10 % in a fine soil or 1 % in a coarse soil), these can be mentioned before the principal soil type, using qualitative terms such as "slightly calcareous", "glauconitic" or "very shelly", or at the end of the description using qualitative terms such as "rare", "occasional" or "frequent", e.g. "SAND with rare gravel size brick fragments". These qualitative terms are relative, for which no definition of percentage is given here, but should be consistent on any given job. In this regard, reference samples or photographs provide a useful record.

41.4.5 ***Man-made soils***

Man-made soils ("Made Ground" or "Fill") have been placed by man and can be divided into those composed of reworked natural soils and those composed of man-made materials. A common and useful distinction is that FILLS are placed in a controlled manner, and those called "made ground" are placed without strict engineering control. Mapping geologists, such as those in the British Geological Survey, often distinguish Made Ground as placed above and Fill as placed below the original ground surface.

The description and testing of reworked natural soils is usually straightforward. It is rarely possible, though, to carry out significant soil tests on man-made materials, and descriptions are all that remain after the samples have been discarded or pits filled in. Good descriptions are therefore of even greater importance with this type of material and should include information on the following aspects, as well as on the soil constituents (this list is not exhaustive);

a) origin of the material;

b) presence of large objects such as concrete, masonry, old motor cars etc;

c) presence of voids or collapsible hollow objects;

d) chemical waste, and dangerous or hazardous substances;

e) organic matter, with a note on the degree of decomposition;

f) odorous smell;

g) striking colour tints;

h) any dates readable on buried papers etc.;

i) signs of heat or combustion under ground e.g. steam emerging from borehole;

j) structure, variability and method of placement.

Some examples of descriptions of man-made soils are as follows; these illustrate the need for some flexibility in approach and word order.

"MADE GROUND comprising plastic bags, window frames, garden refuse, newspapers (1964)".

"MADE GROUND: dense brown sandy GRAVEL with occasional tiles, wire, glass, tyres".

"Soft grey sandy CLAY. Rare gravel size brick fragments (MADE GROUND)".

"Firm yellow brown slightly sandy CLAY with clods (up to 200 mm) of firm to stiff orange CLAY (EMBANKMENT FILL)".

41.4.6 ***Organic soils and peats***

Small quantities of dispersed organic matter can have a marked effect on plasticity and hence the engineering properties and produce a distinctive odour and dark colour tints; increasing quantities of organic matter heighten these effects [177]. If organic matter is present as a secondary constituent, the following qualifying terms may be appropriate [178].

Term	Organic content (% by weight)	Typical colour
Slightly organic clay or silt	2 – 5	Grey
Slightly organic sand	1 – 3	As mineral
Organic clay or silt	5 – 10	Dark grey
Organic sand	3 – 5	Dark grey
Very organic clay or silt	> 10	Black
Very organic sand	> 5	Black

Laboratory determinations of organic content are not essential to the description, but can be used to assist consistency of terminology if required.

Soils with organic contents up to about 30 % by weight and moisture contents up to about 250 % behave largely as mineral soils, albeit with different parameters [177] and [179]. Such materials are usually transported and would not be described as peat, which accumulates in-situ in a mire. This morphological distinction may be difficult to define, e.g. within a fluvial sequence, and therefore a distinction based on engineering behaviour should be made (see also **43.3.5**). The organic upper soil layer is usually referred to as "Topsoil".

Peat is classified according to the degree of decomposition and strength, as given in Table 13 (see the middle of the penultimate row, Organic soils). Inorganic soils may occur as secondary constituents of a peat, and should be described for example as slightly clayey or very sandy, these terms being used qualitatively here. If the peat forms a horizon of major engineering significance, a fuller description using the scheme of von Post may be appropriate [177].

The previous comments all refer to recent vegetable matter, not to older materials such as coal or lignite. It should also be borne in mind that dark colours and low densities can be associated with volcanic materials.

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41.5 Geological formation, age and type of deposit

A guide to the name of a geological formation (senso lato) is given on the maps of the British Geological Survey or its antecedents, and it should be written with at least capital initial letters, e.g. London Clay, Bagshot Beds, Lower Lias. Alternatively, the formation can be given in brackets and/or in upper case letters for clarity. The geological formation should be named where this can be done with confidence, but it may not be easy to tell to which formation a sample belongs, or to locate formation boundaries in a borehole or exposure; conjecture should be avoided, but degrees of uncertainty can be indicated. Revisions to stratigraphic nomenclature can cause problems in this regard. The published geological map or memoir should normally be used as a guide and is often adequate and acceptable. Specialized geological knowledge of a region may be required if a label other than that published is applied, for instance in the recent subdivisions away from Middle or Upper Chalk. On the other hand broad nomenclature changes can be applied more readily, e.g. Sherwood Sandstone for Bunter Sandstone.

The characteristic lithology is sometimes indicated in the formation name, e.g. London Clay, but it should be remembered that at a particular location or horizon the lithology or material type may be completely different from that indicated in the formation name. Formations may be quite variable in their lithology, and a knowledge of the formation indicates the possible range of material to be expected. Some indication may be obtained from the key to the one-inch or 1:50 000 and six-inch or 1:10 000, geological maps; from the Sheet Memoir, if published; and from the British Regional Geology Guide (see annex B).

A term indicating the geological origin or type of the deposit may be given on the map legend, e.g. Made Ground, Peat, Head, Alluvium, River Terrace, Brickearth, Blown Sand, Till. The term can indicate to the engineer some of the characteristics that the deposit may be expected to show.

41.6 Additional information

Any additional information on the composition, structure, behaviour or other characteristics of the soil that would be of value in assessing its nature and properties should be recorded. A special note should be made if the properties of the material are considered to be unusual in relation to the rest of its description. A note should also be made if there is doubt about whether the description is representative of the ground at the level from which it was sampled. This could be caused, for instance, by the fracture of particles or a loss of fines during sampling, or to the sample size or borehole diameter being too small in relation to the grading or structure of the material being sampled. Where relevant, it should be made clear whether the samples on which the description is based were disturbed or undisturbed.

42 Classification of soils

A full soil description (in accordance with clause **41**) gives detailed information on the colour, nature (plasticity and particle characteristics) of a soil, as well as on the state (strength condition) and structure (bedding, discontinuities) in which it occurs in a sample, borehole or exposure. Few, if any, soils have identical descriptions. For the purposes of engineering interpretation on a particular project it is often useful to classify the soils on the basis of geological origin of the strata, on some engineering property or properties of the strata, or on any of a large number of combinations of geological and engineering parameters. Such classifications are usually adopted to provide a framework for description and assessment of the ground conditions at a site or series of sites for the particular engineering problem in hand.

Probably the most common type of classification, however, places a soil in a limited number of groups with shorthand identifiers on the basis of grading and plasticity of disturbed samples. These characteristics are independent of the particular condition in which a soil occurs, and disregard the influence of the structure, including fabric, of the soil. For this and other reasons, such a classification may appear to differ from the field description of a soil determined in accordance with clause **41**, although it can be useful for those soils to be used as construction materials and is widely used in Europe, e.g. [180]. The more general classification scheme for this purpose, which was devised by the Department of Transport [29], is widely used in the UK. It is recommended that the field descriptions should normally stand as a record of the undisturbed character of the soil. The classification can provide additional useful information as to how the disturbed soil behaves when used as a construction material under various conditions of moisture content.

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43 Field procedures for description of principal soil type

43.1 Choice of procedure

For fieldwork where laboratory facilities are not available, and for the rapid assessment of soils in or out of the laboratory, or where laboratory tests are not warranted, judgement and the simple tests described below may be used in conjunction with Table 13 for the naming of soils. It is not intended that the procedures below replace laboratory testing, which is necessary to determine relevant properties as and when required; indeed, the results of the testing should be reported together with the original description. It is also desirable that operators check their results against laboratory tests from time to time to ensure that their judgement is sound.

43.2 Field assessment of grading

Coarse and fine soils are distinguished by whether they stick together when wet; the water content may need to be adjusted to correctly assess this. The coarse silt/fine sand boundary (0.06 mm) can be assessed by eye, the coarse silt particles only being visible with the aid of a hand lens.

It is easier to distinguish between gravels and sands, or between gravelly and sandy fine soils, because the size that distinguishes the gravel sizes from the sand sizes (2 mm) is easily visible. In visually assessing the particle size distribution, an additional judgement is required in order to report the relative proportions by weight rather than by volume; a ratio of 2.7:1.7 would often be appropriate. In addition the size of a non-equidimensional particle is that of the square sieve aperture through which it would pass. Particles of 2 mm size are approximately the largest that cling together when moist. Because of the capillary attraction of water, though, sand is not cohesive, as it is not possible to roll a thread of any strength.

43.3 Field assessment of plasticity

43.3.1 *Methods*

In the laboratory, the effect of the finer particles on the properties of a soil is assessed from measurements of the plasticity (Atterberg Limits) of the fraction finer than 0.425 mm. The distinction between clay and silt is then made using the plasticity chart, on which the A-line is an empirical boundary. This chart should be used with caution as the presence of different clay minerals, fine and medium sand or organic matter, for example, can affect the measured plasticity. For instance, increasing sand content decreases plasticity, whereas, conversely, increasing organic content increases plasticity. This effect on the measured plasticity is different when hand tests are used for a field assessment. In the field the extent to which the soil shows cohesion and plasticity may be used for the assessment of the fine fraction of coarse soils and dry strength, toughness and dilatancy may be used for the assessment of fine soils. The term "CLAY/SILT" should only be used when it is not possible to make the distinction between "CLAY" and "SILT", because the tests to try and establish this are genuinely inconclusive.

43.3.2 *Cohesion and plasticity of the fine fraction of coarse soils*

To examine a soil sample for these characteristics it should first be loosened; this could necessitate, for instance, crushing the soil with the foot or a mallet, and then moulding and pressing a handful of the material in the hands. It may be necessary to add water and to pick out the larger pieces of gravel. A soil shows cohesion when, with a certain moisture content, its particles stick together to give a relatively firm mass. A soil shows plasticity when, with a certain moisture content, it can be deformed without rupture, i.e. without losing cohesion. Clays, silts and some peats are cohesive and plastic; sands may cohere when wet but are neither cohesive nor plastic. It should normally be possible to distinguish between the presence of silt and the presence of clay. Soils plotting near the A-line tend to be classified as clays or clay/silts, as indicated in Figure 18.

43.3.3 *Toughness of fine soil*

Toughness refers to the character of a thread of moist soil rolled on the palm of the hand, moulded together, and rolled again until it has dried sufficiently to break at a diameter of about 3 mm, as in the plastic limit test. In this condition, inorganic clays of high plasticity are fairly stiff and tough; those of low plasticity are softer and more crumbly. Inorganic silts give a weak and often soft thread that breaks up, crumbles readily, and may be difficult to form. Organic soils and peat have a very weak, spongy or fibrous thread, which may be difficult to form at all, and their lumps crumble readily.

43.3.4 *The dilatancy test*

A pat of soil moistened to be soft, but not sticky, is held on the open horizontal palm of the hand. The side of the hand is then jarred against the other hand several times. Dilatancy is shown by the appearance of a shiny film of water on the surface of the pat. When the pat is squeezed or pressed with the fingers, the surface dulls as the pat stiffens and finally crumbles. These reactions are marked only for predominantly silt-size material and fine sand, and normally indicate the presence of these materials.

43.3.5 *Organic soil and peat*

Small quantities of dispersed organic matter in a soil can produce a distinctive odour and a dark grey, dark brown or dark bluish grey colour. With larger quantities of organic matter, fine soils usually plot below the A-line as organic silt. They have high, very high or extremely high liquid limits, sometimes extending up to several hundred per cent; and the liquid limit, plastic limit and plasticity index show a very marked drop on rewetting or remoulding following air or oven drying. Plant remains may be recognizable in the soil. Soils that consist predominantly of plant remains, either fibrous, or pseudo-fibrous or amorphous, may be described as peat.

Peats may be distinguished from organic soils by their low density (but see also **41.3.4**); soils with a high organic content may oxidize and change colour rapidly.

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44 Description and classification of rocks

44.1 The scope of rock description

As for soils, rock descriptions are made on samples recovered from boreholes and excavations and/or from examination of the in-situ materials. In the following clauses "material characteristics of rocks" refers to those visible in an intact block free from discontinuities, and "mass characteristics of rocks" refers to the overall structure including, particularly, the discontinuities. Thus, only material characteristics can be described on a hand specimen, but an in-situ exposure would permit description of material and mass characteristics. Samples or cores from boreholes normally only allow a limited description of mass characteristics. The quality of the observed sample or exposure is reflected in the level of detail in the description; it is essential that any doubts as to the representativeness or reliability of the sample be stated. An accompanying report should give the origin, type and quality of each sample. In this regard, it should be noted that, particularly for rocks, it is often the 5 % not recovered that may be more critical than the 95 % actually recovered.

The characteristics of a rock, which play a major role in determining its engineering properties and need to be given due attention when describing the rock, are the strength, weathering effects and the discontinuities. The discontinuities are the most significant of these (unless the discontinuity spacing is wide with respect to the engineering structure) and so particular attention is paid to this aspect.

The classification of, or nomenclature for, rock material used by geologists requires (often detailed) consideration of mineralogy and petrography, which may be of interest to engineers, but only in special circumstances. Engineering properties of rock are not included in and cannot be reliably inferred solely from this type of geological classification, although a particular rock name can often indicate a range of typical engineering characteristics. Geological classification of rock materials is necessary to appreciate the geological origin and structure of an area, to establish geological correlation between boreholes, and to distinguish boulders from bedrock.

This knowledge is also of importance when rock material is required for construction purposes, for example as building stone, concrete aggregate or roadstone.

The following clauses deal with the characteristics of rock material and rock mass that can be inferred from natural outcrops, excavations and rock cores. The amount of information that can be obtained from cores is usually limited, compared to in-situ exposures, unless special techniques are employed.

44.1.1 *General description*

Rocks seen in natural outcrops, cores and excavations should normally be described in the following sequence:

a) material characteristics (see **44.2**):
   1) strength;
   2) structure;
   3) colour;
   4) texture;
   5) grain size;
   6) rock name (in capitals, e.g."GRANITE");

b) general information (see **44.3**):
   1) additional information and minor constituents;
   2) geological formation;

c) mass characteristics (see **44.4**):
   1) state of weathering;
   2) discontinuities;
   3) fracture state.

44.2 Description of rock materials

44.2.1 *Strength of rock material*

A scale of strength, based on the uniaxial compressive test is shown in the following table.

The strength of a rock material determined in the uniaxial compression or point load test is dependent on the moisture content of the specimen, anisotropy and the test procedure adopted, all of which should be reported. The use of simpler index tests in the field is recommended to provide additional data and as a check on the manually assessed strengths; the Point Load Test [181] and Schmidt Hammer are amongst the more commonly used. The size and shape of lumps, strength of operator, weight of hammer and surface on which lumps rest affect the assessment of the strength. It is therefore vital that each logger ensures their descriptions are calibrated by strength determinations.

Term	Field definition	Unconfined compressive strength (MN/m^2)
Very weak	Gravel size lumps can be crushed between finger and thumb.	< 1.25
Weak	Gravel size lumps can be broken in half by heavy hand pressure.	1.25 to 5
Moderately weak	Only thin slabs, corners or edges can be broken off with heavy hand pressure.	0 5 to 12.5
Moderately strong	When held in the hand, rock can be broken by hammer blows.	12.5 to 50
Strong	When resting on a solid surface, rock can be broken by hammer blows.	50 to 100
Very strong	Rock chipped by heavy hammer blows.	100 to 200
Extremely strong	Rock rings on hammer blows. Only broken by sledgehammer.	> 200

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44.2.2 ***Structure***

The structure of the rock is concerned with the larger-scale inter-relationship of textural features and lithology. Common terms should be used where possible. Terms frequently used to describe sedimentary rocks include "bedded", "laminated"; metamorphic rocks may be "foliated", "banded"; igneous rocks may be "flow-banded".

Descriptive terms used for the thickness of these structures are as follows. Note that structure features are not synonymous with mechanical discontinuities, but they may include potential planes of weakness or incipient discontinuities in the rock mass (see **44.4.3**).

Term	Thickness
Very thick	greater than 2 m
Thick	600 mm to 2 m
Medium	200 mm to 600 mm
Thin	60 mm to 200 mm
Very thin	20 mm to 60 mm
Thickly laminated (Sedimentary) Narrow (Metamorphic and Igneous)	6 mm to 20 mm
Thinly laminated (Sedimentary) Very narrow (Metamorphic and Igneous)	less than 6 mm

For sedimentary rocks, structures such as bedding may be described as "thick beds" or "thickly bedded", for example, "a thickly bedded SANDSTONE". For igneous and metamorphic rocks, the appropriate descriptive term for the structure should be used, for example, "narrowly foliated GNEISS", "very thinly flow-banded DIORITE".

44.2.3 ***Colour***

Rock colours should be described according to the scheme for soils recommended in **41.3.2**.

44.2.4 ***Texture***

The texture of a rock refers to individual grains and their arrangement, which may show a preferred orientation. Terms frequently used include "porphyritic", "crystalline", "cryptocrystalline", "amorphous" and "glassy". Definitions of such terms are provided in standard geological dictionaries [182]. Geologists often subdivide texture into texture (geometric aspects of particles or crystals) and fabric (arrangement of grains), but this is seldom appropriate in field descriptions for engineering purposes.

Examination of the rock texture may require the use of a hand lens or the microscopic examination of a thin slice of the rock.

44.2.5 ***Grain size***

A descriptive classification scheme is given in Table 14. Quantified grain size boundaries are only appropriate in sedimentary and derived metamorphic rocks. In other rocks the grain size distinctions are relative. Grain size refers to the average dimension of the minerals or rock fragments dominating the rock's behaviour; this usually means the groundmass, but sometimes descriptions of the cement and grains may also be necessary. Where a rock has bi-modal grain sizes, the grains or clasts and matrix or groundmass should be described separately and linked by quantified terms, as given in clause **41**, together with an indication of which size of matrix or groundmass is likely to affect the engineering behaviour. It is usually sufficient to estimate the size by eye, which may be aided by a hand lens in the assessment of fine-grained or amorphous rocks. The limit of unaided vision is approximately 0.06 mm. The terms "fine", "medium" and "coarse grained" can be applied either to the whole range of grain sizes, e.g. "medium grained limestone", which means it has grains of 0.06 mm to 2 mm, or it can apply to a grain size subdivision, e.g. a "medium grained sandstone", meaning it is made up of grains 0.2 mm to 0.6 mm. Wherever necessary, the terminology used should be reported and explained to avoid any confusion.

44.2.6 ***Rock name***

An aid to the identification of rocks for engineering purposes is given in Table 14. The table follows general geological practice, but is intended as a guide only; geological training is required for the satisfactory identification of rocks. Engineering properties cannot be inferred directly from the rock names in the table, but the use of a particular name does indicate a likely range of characteristics to the reader. Combinations of rock names from Table 14 is possible, for instance in the siliceous sedimentary rocks "a sandy MUDSTONE", or combinations of clastic/calcareous/carbonaceous such as "sandy LIMESTONE" or "carbonaceous MUDSTONE".

44.3 General information

44.3.1 ***Additional information and minor constituents***

Much as for soils (see clause **41**), any other information or observations on the rock material should be included here (minor constituents, abnormal mineralogy, presence of vugs). Similar information relating to the rock mass should be included at the end of the description (e.g. face stability, voids). The size, spacing or proportion of any qualitative terms should also be given.

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Table 14 — Aid to identification of rocks for engineering purposes

Grain size mm	Bedded rocks (mostly sedimentary)				
Grain size boundaries approximate	Grain size description		At least 50 % of grains are of carbonate	At least 50 % of grains are of grained volcanic rock	
20 – 6 – 2	RUDACEOUS	CONGLOMERATE Rounded boulders, cobbles and gravel cemented in a finer matrix Breccia Irregular rock fragments in a finer matrix	LIMESTONE and DOLOMITE (undifferentiated) Calcirudite	Fragments of volcanic ejecta in a finer matrix. Rounded grains AGGLOMERATE Angular grains VOLCANIC BRECCIA	SALINE ROCKS HALITE ANHYDRITE
2 – 0.6 – 0.2 – 0.06	ARENACEOUS (Coarse, Medium, Fine)	SANDSTONE Angular or rounded grains commonly cemented by clay, calcitic or iron minerals Quartzite Quartz grains and siliceous cement Arkose Many feldspar grains Greywacke Many rock chips	Calcarenite	Cemented volcanic ash TUFF	GYPSUM
0.06 – 0.002	ARGILLACEOUS	MUDSTONE SILTSTONE Mostly silt	Calcareous mudstone Calcisiltite	Fine-grained TUFF	
< 0.002	ARGILLACEOUS	MUDSTONE	Calcareous mudstone Calcilutite CHALK	Very fine-grained TUFF	
Amorphous or crypto-crystalline		Flint: occurs as bands of nodules in the Chalk Chert: occurs as nodules and beds in limestone and calcareous sandstone			COAL LIGNITE
		Granular cemented - except amorphous rocks			
		SILICEOUS	CALCAREOUS	SILICEOUS	CARBON-ACEOUS

SEDIMENTARY ROCKS

Granular cemented rocks vary greatly in strength, some sandstones are stronger than many igneous rocks. Bedding may not show in hand specimens and is best seen in outcrop. Only sedimentary rocks, and some metamorphic rocks derived from them, contain fossils.

Calcareous rocks contain calcite (calcium carbonate) which effervesces with dilute hydrochloric acid.

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Table 14 — Aid to identification of rocks for engineering purposes *(continued)*

Igneous rocks: generally massive structure and crystalline texture				
Grain size description				
COARSE	GRANITE[1]	DIORITE[1,2]	GABBRO[1,2]	Pyroxe-nite Perido-tite
	These rocks are sometimes porphyritic and are then described, for example, as porphyritic granite			
MEDIUM	MICROGRANITE[1]	MICROIORITE[1,2]	DOLERITE[3,4]	
	These rocks are sometimes porphyritic and are then described as porphyries			
FINE	RHYOLITE[4,5]	ANDESITE[4,5]	BASALT[5]	
	These rocks are sometimes porphyritic and are then described as porphyries			
Amorphous Crypto - crystalline	OBSIDIAN[5]	VOLCANIC GLASS		
	Colour: Pale ← → Dark			
	ACID Much quartz	INTER-MEDIATE Some quartz	BASIC Little or no quartz	ULTRA BASIC

Increasing grain size ↑

Metamorphic rocks	
Foliated	Massive
GNEISS Well developed but often widely spaced foliation sometimes with schistose bands Migmatite Irregularly foliated; mixed schists and gneisses	MARBLE QUARTZITE GRANULITE HORNFELS AMPHIBOLITE SERPENTINE
SCHIST Well developed undulose foliation; generally much mica	
PHYLLITE Slightly undulose foliation; sometimes spotted SLATE Well developed plane cleavage (foliation) MYLONITE Found in fault zones, mainly in igneous and metamorphic areas	
CRYSTALLINE	
SILICEOUS	Mainly SILICEOUS

IGNEOUS ROCKS

Composed of closely interlocking mineral grains. Strong when fresh; not porous.

Mode of occurence: 1. Batholiths, 2. Laccoliths, 3. Sills, 4. Dykes, 5. Lava flows, 6. Veins

METAMORPHIC ROCKS

Generally classified according to fabric and mineralogy rather than grain size

Most metamorphic rocks are distinguished by foliation which may impart fissility. Foliation in gneisses is best observed in outcrop. Non-foliated metamorphics are difficult to recognise except by association.

Most fresh metamorphic rocks are strong although perhaps fissile.

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44.3.2 ***Geological formation***

A guide to the name of a geological formation is given on the maps of the British Geological Survey or its antecedents, and it should be written with at least capital initial letters, e.g. "Wilmslow Sandstone", "Middle Chalk", "Lower Lias". Alternatively, the formation can be given in brackets and/or in upper case letters for clarity. The geological formation should be named where this can be done with confidence, but it may not be easy to tell to which formation a core or exposure belongs, or to locate formation boundaries in a borehole or exposure; conjecture should be avoided but degrees of uncertainty may be indicated. The comments in **41.4** are also pertinent in applying formation names to rocks.

44.4 Description of rock masses

44.4.1 ***Introduction***

The description of rock masses requires information, additional to and following the description of the rock material, about discontinuities and other features of engineering significance. Such additional information includes:

a) details of the weathering profile;

b) a full description of the discontinuities or sets of discontinuities;

c) evaluation of the fracture state.

44.4.2 ***Weathering***

44.4.2.1 *State of weathering*

The description of the weathering of rocks is of particular importance in ground investigations as most construction on or in a rock mass is undertaken at shallow depth within the zone of surface weathering. Many attempts have been made to devise weathering grade scales for particular rock masses. Scales have been devised for granite [183], chalk [184] [185] [186], mudstones [170] and Mercia Mudstone [187]. Working parties of the Engineering Group of the Geological Society have also devised general scales [188] [189] [190]. The best method of classification of rock weathering is a continuing subject for debate. The relative merits of previous schemes are discussed in [189].

The primary requirement is that a full factual description be given of the degree, extent and nature of weathering (Approach 1 in [189]). Subsequent formal classification may often be inappropriate; indeed, although it is useful, it should only be applied where both well established and sufficient information is available, in order to make an unambiguous classification, and where it would be clearly beneficial to do so.

In any rock description, full details of the degree, extent and nature of weathering effects should be included so that readers can appreciate their influence on engineering properties. Prescriptive classification may be inappropriate in many cases, whereas factual description of weathering:

— is a mandatory part of the full description;

— is of use for subsequent classification;

— is often the only possible way of dealing with weathering where the full profile is not seen;

— should be carried out at material and mass scales as appropriate;

— assists interpretation of how the rock has reached its observed condition;

— provides information for separating rock into zones of similar engineering character.

On those occasions when it is apparent that changes in engineering properties have been caused by weathering, this should be highlighted. In the event that the cause is uncertain, then terms such as "probably" or "possibly" should be used. In addition to the "standard" terminology, a typical description includes, "non-standard English" descriptors commenting on whether features are due to weathering or not, or which weathering processes or combinations of processes may have resulted in the observed state of the rock. All "standard" terms should be used according to their defined meanings. To avoid confusion, terms that are used in the prescriptive classifications, such as "slightly", "highly", "completely" should not be used in this description. The features most commonly to be examined and reported on include the following:

Strength and reduction of strength should be reported using defined terminology. The inclusion of any direct or indirect strength measurements made is to be encouraged, whether the test used is "standard" or not. Where it is thought that the change is due to weathering, this information should be provided, for example "very weak within weathered zones", or, "generally strong but weak adjacent to weathered discontinuities" (the extent of any such feature should also be reported as a measurement wherever possible).

Colour and discoloration. The degree of colour change can be described using terms such as "faintly discoloured", "discoloured" or "strongly discoloured". The extent of colour change can be described using terms such as "locally discoloured " or "pervasively discoloured". These terms are not quantitatively defined for general use, although specific criteria can be applied if appropriate. It is often useful to provide additional information on, for example, the extent of colour change by reporting measurements of inward penetration from discontinuities. Comment should include the nature of the colour changes, and whether they are considered to be as a result of weathering, alteration, or some other process. Standard colour charts should be used where appropriate.

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The nature and extent of weathering products should always be described using the appropriate rock or soil descriptive terminology and measurement and should be quantified wherever possible.

Fracture state and changes therein should be reported using defined terminology. The reporting of actual measurements is encouraged, because these are more precise, although the terms provide a useful shorthand. Where it is thought that changes are attributable to weathering, this information should be provided, for example, "closely spaced, becoming very closely spaced due to weathering between 15.00 m and 15.75 m".

44.4.2.2 *Weathering classification*

The classification of weathering in rocks as diverse as, for example, karstic limestones, granites and shales, affected by different weathering process requires a variety of approaches for different situations and scales. Formal classification may often not be appropriate, and so is not mandatory. Classifications are often useful but should only be applied where well established, where there is sufficient information to classify unambiguously and to do so would be clearly beneficial. Where classification is used, the particular system adopted should be recorded. The approach to description and classification is given in the flow chart in Figure 19 [190]. Within the various classifications presented as Approaches 2 to 5 (Approach 1 is factual description), definitions of sub-classes in terms of typical characteristics are necessarily broad. Classes may be more rigorously defined by following local experience, site-specific studies or through reference to established schemes.

In logging cores, the distribution of weathering classes of rock material may be recorded [191]; distribution of weathering classes of the rock mass from which the cores were obtained has to be inferred from this type of evidence and this is not always possible. The weathering of a rock mass cannot be deduced from individual boreholes.

Distribution of weathering classes in a rock mass may be determined by mapping natural and artificial exposures. It should be borne in mind, however, that isolated natural exposures of rock, and excavations of limited extent, are not necessarily representative of the whole rock mass influenced by the project.

44.4.2.3 *State of alteration*

Common terms should be used where possible, e.g. kaolinized, mineralized. The terms used for the description of weathering grades of rock material may be used where appropriate, because in many instances the effects of alteration are not easily distinguished from those brought about by weathering.

A full petrographic determination involving microscopic examination of thin slices of the weathered or altered rock may be required to determine the suitability of the rock for particular purposes, for example as a concrete aggregate.

44.4.3 ***Discontinuities***

Discontinuities within the rock mass are, in most cases, of such primary importance to the rock's overall engineering properties, that it is advisable to identify and report the maximum possible amount of information from the investigation. Full and accurate description of recovered cores is required and more frequent use should be made of the borehole itself with downhole logging (geophysical, scanning) or cameras. Only by the use of such methods are aspects such as joint directions and aperture able to be identified at depth. In addition, exposures, whether existing or created for the investigation, should be used wherever practicable to look at the in-situ mass. A distinction can be drawn between "mechanical discontinuities", which are already open and present in the rock, and "integral discontinuities", which are built-in potential planes of weakness. Full recommendations for the recording of discontinuities are given in [192] [193] [194] [195].

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APPROACH 1: FACTUAL DESCRIPTION OF WEATHERING (MANDATORY)

Standard descriptions should always include comments on the degree and nature of any weathering effects at material or mass scales. This may allow subsequent classification and provide information for separating rock into zones of like character. Typical indications of weathering include

- Changes in colour
- Reduction in strength
- Changes in fracture state
- Presence, character and extent of weathering products

These features should be described using standard terminology, quantified as appropriate, together with non-standard English descriptions as necessary to describe the results of weathering. At the mass scale the distribution and proportions of the variously weathered materials (e.g. corestones vs matrix) should be recorded

Can classification be applied unambiguously?

No — DO NOT CLASSIFY

Yes

Rock is moderately strong or stronger in fresh state

APPROACH 2: CLASSIFICATION FOR UNIFORM MATERIALS

Grade	Classifier	Typical characteristics
I	Fresh	Unchanged from original state
II	Slightly weathered	Slight discoloration, slight weakening
III	Moderately weathered	Considerably weakened, penetrative discoloration Large pieces cannot be broken by hand
IV	Highly weathered	Large pieces cannot be broken by hand Does not readily disaggregate (slake) when dry sample immersed in water
V	Completely weathered	Considerably weakened Slakes Original texture apparent
VI	Residual soil	Soil derived by in situ weathering but retaining none of the original texture or fabric

Is a zonal classification appropriate and is there enough information available?

No — Use Rock Mass Classification if appropriate

Yes

APPROACH 3: CLASSIFICATION FOR HETEROGENOUS MASSES

Zone	Proportions of material grades	Typical characteristics
1100	% G I - III (not necessarily all fresh rock)	Behaves as rock: apply rock mechanics principles to mass assessment and design
2	>90 % G I -III <10% G IV - VI	Weak materials along discontinuities. Shear strength stiffness and permeability affected
3	50 % to 90 % G I - III 10 % to 50 % G IV - VI	Rock framework still locked and controls strength and stiffness; matrix controls permeability
4	30 % to 50 % G I - III 50 % to 70 % G IV -VI	Rock framework contributes to strength; matrix or weathering products control stiffness and permeability
5	<30 % G I - III >70 % G IV - VI	Weak grades will control behaviour. Corestones may be significant for investigation and construction.
6	100% G IV - VI(not necessarily residual soil)	May behave as soil although relict fabric may still be significant

Rock is moderately weak or weaker in fresh state

APPROACH 4 CLASSIFICATION INCORPORATING MATERIAL AND MASS FEATURES

Class	Classifier	Typical characteristics
A	Unweathered	Original strength, colour, fracture spacing
B	Partially weathered	Slightly reduced strength, slightly closer fracture spacing, weathering penetrating in from fractures, brown oxidation
C	Distinctly weathered	Further weakened, much closer fracture spacing grey reduction
D	Destructured	Greatly weakened, mottled, ordered lithorelics in matrix becoming weakened and disordered, bedding disturbed
E	Residual or reworked	Matrix with occasional altered random or 'apparent' lithorelics, bedding destroyed. Classed as reworked when foreign inclusions are present as a result of transportation.

APPROACH 5: SPECIAL CASES

For rocks whose weathering state does not follow the other patterns indicated here, such as karst in carbonates and the particular effect of arid climates

Figure 19 — Description and classification of weathered rock for engineering purposes

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Discontinuities are breaks, fractures or planes of weakness in the rock mass and include the following:

Type of discontinuity	Description
Joint	A discontinuity in the body of rock along which there has been no visible displacement.
Fault	A fracture or fracture zone along which there has been recognizable displacement.
Bedding fracture	A fracture along the bedding (bedding is a surface parallel to the plane of deposition).
Cleavage fracture	A fracture along a cleavage (cleavage is a set of parallel planes of weakness often associated with mineral realignment).
Induced fracture	A discontinuity of non-geological origin, e.g. brought about by coring, blasting, ripping etc.
Incipient fracture	A discontinuity which retains some tensile strength, which may not be fully developed or which may be partially cemented. Many incipient fractures are along bedding or cleavage.

The inclusion of induced and incipient discontinuities is important as they may indicate weakness within the mass, but they would normally not be included within the assessment of fracture state, see **44.4.4**. The conventional exclusion of such integral discontinuities from reported indices is conservative, but only for foundation studies; for bulk excavation studies, for instance, it may be preferable not to exclude them. If incipient or induced fractures are included in the fracture state, this should be clearly stated on the borehole log.

Discontinuities usually occur in more than one direction in a rock mass, and may be present as distinct sets. Borehole cores provide essentially one dimensional data on discontinuity spacing; exposures or orientated cores are usually needed for full evaluation of the discontinuity pattern.

The following features of discontinuities can be described. The amount of detail included depends on the quality of the exposure or core, whether it is representative and the requirements of the problem in hand. The descriptive terms are summarized in Table 15, which provides a checklist of terms in the order in which they should be used in a description. More detailed guidance can be found in [193] [194] [196] [197] from which the table is compiled.

Orientation: the convention dip direction/dip should be used, e.g. 015°/26°. In cores only dip can normally be determined unless core orientation or downhole measurement methods have been used.

Spacing: the descriptive terms on Table 15 should be used for discontinuity spacing in one dimension. The spacing should be measured for each joint set; the convention is to measure discontinuity spacings perpendicular to the discontinuities. In cores with steeply dipping discontinuities, it may only be possible to measure spacing along the core axis; if so, this should be stated.

The spacing of discontinuities in three dimensions may be described with reference to the size and shape of the rock blocks bounded by discontinuities. Rock blocks may be approximately equidimensional, tabular or columnar in shape. Descriptive terms may be used in accordance with the following.

The use of these terms requires an understanding of the distribution of discontinuities in three-dimensions; in consequence they cannot be used in the description of drill core.

First term	Dimension
Very large	Greater than 2 m
Large	600 mm to 2 m
Medium	200 mm to 600 mm
Small	60 mm to 200 mm
Very small	Less than 60 mm
Second term	**Nature of block**
Blocky	Equidimensional
Tabular	One dimension much less than the other two
Columnar	One dimension much greater than the other two

Persistence: the descriptive terminology can be applied to sets; actual measurements are preferred for individual discontinuities. Very limited information is available from cores.

Termination: the nature of the discontinuity termination should be recorded in the context of the size of the exposure. A discontinuity may start and end within the exposure.

Roughness: descriptions should be made at three scales, where possible [193]. The intermediate scale (several metres) is divided into stepped, undulating or planar. The small scale (several centimetres) of rough, smooth or striated is superimposed on the intermediate scale. Smooth surfaces may be matt or polished; the degree of polish can be qualitatively described. The term striated should only be used where there is clear evidence of previous shear displacement. There may also be a large scale (several tens of metres) which is best reported as measured wavelength and amplitude; the smaller scales may be reported similarly. An individual joint may therefore be described as wavy (wavelength 12 m, amplitude 1 m), stepped (wavelength 2 m, amplitude 0.2 m) and smooth. If more precise detail is required, roughness can be measured quantitatively [193].

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Wall strength: use of index tests to measure wall strength is recommended. Numerical results should be reported, and can be summarized using the terms in **44.2.1**.

Wall weathering and alteration should be described in accordance with **44.4.2**.

Aperture and infilling: where possible, measurements of aperture should be reported. Full description of rock, soil or mineral infill should be provided. Care is required in reporting aperture in rock cores; the observer o should comment on whether the reported apertures are present in the intact rock mass, or a consequence of geomorphological/weathering agencies, or whether due to engineering activities or creation of the exposure. The thickness and type of infill should be reported using standard terms, e.g. 1 mm surface film of calcite, 10 mm cemented breccia, stiff brown sandy clay.

Number of sets: the descriptive terminology can be applied to individual discontinuities, or summarized to sets or to zones of uniform character.

Where extensive detail of the rock mass is required, systematic record sheets may be appropriate and recording as numerical data for use in rock mass rating schemes can facilitate data handling.

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Table 15 — Terminology and checklist for rock discontinuity description

Spacing	Orientation	Persistence	Type of termination	Roughness	Wall strength	Aperture	Filling	Seepage	No. of sets
Extremely wide > 6 m	Dip amount only in cores	Discontinuous	Cannot normally be described	Small scale (cm) and intermediate scale (m) Stepped Rough Smooth Striated	Schmidt hammer	Cannot normally be described in cores	Clean	Cannot be described in cores	Cannot be described in cores
Very wide 2 to 6 m							Surface staining (colour)		
Wide 600 mm to 2 m		Continuous in cores		Undulating Rough Smooth Striated			Soil infilling (describe in accordance with **41**)		
Medium 200 to 600 mm	Take No. of readings, of dip direction/dip e.g. 015/08° Report as ranges and on stereo net if appropriate	Very high > 20 m	Termination x (outside exposure) r (within rock) d (against discontinuity)		Point load test	Very open > 10 mm		Moisture on rock surface	Record spacing and orientation of sets to each other and all details for each set
Close 60 to 200 mm		High 10 to 20 m		Planar Rough Smooth Striated		Open 2.5 mm to 10 mm	Mineral coatings (e.g. calcite, chlorite, gypsum etc.)	Dripping water	
Very close 20 to 60 mm		Medium 3 to 10 m		Large scale (dm) Waviness Curvature Straightness	Other index tests	Moderately open 0.5 mm to 2.5 mm	Other - specify	Water flow measured per time unit on an individual discontinuity or set of discontinuities	
Extremely close < 20 mm		Low 1 to 3 m			Visual assessment	Tight 0.1 mm to 0.5 mm Very tight < 0.1 mm	Record width and continuity of infill	Small flow 0.05 - 0.5 l/s Medium flow 0.5 - 5.0 l/s	
Take number of readings state min. average and max.		Very low < 1 m	Record also size of exposure	Measure amplitude and wavelength of feature		Take number of readings state min. average and max.		Strong flow > 5 l/s	

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44.4.4 ***Fracture state***

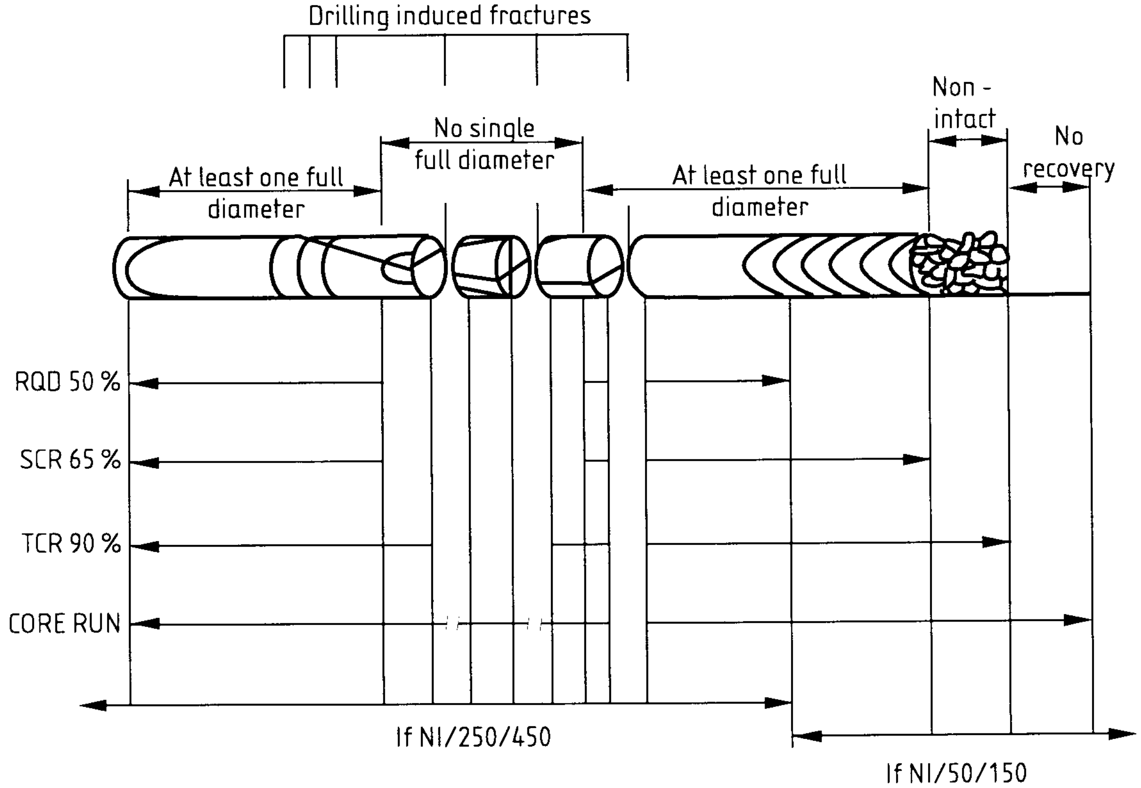
Various criteria may be used for quantitative description of the fracture state of rock cores; these are the total core recovery (TCR), solid core recovery (SCR), fracture index and Rock Quality Designation (RQD). The fundamental definition is that for solid core. Solid core has a full diameter, uninterrupted by natural discontinuities, but not necessarily a full circumference and is commonly measured along the core axis or other scan line [193] [198]. By this definition, core is solid unless intersected by more than one joint set with different strike directions.

The standard indices are defined as follows:

TCR (%)	ratio of core recovered (solid and non intact) to length of core run
SCR (%)	ratio of solid core recovered to length of core run;
RQD (%)	ratio of solid core pieces longer than 100 mm to length of core run
Fracture Index	a count of the number or spacing of fractures over an arbitrary length of core of similar intensity of fracturing. Commonly reported either as Fracture Index, (FI, number of fractures per metre) or as Fracture Spacing (I_f, mm).

The application of these terms is illustrated on Figure 20 [199]. It is conventional to include only natural fractures in determining these indices; departure from this convention should be stated on the log, as should any uncertainties. Useful guidance on the interpretation of natural and induced fractures is provided in [200]. The treatment of incipient discontinuities should be defined. The fracture state of in-situ exposures would normally follow the general procedures below, but RQD and Fracture Index can be determined from scanlines where appropriate. It is not usually appropriate to record these indices, other than TCR, in soils recovered by rotary core drilling.

Alternative definitions or applications of these indices, and particularly of RQD, have been put forward widely in the literature. If any alternative definitions are used this shall be indicated on the borehole or exposure log.



NOTE All features shown are natural discontinuities unless stated otherwise.

Figure 20 — Application of fracture state terms for rock cores

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44.4.5 ***Example rock material and rock mass descriptions***

An example of a description of a rock mass seen in a section of drill core might be "Very strong thinly flow banded dark greyish-green fine-grained quartz DOLERITE. Very widely spaced joints dipping 5 degrees with red penetrative staining to 10 mm and locally weathered to moderately strong to 5 mm penetration". The borehole log also includes the indices giving the fracture state.

An example of the description of a rock mass seen in a trial pit might be "Very stiff fissured brown mottled grey CLAY grading to very weak grey mottled brown MUDSTONE. Thickly laminated to very thinly bedded. Occasional gypsum crystals up to 5 mm, and rare pyritized wood fragments. Fissures very closely spaced (20 mm to 40 mm) with brown oxidation penetrating up to 3 mm. (Class B London Clay)".

An example of the description of a rock mass seen in a quarry face might be "Moderately weak very thinly bedded red brown fine and medium grained SANDSTONE. Rare moderately weak light green siltstone elliptical inclusions up to 20 mm by 5 mm. (Grade III Sherwood Sandstone). Small blocky jointing. Joint set 1 — 045/75, medium spaced, medium persistence, terminations outside exposure, curved planar rough, weak friable up to 5 mm penetration moderately open, clean. Joint set 2 — 110–130/80–90, closely spaced, low persistence, terminations outside exposure and against discontinuity, planar smooth weak friable up to 3 mm penetration, tight, clean. Bedding fracture set 3 — 180–190/0–10, medium spaced, high persistence, no termination seen, straight stepped smooth, slightly polished, moderately open up to 1 mm infilled with firm grey clay. Joints generally dry, locally dripping". If this is from a cored borehole, the log should also include the indices giving the fracture state; comparable information can also be obtained from scanlines on an exposure.

Such a description could refer to the rock mass in a specified location in a quarry. An account of the whole quarry would require many such descriptions perhaps displayed on engineering geological maps, plans and sections. Data on the orientation of discontinuities can be displayed and analysed using stereonets [201] or rosette diagrams.

In addition to classification on a geological basis, the assessment of rock mass conditions can be summarized and assessed for engineering purposes using one of the many available rating systems. By ascribing weighted scores to certain pertinent characteristics of the rock mass, a classification or zonation of the rock in the area influenced by the engineering works can be derived. A useful summary of these ratings is given in [202].

45 Legends for engineering geological maps and plans

45.1 Symbols for soils and rocks

Symbols are listed in Table 16 for the principal soil and rock types that are likely to be encountered in the United Kingdom. The symbols are simple and distinctive, permitting the same basic ornament to be used for unlithified (soil) and lithified (rock) states. They combine easily into symbols for composite types of soils and rocks.

The symbols are based upon those given in the previous edition of this standard, and elsewhere [188], with some alterations.

45.2 Special symbols for the borehole record

Recommended symbols for borehole records are given in Table 17.

45.3 Suggested symbols for structural features on plans

45.3.1 ***Symbols for general planar structures***

Recommended symbols for general planar structures are given in Table 19. For each planar structure the long bar of the symbol indicates the strike direction, and the short bar the dip direction; the dip amount is given in degrees measured from the horizontal. Formerly, the dip arrow was used exclusively to indicate the direction and amount of dip of bedding planes. It is still used occasionally and provides an acceptable alternative to the bar symbol. Bedding, foliation, banding and cleavage in sedimentary and metamorphic rocks may be crumpled, corrugated or undulating, although the general disposition may be horizontal, inclined or vertical. These conditions may be indicated by sinuous strike bars.

45.3.2 ***Traces of geological structures and geological boundaries***

Recommended symbols for geological structures and geological boundaries are given in Table 19. A distinction is usually made on geological maps between boundaries of drift (superficial) deposits and boundaries of solid deposits. Some indication is usually given of the accuracy of boundaries, broken lines denoting uncertainty in the positions of solid geological boundaries and faults. This principle may be applied to the trend, and where appropriate to the position, of the traces of other planar structures.

On large-scale engineering geological plans, faults and fault zones do not call for distinctive structural symbols. They are usually mapped as zones of which the margins are plotted and the internal structures and filling materials are mapped in detail. The symbols in Table 19 may be used to indicate the margins of the fault and the same principle may be applied to the details on the borehole record.

Additional symbols may be found elsewhere [188].

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Table 16 — Recommended symbols for soils and rocks

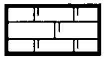
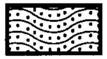
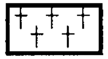
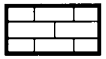
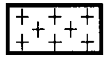
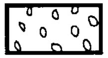
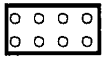
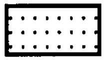
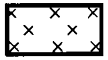
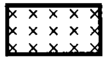
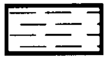
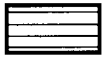
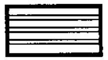
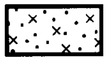
SOILS		ROCKS							
		SEDIMENTARY			METAMORPHIC		IGNEOUS		
	Made ground			Chalk		Coarse grained		Coarse grained
	Boulders and cobbles			Limestone		Medium grained		Medium grained
	Gravel	Rudaceous		Conglomerate		Fine grained		Fine grained
		Rudaceous		Breccia				
	Sand	Arenaceous		Sandstone				
	Silt	Arenaceous		Siltstone				
	Clay	Argillaceous		Mudstone				
	Peat	Argillaceous		Shale				
		Argillaceous		Coal				
NOTE Composite soil types will be signified by combined symbols, e.g.				Pyroclastic (volcanic ash)				
	Silty sand			Gypsum, Rocksalt etc.				

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Table 17 — Special symbols for borehole record

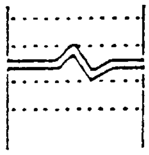
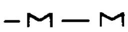
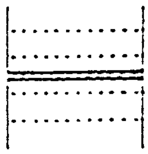
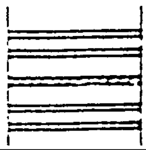
	Fault	Examples: 	Medium-grained igneous faulted against coarse-grained metamorphic rock
	Slip surface		Fault in sandstone
	Marine band		Slip surface in sandstone
	Shell band		Slip surface in shale

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Table 18 — Symbols for general planar structures

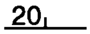
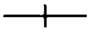
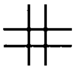
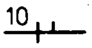
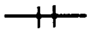
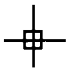
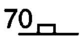
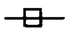
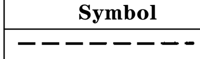
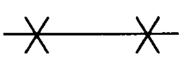
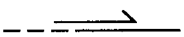
	Horizontal strata
	Inclined strata, dip in degrees, long axis is strike direction
	Inclined strata, dip in degrees, alternative symbol
	Vertical strata, long axis is strike direction
	Foliation or Banding, horizontal
	Foliation or Banding, inclined, dip in degrees, long axis is strike direction
	Foliation or Banding, inclined, dip in degrees, alternative symbol
	Foliation or Banding, vertical, long axis is strike direction
	Cleavage, horizontal
	Cleavage, inclined, dip in degrees, long axis is strike direction
	Cleavage, inclined, dip in degrees, alternative symbol
	Cleavage, vertical, long axis is strike direction
	Joint, horizontal
	Joint, inclined, dip in degrees, long axis is strike direction
	Joint, inclined, dip in degrees, alternative symbol
	Joint, vertical, long axis is strike direction

Table 19 — Symbols for geological structures and boundaries

Symbol	Detail	
	Geological boundary, drift	
	Geological boundary, solid (broken lines denote uncertainty)	
	Axial trace of anticline	
	Axial trace of syncline	
	Fault, crossmark on downthrow side, dip in degrees, throw in metres	(broken lines denote uncertainty)
	Fault, with horizontal component or relative movement	

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Section 7. Reports and interpretation

46 Field reports

The essential requirement of any field report form is that it should help and encourage the operator to record all the data necessary for the eventual interpretation of the borehole or field test. The information to be recorded in the field is detailed in the lists that follow. Report forms should be provided in the field for recording information, and these should have prompts (questions, spaces or boxes) to ensure that all the required information is recorded at the time. Most of the information called for is needed to draw appropriate conclusions from the results of boring and field tests, and whatever layout is adopted, all such items should be recorded somewhere. Other data, which may be needed for administrative purposes, have not been considered.

It is important that the reference number, location and depth of each test be recorded. The location may take the form of a reference that can be related to the national or other appropriate grid, and marked on a drawing in such a way that it can be easily located at a later date if needed. The ground level at the test position should also be measured and included where available; this is often not surveyed until after the test, though, so it may not be available for inclusion in the field report.

Drilling foremen, technicians, engineers or geologists, as appropriate, fill in field report proformas. Certain of the more complex tests may well have several proformas, and are often referenced on more than one report; for instance bearing or permeability tests in boreholes or trial pits. It is usual to require copies of the field reports to be made available soon after the tests, normally within a few days, or as specified, to demonstrate that the field data has been satisfactorily collected. These then form a complete contemporaneous record (and as such can be included in the final report), although further processing of the data is usually necessary to derive a satisfactory presentation for the descriptive report and the parameters for use in design.

Separate field report forms are required for the following common operations (this list is not exhaustive):

— cable percussion boring;

— rotary drilling (coring, open holing or percussive);

— trial and inspection pits;

— vane shear tests;

— permeability tests (variable head, constant head, packer injection or pumping);

— penetrometers (static CPT, dynamic, piezocone, seismic);

— pressuremeters (Menard, self-boring, dilatometer);

— in-situ density;

— overwater investigations by vibrocoring or grab sampling;

— geological fracture logging, including scan lines;

— calibration certificates and quality assurance records;

— bearing tests (CBR, plate bearing).

Separate drillers' daily report forms for cable percussion boring and rotary drilling are considered essential. The practice of using one form for both purposes inevitably leads to much necessary data not being recorded, particularly in rotary drilling. It is not uncommon for drillers to keep their records on odd scraps of paper while boring is in progress and to make up their daily report forms from these notes at the end of the day. This practice should be strongly discouraged. Its main dangers can be greatly reduced by providing the driller with a standard notebook for use during boring, which can be checked against his daily report at a later date if necessary.

Although it is not appropriate here to provide a full list of the information that should be recorded in the field for all operations, the following information should be recorded wherever applicable:

— contract title and departmental reference number (if appropriate);

— contractor's name and contract reference number;

— exploratory location/test reference numbers, location or co-ordinates, diameter or size, sketch plan and distances to referenced objects;

— orientation of open excavations;

— type of operation or test;

— dates of operation and weather;

— details of equipment in use, including serial numbers where appropriate;

— data of hole stability and details of casing/shoring used;

— depth and descriptions of strata;

— records of groundwater and flushing media;

— depth (depth ranges) of samples, sampling effort required (see section 3);

— depth, results and observations of all in-situ tests (see section 3);

— contractors personnel involved;

— any other pertinent observations.

More detailed information, such as the procedures to be followed and the reporting requirements for the test results, is usually outlined in the specification or standard in accordance with which the test is being carried out.

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47 Report

47.1 General

Interpretation is a continuous process, which should begin in the preliminary stages of data collection and should proceed as information from the ground investigation becomes available. By using this information it is often possible to detect and resolve anomalies as field and laboratory work progresses. Engineering problems should be considered as the data becomes available, so that the engineer in charge of the investigation can decide either what additional exploration and testing needs to be carried out or, where appropriate, what reductions in his original programme are possible.

When an engineering interpretation and recommendations are required, the report is prepared in two distinct and separate parts: one a descriptive report covering the procedures employed and the data obtained; and the other the analysis, conclusions and recommendations. An intermediate phase between these two comprises a thorough description of the ground and groundwater conditions, together with an assessment of material and mass ground parameters (see **47.3**). On larger investigations, or where otherwise appropriate, these parts can be separately bound.

47.2 Descriptive report

47.2.1 *Report as record*

When preparing a report it should be borne in mind that soon after it is written, which is a few months at most, when all the samples have been destroyed or rendered unrepresentative, the report is the only record of what was found. The results are normally issued in the form of a limited number of bound copies of an official written report. In future years, an increasing number of reports and the data contained therein will be presented on disk as computer files. Printed copies of the reports should be retained by interested parties, as these remain the definitive document; the data presented on electronic media is to aid flexibility and data handling. The question of electronic transfer of data, however, is currently changing; [203] explains the position. The formal report contains a description of the site and the procedures used, together with tables and diagrams giving the results. In addition, there are field and laboratory report forms and data sheets, which provide a detailed record of the data obtained. These forms are not usually included in the formal report, but should be preserved for a suitable period so that they are available for later reference. The practice of binding copies of these forms as a report volume for permanent reference has much to commend it; the client's requirements in this regard should be specified in the tender document.

47.2.2 *Introduction*

The report should have an introduction stating for whom the work was done, the dates and nature of the investigation, and its purpose and general location.

47.2.3 *Description of site*

The report should contain an unambiguous description of the geographical location of the site, so that the area covered can be readily located at a later date, when possibly most, if not all of the existing landmarks have disappeared. Where appropriate this should include street names and the National Grid reference. It may also include a reproduced section of the relevant Ordnance Survey map of the appropriate scale for clarity. Details of all relevant topographic features should be included.

The description should also include details of what was standing on the site at the time of the investigation and information on its past use, including the possibility or knowledge of any contaminated ground or landfill gases. In addition, details of any past or present man-made underground features, such as basements, mineral or other extractive workings, access or drainage adits and other tunnels, should be included. Some comment should be made on the relative levels between the site and its surroundings, and whether there are conspicuous differences in level over the site itself. Where actual levels relevant to the Ordnance Datum are available, these should also be given. A full description of the site often needs to include comment on features outside the site boundary.

47.2.4 *Geology*

An account should be given of the geology of the site, and the sources from which the information was obtained should be stated. The amount of the data included depends on the nature of the work being planned and the amount of available data. Information from previous ground investigations on or adjacent to the site should be emphasized. Where the information is scanty, it should either be given in total or a judicious selection made of those items most relevant to the problem. The soil and rock types identified and described in the report should be linked with the known geology of the site. Where published information and current usage provide conflicting geological nomenclature, this should be explained and the terminology selected for use in the report put into context.

47.2.5 *Fieldwork*

An account should be given of the methods of investigation and testing used. It should include a description of all the equipment used, e.g. types of drilling rigs and tools, together with the relevant standards for testing, sampling or drilling to which the work has been carried out. A note should be made of any difficulties experienced, e.g. problems in recovering samples. It is essential that any testing for gases and other contaminants, or observations of these in the boreholes and around the site generally be reported. The dates when the exploratory work was done should also be recorded, together with a note about the weather conditions, if relevant. The report should contain a drawing indicating the positions and ground levels of all pits, boreholes, field tests, etc. It should contain sufficient topographical information so that the several positions can be located at a later date.

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47.2.6 *Borehole logs*

47.2.6.1 *General*

The borehole log should be a record that is as objective as possible of the ground conditions at the borehole position before the ground was subjected to disturbance and loss by the boring process. Some interpretation is necessary, if only to link sets of samples with a stratum description, but the degree of interpretation should be kept to a minimum; unless this is necessary to provide information, when it should be clearly identified; the use of words such as "probably" or "possibly" is useful in this regard. The borehole log should be taken to be part of the factual data arising from the investigation. The final borehole logs should be based on the visual examination and description of the samples, the laboratory test results, the driller's daily report forms and what is known of the geology of the site. The logs can only be finalized when the appropriate field and laboratory work has been completed. It is important that all the relevant data collected by the driller, once checked and amended where necessary, should be recorded.

There can be no hard and fast rules on the method of presentation of the data. In principle, the borehole logs should present all the data obtained in a readable form and give a picture in diagrams and words of the ground profile at the particular point where the hole was bored. The extent to which minor variations in soil and rock types should be recorded, together with any discrepancies and discontinuities, depends on the various purposes to which the information is put. The use of detail indents on the borehole log to clearly represent variability or complexity of the ground is recommended.

Most organizations carrying out site investigations have standard forms for borehole logs. It is seldom practical in these to make allowance for all data that may need to be recorded, so it is important for an adequate remarks column to be available, which allows a record to be made of items not specifically covered. A standard form can be made more flexible by leaving one or more columns without headings, so that these can be used for whatever particular data needs to be recorded. Full use should be made of any unused space on the last sheet of a log. Examples of composite boring and drilling logs are given in Figures 21 and 22 with an explanation of the recommended or typical symbols, abbreviations and legends appearing in Tables 19 and 20.

47.2.6.2 *General data common to all logs*

The following should be recorded on all logs:

a) title of investigation;

b) report number;

c) name of client;

d) location detailed by a national or site grid reference;

e) date of start and finish of boring;

f) unique borehole number and sheet number, e.g. sheet 2 of 2;

g) type of boring, e.g. cable, percussion or rotary;

h) details, including sizes, of boring tools or drilling equipment used;

i) ground level related to Ordnance Datum;

j) diameter of borehole and/or types of core barrel including depths of any reductions in size;

k) diameter of casings and depth to which taken;

l) a depth scale so that the depth of sampling, tests and change in strata can be readily determined;

m) depth of termination of borehole;

n) details of backfilling or instruments installed;

o) ground and surface water records;

p) details of all samples and tests taken.

47.2.6.3 *Legend and symbols*

The ground profile should be illustrated by means of a legend using the symbols given in Tables 19 and 20.

Apart from those symbols just mentioned, the code makes no recommendations for the many others required in the preparation of borehole logs. Many different types are in use and, provided an adequate key is given with every set of borehole logs, there should be no difficulty in interpreting them. The key to symbols used may be given on a separate sheet or on each sheet.

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	BOREHOLE No. 1
Equipment & Methods Hand dug inspection pit to 0.6 m. Cable tool boring 200 mm dia. to 8.50 m then 150 mm dia. to 12.00 m. Rotary drilling 76 mm dia. core (T6H with Coreline) to 21.50 m using water flush	Location No. 5903 Location CATCAIRN BUSHES, HAMPSHIRE
Carried out for Notable Developments Limited	Ground level 33.68 — Coordinates 5423 m E, 4256 m N — Date 17/10/89 to 21.10/89

Description	Reduced level	Legend	Depth (thick)	Sample/Tests Depth	Sample Type	Sample No.	Test	
Friable brown gravelly TOPSOIL	33.68		(0.40)					
	33.68		0.40	0.50	D	1		
Stiff fissured brown mottled yellow and light grey CLAY. Frequent rootlets. Fissures are very closely spaced, sub-vertical rough.				1.00 - 1.45	U	2		40 blows
				1.50	D	3		
			(2.70)					
(ESTUARINE DEPOSITS - DESICCATED CRUST)				2.00 - 2.50	B	4		
Firm brown and dark grey mottled CLAY. Occasional rootlets. (ESTUARINE DEPOSITS)	30.58		3.10	2.90 - 3.35	U	5		32 blows
			(0.80)	3.40	D	6		
	29.78		3.90	4.00	D	7		
Soft grey and dark grey CLAY with closely spaced sub-horizontal partings and thin laminae of light grey fine sand.				4.50 - 5.50	P	8		
				5.40			HVp HVr	25.32.10 6.10.3
			(4.60)	6.00 6.00	D	9		
(ESTUARINE DEPOSITS)				6.50 - 7.50	P	NR		
Occasional shell debris				8.00 - 8.45	U	10		
							PP	25.30.40.20 Water entry
Possibly medium dense, light brown slightly gravelly fine and medium SAND. Gravel is fine and medium of rounded quartz and sub-angular limestone.	25.18		8.50	8.45 8.50 8.70 8.70	D W	11 12		
			(1.70)	9.00 - 9.50	D	13	S N-9	SW = 60 1,1/-,1,3,5 5,6,9,7
(ALLUVIAL DEPOSITS)				10.00	D	14		

Remarks 1. Borehole geophysically logged by ANO completion 2. Piezometer installed	Logged by DRN
	Scale 1:50
Notes Materials are described in accordance with Appendices. For explanation of symbols and abbreviations see FIG. 1 (c) 25/08/97 16:52.00	FIG.

Figure 21 — Example 1 of typical borehole log

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	BOREHOLE No. 1
Equipment & methods As sheet 1	Location No. 5903 Location CATCAIRN BUSHES, HAMPSHIRE
Carried out for Notable Developments Limited	Ground level / Coordinates: As sheet 1 / Date

Description	Reduced level	Legend	Depth (thick)	Sample/Tests Depth	Sample Type	Sample No.	Test		Field records	
SAND (As sheet 1)			(1.70)	10.00	D	14	C		kRH	
Probably dense, slightly sandy sub-angular to rounded GRAVEL and COBBLES of quartz and limestone (ALLUVIAL DEPOSITS)	23.48		10.20	10.00 - 10.45			N=30		5.6/5.7.7.10.8	
			(1.10)	10.00 - 10.50	D	15				
				10.00 - 11.00	B	16				
				11.00	D	17	C			
Very weak to moderately weak, very thinly bedded grey fine and medium grained LIMESTONE. Recovered as gravel size fragments. Fracture surfaces stained orange brown (CARBONIFEROUS LIMESTONE)	22.38		11.30	11.00 - 11.40	D	18	30		kFH	
				11.60	D	19			6.8/12.11.30.50 for 25 mm	
			(1.40)	11.90 - 11.91						
				11.80 - 12.00	D	20			.25 for 10 mm	
	20.98		12.70	12.00 - 12.50	100 grey	30	0	0	NI	
Moderately weak thinly laminated black carbonaceous MUDSTONE. Sub-horizontal very closely spaced polished striated surfaces. Fractures closely spaced. 45° dip, smooth light orange stained (CARBONIFEROUS LIMESTONE)				12.50 - 14.00	100 grey	95	30	6		
			(2.30)	14.00 - 14.45 SPT sample D21					NI 175 200	S N=46 5.7/10.11. 12.13
				14.00 - 15.50	100 dark grey	100	50	25		CRB= 0.30 m
Strong thinly to medium bedded dark grey medium grained LIMESTONE. Fractures often coated with calcite crystals. Occasional voids up to 50 mm. 65 fracture with 50 mm clay infill. Fractures dip 90° up to 0.5 m long, stepped rough tight clean (CARBONIFEROUS LIMESTONE)	18.68		15.00	15.50 - 15.53 SPT	0				50 175 200	C50 25 for 10 mm/50 for 20 mm CS1 15.8-16.10
			(4.00)	15.50 - 18.00	0	90	80	80		CRB= 0.20 mm
					0				80 250 250	Kp1 18.00-19.50
LIMESTONE (as sheet 3)	14.68		19.00	18.00 - 20.00	70 grey	100	85	80	150 300 750	CS2 17.30-17.49
			(2.50)							

Remarks

Notes
Materials are described in accordance with Appendices. For explanation of symbols and abbreviations see FIG.

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Logged by DRN
Scale 1:50
FIG.

Figure 21 — Example 1 of typical borehole log *(continued)*

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	BOREHOLE No. 1
Equipment & methods As sheet 1	Location No. 5903 Location CATCAIRN BUSHES, HAMPSHIRE
Carried out for Notable Developments Limited	Ground level / Coordinates: As sheet 1 / Date

Description	Reduced level	Legend	Depth (thick)	Drilling records: TCR.	Drilling records: W	Mechanical core log: TCR.	Mechanical core log: SCR.	Mechanical core log: RQD	Mechanical core log: If.	Field records and test results
Strong to very strong medium bedded grey fine grained LIMESTONE. Fractures medium spaced, sub-horizontal, rough very tight, clean (CARBONIFEROUS LIMESTONE)			(2.50 pen)	21.00 - 21.50	100 grey	100	100	100	100	kP1 20.00 - 21.50
	12.18		21.50							
BOREHOLE ENDS AT 21.50 m										

Remarks	Logged by DRN
	Scale 1:50
Notes Materials are described in accordance with Appendices. For explanation of symbols and abbreviations see FIG. (c) 25/08/97 16:52.00	FIG.

Figure 21 — Example 1 of typical borehole log *(continued)*

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	BOREHOLE No. 1
Equipment & methods As sheet 1	Location No. 5903 Location CATCAIRN BUSHES, HAMPSHIRE
Carried out for Notable Developments Limited	Ground level; Coordinates: As sheet 1; Date

Water level observations during boring					
Date	Time	Depth of hole, m	Depth of casing, m	Depth of water, m	Remarks
17/10/89	18.00	4.00	2.50	Dry	
18/10/89	08.00	4.00	2.50	Damp	
18/10/89	13.00	10.00	10.00	0.00	Before kRHT
18/10/89	14.45	11.00	11.00	1.35	Before kFHT
18/10/89	17.00	12.00	12.00	9.30	
20/10/89	08.00	12.00	12.00	2.50	
20/10/89		18.00	12.50	0.40	Before kP1
20/10/89	18.00	18.00	14.00	10.00	
21/10/89	08.00	18.00	14.00	11.00	
21/10/89	12.00	21.50	14.00	1.75	Before kP1
21/10/89	15.00	21.50	14.00	1.05	End of borehole

Hole diameter by depth table			
Depth of hole, m	Dia. of hole, m	Dia. of casing, m	Depth of casing, m
8.50	200	200	8.00
12.00	150	150	12.00
21.50	100	100	14.00

Water strike table							
Depth of strike, m	Casing depth, m	Date	Time	Post strike depth, m	Minutes after strike	Sealed at m	
8.50	8.00	18/10/89	-	8.50	-	12.50	Water strike
8.50	8.00	18/10/89	-	7.90	5	-	
8.50	8.00	18/10/89	-	7.40	10	-	
8.50	8.00	18/10/89	-	6.90	15	-	
8.50	8.00	18/10/89	-	6.50	20	-	

Depth related remarks table		
Top depth, m	Top depth, m	Remarks
0.60	3.50	Water added to assist boring
8.50	12.00	Borehole kept topped up to ground level. Borehole probably piping
10.50	11.30	Chiseling for 1.5 h.
11.30	12.00	Chiseling for 1.8 h.

Remarks	Logged by
	Scale
Notes Materials are described in accordance with Appendices. For explanation of symbols and abbreviations see FIG. (c) 25/08/97 16:52.00	FIG.

Figure 21 — Example 1 of typical borehole log *(continued)*

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Start date: 17 October 1989 End date: 21 October 1989	Casing diameter: 200 mm to 8.00 m 150 mm to 12.00 m 100 mm to 14.00 m	BOREHOLE No. 1	
Drilling method: Cable percussion to 12.00 m Equipment: Rotary coring to 21.50 m TH6 core barrel, water flush	Borehole diameter: 200 mm to 8.50 m 150 mm to 12.00 m 100 mm to 21.50 m	National grid Coordinates: 5423.0 E 4256.0 N	Ground Level: 33.68 m OD

Date and time	Casing depth (m)	Depth to water (m)	Sample details: Depth (m) from to	Type	No.	U 100 Blows / SPT Blows/N	U 100 Rec. / SPT Drive	Description of strata	Depth (thickness) (m)	Level m OD
17/10 15.00	NIL	DRY	0.50	D	1		-	Friable brown gravelly TOPSOIL	(0.40) 0.40	33.28
	NIL	DRY	0.50 - 1.45	U	2	40	450	Stiff fissured brown mottled yellow and light grey CLAY. Frequent rootlets. Fissures are very closely spaced, subvertical, rough. (ESTUARINE DEPOSITS - DESSICATED CRUST)		
			1.50	D	3					
			2.00 - 2.50	B	4				(2.70)	
	NIL	DRY	2.90 - 3.35	U	5	32	450			
			3.40	D	6					
18.00 17/10	2.50	DRY						Firm brown and dark grey mottled CLAY. Occasional rootlets. (ESTUARINE DEPOSITS)	3.10	30.58
18/10 08.00		DAMP	4.00	D	7				(0.80)	
	2.50	DRY	4.50 - 5.50	D	8	-	1000	Soft grey and dark grey CLAY with closely spaced sub-horizontal partings and thin laminate of light grey fine sand. (ESTUARINE DEPOSITS)	3.90	29.78
			5.40			HV	(22/6)			
	5.50	DRY	6.00	D	9	FV	(25/6)	Occasional shell debris, from 6.00 m to 8.50 m		
	6.00	DRY	6.50 - 7.50	NR	-				(4.60)	
			8.00 - 8.45	D	10	-	450			
	8.00	6.50	8.70	D	11			Possibly medium dense, light brown slightly gravelly fine and medium SAND. gravel is fine and medium of rounded quartz and sub-angular limestone (ALLUVIAL DEPOSITS)	8.50	25.18
			8.70	D	12					
	9.00	0.00	9.00	D	13				(1.50)	
	10.00	0.00	10.00	D	14	C 30			10.00	23.68

Remarks
1. See key sheets for explanation of abbreviations and symbols
2. An inspection pit was excavated by hand to 0.6 m
3. Small amounts of water were added to assist boring from 0.6 m to 3.50 m
4. Ground water was encountered at 8.50 m rising to 6.50 m to 3.50 m
5. SPT blows were at 9.00 m 1.1.0.1.3.5.5.6.9.7. Test was extended due to initial blows. Final 300 m was used to derive N value

Logged by	DRN 23/10/89
Compiled by	ANO 23/10/89
Checked by	VIP 23/10/89

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Figure 22 — Example 2 of typical borehole log

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Start date	17 October 1989	Casing diameter	200 mm to 8.00 m 150 mm to 12.00 m 100 mm to 14.00 m
End date	21 October 1989		
Drilling method	Cable percussion to 12.00 m Rotary coring 21.50 m	Borehole diameter	200 mm to 8.50 m 150 mm to 12.00 m 100 mm to 21.50 m
Equipment	T6H core barrel, water flush		

BOREHOLE No.	
National grid Coordinates	5423.00 E 4256.00 N
Orientation	Vertical
Ground Level	33.68 m OD

Date and time	Casing depth (m)	Depth to water (m) / Flush return (%)	Sample/core recovery: Depth (m) AL/ from to	Type / TCR	No. / SCR	RQD	SPT blows /N / Core size (mm)	Fracture spacing (minimum average measurement)	Description of strata	Depth (thickness) (m)	Level m OD	Legend
			10.00 - 10.50 10.00 - 11.00	D B	15 16		kV	(k = 1.0 x 10.6)		10.00	23.68	
									SAND (as sheet 1)	10.20 (0.55)	23.48	
	11.00	1.35	11.00 11.00 - 11.40 11.00 - 11.50	D B	17 18		C 103	25 mm (k = 5.5 x 10.6)	Probably dense, slightly sandy angular to rounded GRAVEL and COBBLES of quartz and limestone. (ALLUVIAL DEPOSITS)	11.30	22.38	
17.00 18/10	12.00	9.30	11.60 11.80 - 12.00	D D	19 20		kV	10 mm	Very weak to moderately weak very thinly bedded grey fine and medium grained LIMESTONE. Fracture surfaces stained orange brown (CARBONIFEROUS LIMESTONE)	(0.70)		
20/10 08.00	12.00	2.50 (100)	12.00 - 12.50	30	0	0	76	NI	Recovered as gravel size fragments from 11.30 m to 12.20 m	12.70	20.98	
		(100)	12.50 14.00	95	30	6		NI 50 175	Moderately weak thinly laminated black carbonaceous MUDSTONE. Fracture closely spaced 45° dip, smooth lightly orange stained. (CARBONIFEROUS LIMESTONE)			
			14.00 - 14.45	D	21		S 46		Sub-horizontal very closely spaced polished striated surfaces from 13.30 m to 14.20 m	(1.15)		
		(100)	14.45 - 15.50	100	50	25				15.00	18.68	
		(0)							Strong thinly to medium bedded dark grey medium grained LIMESTONE. Fractures medium spaced, dip 45° and 60° rough, stained. Fractures dip 90° up to 0.5 m long, stepped rough, tight, clean (CARBONIFEROUS LIMESTONE)			
			15.80 - 16.10	CS	1		C 50	20 mm 50 175 200				
		(0)	15.50 - 18.00	90	80	80			Sub-horizontal very closely spaced polished striated surfaces from 13.30 m to 14.20 m			
			17.31 - 17.49	CS	2							
18.00 20/10	14.00	10.00	18.00 - 19.50				kP	80 250 350 (L = 50)	65° fracture with 50 mm day infill at 17.50 m	(2.00)		
21/10 08.00	14.00	11.00 (0)	18.00 - 20.00	100	85	80				19.00	14.68	
		0.40 (75)	19.20 - 19.73	CS	3				LIMESTONE (As sheet 3)	(0.50)		
			20.00 - 21.50				kP	(L = 10)		20.00	13.68	

Remarks
6. In situ borehole vane test carried out at 6.00 m
7. In situ variable head permeability tests (kV) were carried out from 10.00 m to 10.50 m and 11.00 m to 11.50 m depth
8. In situ 'Packer' water injection tests (kP) were carried out from 18.00 tp 19.50 m and 20.00 m to 21.50 m depth
9. Geophysical borehole logging was carried out by ANO on completion

Logged by	DRN 21/10/89
Compiled by	ANO 25/10/89
Checked by	VIP 26/10/89

Project	CATCAIRN BUSHES, HAMPSHIRE Notable Developments Limited	Contract No. 5903 Sheet No. Sheet 2 of 3

Figure 22 — Example 2 of typical borehole log *(continued)*

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Start date	17 October 1989	Casing diameter	200 mm to 8.00 m 150 mm to 12.00 m 100 mm to 14.00 m	BOREHOLE No.	
End date	21 October 1989			National grid Coordinates	5423.00 E 4256.00 N
Drilling method	Cable percussion to 12.00 m Rotary coring to 21.50 m	Borehole diameter	200 mm to 8.50 m 150 mm to 12.00 m 100 mm to 21.50 m	Orientation	Vertical
Equipment	T6H core barrel, water flush			Ground Level	33.68 m OD

Date and time	Casing depth (m)	Depth to water (m) / Flush return (%)	Sample/core recovery: Depth (m) AL/ from to	Type / TCR	No. / SCR	RQD	SPT blows /N / Core size (mm)	Fracture spacing (minimum average measurement)	Description of strata	Depth (thickness) (m)	Level m OD	Legend
17.00 18/10	12.00	(100) 9.30	10.00 - 11.00	100	100	100		150 300 750	Strong to very strong, medium bedded grey fine grained LIMESTONE. Fractures medium spaced, sub-horizontal, rough, very tight, clean. (CARBONIFEROUS LIMESTONE)	20.00 (0.75) 21.50	13.68 12.18	
									END OF BOREHOLE AT 21.50 m			

Remarks	10. The borehole was advanced by chiseling from 11.50 m to 11.30 m (1.5 h) and from 11.30 m to 12.00 m (1.8 h) 11. On completion a 19.mm stand pipe was installed with the tip at 10.60 m and the following detail. From base of borehole cement grout up to 12.00 m: bentonite seal up to 11.00 m: sand filter up to 9.00 m: bentonite seal up to 8.70 m. Backfill with arisings up to 1.00 m: bentonite seal up to 0.50 m and completed with concrete and stopcock cover
Logged by	DRN 21/10/89
Compiled by	ANO 25/10/89
Checked by	VIP 26/10/89
Project	CATCAIRN BUSHES, HAMPSHIRE Notable Developments Limited
Contract No.	5903
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Figure 22 — Example 2 of typical borehole log *(continued)*

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Table 20 — Typical example of explanation of symbols and abbreviations to accompany borehole log (see Figures 21 and 22)

Samples			
U	Undisturbed driven tube sample, 100 mm nominal diameter unless noted		
P	Undisturbed pushed piston sample, 100 mm nominal diameter unless noted		
TW	Thin wall tube (pushed)		
CBR	CBR mould sample		
BLK	Block sample		
D	Small disturbed sample		
B	Disturbed bulk sample		
WS	Water sample		
CS	Core sample (from rotary core) taken for laboratory testing		
Test results (detailed test results normally presented elsewhere in report)			
S	Standard penetration test, split spoon sampler		
C	Standard penetration test, solid cone		
K	Field permeability test, type of test to be indicated e.g. kFH indicates falling head, kPI indicates packer injection.		
V, PP	Field vane test, vane shear strength quoted for natural (n) and remoulded (r) tests in kN/m^2, e.g. IVp for peak in-situ vane, HVr for residual hand vane, PP for pocket penetrometer		
I_a or I_d	Point load strength quoted for axial (a) and diametral (d) tests in MN/m^2, corrected to 50 mm reference diameter		
CS	Core sample for laboratory testing		
Drilling records			
W or F	Flush returns, estimated percentage returns together with colour where relevant		
TCR	Total core recovery, %		
SCR	Solid core recovery, %		
RQD	Rock quality designation, %		
lf	Fracture spacing, mm. The term non-intact (NI) is used where the core can be fragmented. Additional detail can be often given by quoting minimum, average and maximum fracture spacings		
Strata/sample description details (general)			
(Fg)	(Fine gravel size)	sp	spaced
(Mg)	(Medium gravel size)	cl	closely
(Cg)	(Coarse gravel size)	occ	occasional
(Co)	(Cobble size)	v	very
Vert.	Vertical	sl	slightly
Subv.	Subvertical	lt	light
Horz.	Horizontal	dk	dark
Subh.	Subhorizontal	pkt	pocket
deg.	Degrees	wk	weak

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Table 20 — Typical example of explanation of symbols and abbreviations to accompany borehole log (see Figures 21 and 22) (*continued*)

Discontinuities							
Type		**Surface appearance**		**Geometry**		**Staining**	
Fracture	FR	Very rough	VR	Planar	PL	Heavy	HV
Fissure	F	Rough	RO	Concave	CC	Moderate	MO
Joint	J	Smooth	SM	Convex	CX	Slight	SL
Bedding joint	BJ	Striated	SS	Curved	CU	Very slight	VS
Bedding fissure	BF	Pitted	PT	Undulating	UN		
Cross joint	CJ	Slightly polished	SLP	Conchoidal	CD		
Fault	FA	Polished	P	Irregular	IRR		
Bedding plane	BP	Highly polished	HP	Stepped	ST		

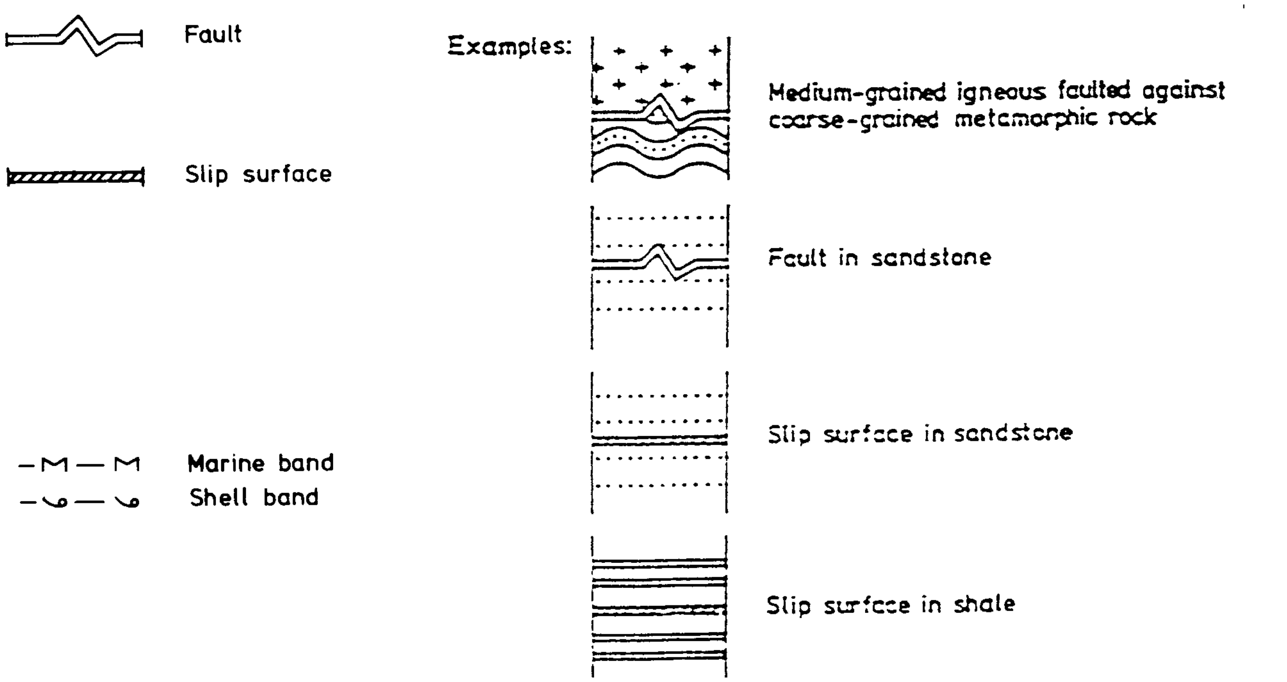
NOTES

A fissure can be a small fracture occurring with a soil and may often be much smaller than a joint. It can be distinguished in a borehole as having a surface area less than the cross-sectional area of the core.

The geometry of the lower surface of the joint/fissure can be described when viewed down the core.

The dip of bedding and joints should be given with respect to horizontal, the borehole axis being assumed to be vertical unless noted otherwise.

Graphical representation of specialist discontinuities



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47.2.6.4 *Light cable percussion boring*

For light cable percussion boring, in addition to the legend referred to in **47.2.6.3**, the following information should be recorded in the log:

a) a description of each stratum together with its thickness;

b) the depth and level of each change of stratum;

c) the depth of the top and bottom of each tube sample, or bulk sample and its type (see clause **22**) and the depth of each small disturbed sample;

d) the depth at the top and bottom of each borehole test and the nature of the test;

e) where standard penetration tests are being recorded, tests made with the thick-walled sampler should be distinguished from those made with a solid 60° cone; this information should be in accordance with BS 1377:1990 and should include all incremental blow counts and penetrations;

f) the date when each section was bored;

g) details of tools in use, including sizes;

h) water levels (including changes) and related casing depths at all samples, tests and water inflows;

i) a record of each water strike, including rate of rise of water level, depth of water in the borehole at start and finish of shift, depth of water at the time of each test or sample and depth of casing when each observation was made;

j) a record of any water added to facilitate boring;

k) where observation wells or piezometers have been installed, their depths should be given, together with details of the installation, preferably in the form of a diagram, and often on a separate report sheet;

l) water levels in observation wells measured subsequent to the completion of the borehole; these may be recorded separately.

47.2.6.5 *Rotary drilling*

For rotary drilling, in addition to the legend referred to in **47.2.6.3**, the following information should be recorded in the log:

a) a description of each stratum together with its thickness;

b) the depth and level of each change in stratum;

c) the depth of the start and finish of each core run;

d) the core recovery for each run, usually expressed as the percentage total core recovery (TCR);

e) the fracture state, expressed in terms of rock quality designation (RQD), solid core recovery (SCR) and fracture index (see **44.3.3**);

f) the date when each section of the core was drilled;

g) details of tools in use (open hole or coring bit sizes and types, core diameter, etc.);

h) an indication of the type of drilling flush and return proportion for each core run, with note of any change in colour;

i) a record of the depth of water in the hole at the start and finish of the shift and the depth of the casing, where used, at the time the observations were made;

j) a record of any observation wells or piezometers which were installed, with a note of their depths and details of the type, preferably in the form of a diagram and often on a separate report sheet;

k) water levels in observation wells measured after the completion of the drillhole (these may be recorded separately);

l) a record of tests carried out, such as permeability and packer tests;

m) the orientation of the drillholes;

n) location of core samples;

o) zones of core loss (where possible);

p) voids.

Rock cores should be photographed when fresh and before any destructive logging is carried out. The photographs should be in colour, to a consistent format on any investigation, include job, borehole and depth references, together with a scale and standard colour chart, and be sensibly free from distortion. The photographs should be presented in the report. Core should preferably take up over half the area of a photograph.

47.2.6.6 *Trial pitting*

For trial pitting the following information should be recorded in the log.

a) A description of each stratum together with its thickness. One advantage of trial pitting over boring is the opportunity to examine in-situ the variability of the strata and strata boundaries. Where these are not variable, a simple diagrammatic borehole log type presentation of a) and b) is acceptable with a comment about the uniformity around the pit. Where the ground is variable a sketch of one or more faces to show the variability should be presented.

b) The depth of each change of stratum.

c) The depth and position of each sample, including the lateral and vertical extent of larger samples such as bulks or blocks.

d) The depth and position of each test and the nature of the test.

e) The dates of excavation and logging.

f) Details of equipment in use, including excavator type, bucket size, shoring and pumps.

g) A record of groundwater conditions, including levels and estimated quantities.

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h) A record of the ease of excavation of the strata.

i) A record of the stability of the sides of the excavation.

j) Comment on the weather conditions.

k) A record of depth range examined in-situ or logged on arisings.

l) Details of any instrument installed and method of backfilling.

m) A sketch plan showing dimensions and orientation of pit, face reference numbers and location of landmarks.

Trial pit photographic records should include one or more faces and the spoil heap; all photographs should include a suitable and legible reference board. Artificial or flash lighting is normally required.

47.2.6.7 *In-situ tests*

For in-situ tests, the information to be recorded should be as outlined in section 3; more detailed requirements for many tests are given in BS 1377 or the Contract Specification may have special or additional requirements.

47.2.7 ***Incidence and behaviour of groundwater***

To obtain a clear understanding of the incidence and behaviour of groundwater, it is essential that all data collected on the groundwater should be included and that, where no groundwater was encountered, this too should be recorded. In addition, where groundwater observations could not be observed this should be noted; for instance, drilling with water flush or overwater, or boring at a rate much faster than water can make its way into the borehole. Where the information derived from boreholes is concise, it should be included in the logs. When this is not possible, the data should be given elsewhere in the report, and the borehole logs cross-referenced. The position of the borehole casing and the borehole depth at the time of an observation should be stated. All other data, including those from separate observation wells, should be given in a separate table. Where water has been added to or removed from the ground by the boring or drilling process, this should be recorded.

47.2.8 ***Location of boreholes***

The report should contain a plan showing the precise position of each borehole so that it is possible to locate each position accurately even after demolitions and excavations have taken place. Extensive tracts of open featureless country present problems that are best solved by linking the position of the borehole to a land survey. Ground levels related to a permanent datum are also required.

47.2.9 ***Results of laboratory tests and visual description of samples***

Where the test is covered by British Standards, the reporting of the results should be in accordance with those standards; where not so covered, all sensibly relevant data should be given. Where an extensive programme of testing has been undertaken, a summary should be provided in addition to the detailed results. Test results may also be presented in line with specific contract requirements. The precise test carried out should also be stated without ambiguity. Where the test is reasonably standard, for instance "consolidated, drained, triaxial, compression test on 100 mm diameter samples", the name alone suffices; but where the test is not standard, a full description should be given.

The precise method of recording the visual descriptions (see clause **38**) depends on the particular circumstances. It may be convenient to show these on the same sheets as the results of the laboratory tests, or in a separate table. At times, the results of the laboratory tests, particularly the identification tests, indicate a soil different from that visually described.

The description should not be discarded on that account but should be preserved as a record of the observer's opinion. The laboratory report forms and data sheets should be filed for possible future reference or separately bound and presented (see **47.2.1**).

47.2.10 ***Special reports***

Where a specialized study has been undertaken, e.g. detailed mineralogical analysis, the report should be included as an annex.

47.3 Summary of ground conditions and parameters

47.3.1 ***Introduction***

After the descriptive report, it is necessary to collate the results into a description of the stratigraphy, an assessment of the ground parameters relating to that stratigraphy and a description of the groundwater conditions. The level of interpretation input to this phase varies widely, depending on the contractual responsibilities of the various parties, so there can be no firm rules as to whether this phase is part of the descriptive report (see **47.2**) or the interpretation (see **47.4**). The collector of the data and writer of the descriptive report can provide significant assistance to the designers and this detailed knowledge of the site should be utilized, so that the text of the report provides an account of the ground conditions with the required degree of interpretation at the agreed place.

It is important that the presentation on the ground conditions be thorough and clear, as this text provides the key point of information to the many different specialists who could be using the report on an individual site (e.g. grouting, groundwater control, piling or tunnelling). The subsequent assessment in the next section of the report can alter with change of end use, but this summary should not.

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47.3.2 ***Ground types***

Where appropriate, the ground should be divided into a series of soil and rock types for which the engineering properties may be regarded as sensibly constant for the purpose in hand. This division is usually, though not always, closely related to the geological succession. A description of each ground type should be given and any anomalies that have been observed should be noted and commented on. In this context, it should be constantly borne in mind that all samples are, to a greater or lesser degree, disturbed and may not be truly representative of in-situ conditions. Similar caution is required in considering the results of in-situ tests.

47.3.3 ***Stratigraphy***

An account should be given of the sequence of ground types as they occur in the various parts of the site. Wherever possible the stratigraphy of the site should be tied into its topographical, geological and geomorphological features. Attention should be specifically drawn to any anomalies that may have a significant effect on the works being considered.

47.3.4 ***Borehole sections***

For the purpose of analysis, it is often necessary to make basic assumptions about the ground profile at the site. These are best conveyed in a report by a series of cross-sections illustrating the ground profile, simplified as required, with groundwater levels shown. The presentation of a borehole section in a descriptive report would usually not include joining up the boreholes by stratum boundaries; the same section in the interpretative report would normally require joining up, using all the available information and suitably qualified in any areas of doubt. Accurate and integrated interpretation of geological maps, boreholes and other data is a prerequisite to a thorough understanding of the ground. It is recommended that borehole sections in a ground conditions report should be interpreted, but this requirement should be clarified at an early stage. Borehole sections should preferably be plotted to a natural scale and, if it is necessary to exaggerate the vertical scale, this should be clearly indicated. Where the ground information is either very variable or too sparse to enable cross-sections to be prepared, individual borehole logs plotted diagrammatically are an acceptable alternative. Where it is particularly important to prepare cross-sections, sparse and variable information can sometimes be supplemented between boreholes by means of information from soundings and geophysical investigations. It can be helpful to indicate relevant soil parameters on sections; e.g., results of standard penetration tests, triaxial tests or earthworks relationship tests.

47.3.5 ***Ground parameters***

There is no universally accepted method of selecting these parameters, but the following approach may help to arrive at reliable values:

a) compare both laboratory and in-situ test results with ground descriptions;

b) cross-check, where possible, laboratory and in-situ results in the same ground;

c) collect individually acceptable results for each formation and decide representative values appropriate to the number of results;

d) where possible, compare the representative values with experience and published data for similar geological formations;

e) consider and explain apparently anomalous or extreme results.

47.3.6 ***Groundwater***

Groundwater is a very important factor in the design of structures and also in the selection of methods of construction. Inadequate or erroneous assessment of groundwater conditions is one of the biggest single contributors to problems on-site during construction. The report should describe regional groundwater conditions and the presence or otherwise of perched, artesian or downward draining conditions. Comment should be made on any anomalies and the possibility of the rise or fall of groundwater with the season, tide or other long term variation.

47.3.7 ***Chemical conditions***

Comment should be made on chemical conditions in the ground and groundwater, not only with regard to attack on buried parts of the structure, but also with regard to possible effects in construction and service life, whether these be due to natural causes or to man's activities. Any conditions that could affect health and safety during construction or in subsequent use should be mentioned. See annex F.

47.4 Engineering interpretation

47.4.1 ***Matters to be covered***

Methods of analysing ground data and applying them to the solution of engineering problems are not covered in this code. Some guidance on this may be found in other codes, such as BS 5493, BS 6031, BS 7361, BS 8002, BS 8004, BS 8006 and CP 2012-1. Some guidance on the format of the report, and on the most common topics for which advice and recommendations are required and on what should be included, are given in **47.4.3** to **47.4.6**. The topics are listed briefly under the general headings of design, construction expedients, sources of materials, and failure. It is likely that in many cases the client commissioning the investigation will indicate those aspects of the project on which advice and recommendations are required, and the topics given are intended as a guide where this has not been done.

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47.4.2 ***Data on which interpretation is based***

A clear statement should always be made about the data on which the analysis and recommendations are based. The information comes under two separate headings, as follows.

a) The information related to the project and usually supplied by the designer. For example, for buildings and other structures this should include full details on the loading, split into dead and live; column spacing, where appropriate; and depth and extent of basements and details of neighbouring structures. For earthworks, heights of embankments and the materials of which they are to be made, together with the depths of cuttings, are all relevant to the interpretation.

b) Ground parameters, selected from the summary of ground conditions report (**47.3**) by the engineer making the analysis and preparing the recommendations.

47.4.3 ***Design***

The following list, which is by no means exhaustive, indicates the topics on which advice and recommendations are often required, and also what should be included. As noted above, the level of comment required here varies. Given the availability of a wide and ever-changing range of proprietary systems, all of which interact with the ground in subtly different ways, it is important that the report writer does not overstate his level of knowledge. Apart from general comments, the detailed recommendations are the province of the specialist suppliers of a service. It is essential for the report to provide all parties with the information needed to assess the suitability of various options and the design of the works.

a) Spread foundations: level, either in terms of a depth or to a stated stratum; safe or allowable bearing capacity; estimated total and differential settlements; possible alternative types of foundation; possible ground treatment.

b) Piles: types suited to the ground profile and environment; estimated safe working loads, or data from which they can be assessed; estimated settlements of structures.

c) Retaining walls: lateral pressures or data from which they can be derived; wall friction; bearing capacity; groundwater conditions.

d) Basements: comment on the possibility of flotation. An estimate of the rise of the basement floor during construction, where appropriate; groundwater levels.

e) Ground anchorages: bearing stratum and estimated safe loads, or data from which they may be calculated;

f) Chemical attack: most commonly takes the form of recommendations for protecting buried concrete against attack from sulfate-bearing soils and groundwater. The results are usually evaluated by reference to BRE Digest No 363 [35]. Also to be considered is the possibility of corrosion of steel in saline waters or in the presence of sulfate-reducing bacteria. The effect of acidic or highly alkaline soils may also need to be considered. Contaminated soils, especially those containing high concentrations of organic chemicals, should be considered for their effects on all building materials, including effects on services. These factors should also be considered with regard to health and safety during construction and in subsequent use of the structure. See annex F.

g) Pavement design: assessment of appropriate design parameters or California Bearing Ratios; type and thickness of pavement; possibility of using soil stabilization for forming pavement bases or sub-bases; recommendations, where appropriate, for sub-grade drainage; comment on susceptibility of soil at formation level to frost heave.

h) Slope stability: recommendations on temporary and permanent slopes for excavations, including where appropriate, drainage measures. Comment should be made, where relevant, on the possibility of weathering of rock faces and the available methods of dealing with this hazard. Recommendations for the monitoring of unstable slopes may also be required.

i) Mining subsidence: description of the workings; voids and stability; possible recommendations for methods of filling known cavities near the surface; the design of structures to withstand movements without damage or measures to limit the damage and simplify repairs.

j) Tunnels and underground works: a description of the ground through which the tunnel is to be driven, by chainage; possible covering of the following points: methods and sequence of excavation; whether excavation is likely to be stable without support; suggested methods of lining in unstable excavations; potential use of rock bolting; likelihood of encountering groundwater and recommendations for dealing with it; special features for pressure tunnels; risk of encountering ground or water contamination; possibility of natural or man-made gases.

k) Safety of neighbouring structures: an assessment of the likely amount of movement caused by adjacent excavations and groundwater lowering, compressed air working, grouting and ground freezing or other geotechnical processes. The possibility of movement due to increased loading on adjacent ground may also need to be considered.

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l) Monitoring of movements: comment on the necessity for measuring the amount of movement taking place in structure and slopes, together with recommendations for the method to be used (see **18.4**); recommendations for taking photographs before the commencement of works (see **6.1.2**).

m) Embankments: comment on stability of embankment foundations; assessment of amount and rate of settlement and the possibility of hastening it by such means as vertical drains; recommendations for side slopes; choice of constructional materials and methods; parameters for control of earthworks.

n) Drainage: comment on possible drainage methods during construction for works above and below ground; general permanent land drainage schemes for extensive areas.

47.4.4 ***Construction expedients***

Comments and recommendations are often required on the points listed below. Safety aspects should be included where appropriate. These matters are often given insufficient attention, although they are comparable in importance to the design of the permanent works.

a) Open excavations: method and sequence of excavation; what support is needed; how to avoid "boiling" and bottom heave; estimated upward movement of floor of excavation. Comment on relative merits of sheet piling and diaphragm or bored pile walls where appropriate.

b) Underground excavations: method and sequence of excavation and the need for temporary roof and side support; dealing with gases.

c) Groundwater: likely flow, head and quantity and how to deal with it.

d) Driven piles, bored piles and ground anchors: methods of driving or construction suited to the ground profile, environment and neighbouring buildings.

e) Grouting: types of grouts likely to be successful in the ground and recommended method of injection.

f) Mechanical improvement of soil below ground level. Comment on the suitability of techniques for the consolidation of loose soils.

g) Contamination: known or suspected contaminants and gases in soil, groundwater and any cavities. Comment on health and safety aspects both during and after construction. see annex F.

47.4.5 ***Sources of materials***

The following are suggested.

a) Fill: possibility of using excavated material for this purpose with an assessment of the proportions of usable material; methods and standards of compaction; possible off-site sources of fill; bulking factor.

b) Aggregates: in areas where no commercial sources are available, the possibilities of winning and processing materials available locally.

47.4.6 ***Failures***

Where site investigation has been undertaken in an attempt to identify the cause of failures the following points may be relevant.

a) Foundations: the nature and dimensions of the foundations, identification of the cause of failure and, where appropriate, an estimate of the amount of settlement that has already occurred, together with an assessment of how much more is likely to occur and its probable effect on the structure; cause of excessive vibrations of machine foundations; recommendations for remedial measures.

b) Landslides: classification of the type of movement and location of the failure surfaces. Recommendations for immediate stabilizing expedients and long term measures.

c) Embankments: identification of whether the seat of failure lies within the embankment itself or the underlying strata, the probable cause and suggested method of repair and strengthening.

d) Retaining walls: comment on cause of failure or excessive deflection; forecast of future behaviour of wall and recommendations where appropriate for strengthening it.

e) Pavements: determination of whether the failure is within the pavement itself or the sub-grade and recommendations for repairs or strengthening or both.

47.4.7 ***Calculations***

Where calculations have been made, they should be included as an annex, or a clear indication of the methods used should be given.

47.4.8 ***References***

All published works referred to in the report should be listed.

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Annex A (informative)

General information required for desk study

A.1 General landsurvey

a) location of site on published maps and charts (see annex B);

b) aerial photographs, all dated where appropriate;

c) site boundaries, outlines of structures and building lines;

d) ground contours and natural drainage features;

e) obstructions to sight lines and aircraft movement, for example transmission lines;

f) indication of obstructions below ground;

g) record of differences and omissions in relation to published maps (see annex B);

h) position of survey stations and benchmarks (the latter with reduced levels);

i) meteorological information (see **B.4**).

A.2 Permitted use and restrictions

a) planning and statutory restrictions applying to the particular areas under the Town and Country Planning Acts administered by appropriate planning authorities;

b) local authority regulations on planning restrictions, listed buildings and building bye-laws;

c) Board of Trade regulations governing issue of industrial development certificates;

d) right of light, support and way, including any easements;

e) tunnels; mine workings, abandoned, active and proposed, mineral rights;

f) ancient monuments; burial grounds, etc;

g) previous potentially contaminative uses of the site and adjacent areas (see **A.12** and annex F).

h) any restrictions imposed by environmental and ecological considerations, e.g. sites of special scientific interest.

A.3 Approaches and access (including temporary access for construction purposes)

a) road (check ownership);

b) railway (check for closure);

c) by water;

d) by air.

A.4 Ground conditions

a) geological maps (see **B.2.1**);

b) geological memoirs (see **B.2.1.3**);

c) flooding, erosion, landslide and subsidence history;

d) data held by central and local authorities;

e) construction and investigation records of adjacent sites;

f) seismicity (see **B.8**).

A.5 Sources of material for construction (see also D.7)

a) natural materials;

b) tips and waste materials;

c) imported materials.

A.6 Drainage and sewage

a) names of sewage, land drainage and other authorities concerned and bye-laws;

b) location and levels of existing systems (including fields, drains and ditches), showing sizes of pipes, and whether foul, storm water or combined;

c) existing flow quantities and capacity for additional flow;

d) liability to surcharging;

e) charges for drainage facilities;

f) neighbouring streams capable of taking sewage or trade effluents provided they are purified to the required standard;

g) disposal of solid waste;

h) flood risk:

1) to proposed works;

2) to proposed works;

A.7 Water supply

a) names of authorities concerned and bye-laws;

b) location, sizes and depths of mains;

c) pressure characteristics of mains;

d) water analysis;

e) availability of water for additional requirements;

f) storage requirements;

g) water source for fire-fighting;

h) charges for connections and water;

i) possible additional sources of water;

j) water rights and responsibilities under the Water Resources Act 1991.

A.8 Electricity supply

a) names of supply authorities concerned and regulations;

b) location, sizes and depth of mains;

c) the voltage, phases and frequency;

d) capacity to supply additional requirements;

e) transformer requirements;

f) charges for installation and current.

A.9 Gas supply

a) names of supply authorities concerned and regulations;

b) location, sizes and depths of mains;

c) type of gas, thermal quality and pressure;

d) capacity to supply additional requirements;

e) charges for installation and gas.

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A.10 Telecommunications

a) addresses of local offices;

b) location of existing lines;

c) BT and other agency requirements;

d) charges for installation.

A.11 Heating

a) availability of fuel supplies;

b) planning restrictions (smokeless zone; Clean Air Act 1956 [204] administered by local authorities);

c) district heating.

A.12 Information related to potential contamination

a) history of the site, including details of owners, occupiers and users any incidents or accidents relating to dispersal of contaminants;

b) processes used, including their locations;

c) nature and volume of raw materials, products, waste residues;

d) waste disposal activities and methods of handling waste;

e) layout of the site above and below ground at each stage of development, including roadways, storage areas, hard-cover areas, and the presence of any extant structures and services;

f) presence of any waste disposal tips, abandoned pits and quarries;

g) presence of nearby sources of contamination from which contaminants could migrate via air and/or groundwater onto site;

Annex B (informative)

Sources of information

B.1 Ordnance Survey

Ordnance Survey produces a wide range of conventional and data map products, which are continually being revised and upgraded to meet customer requirements. The information here is just a sample of the products and services available. For up-to-date information please contact Ordnance Survey Customer Relations at the address given at the end of **B.1.2.2**.

B.1.1 *Graphic map products*

B.1.1.1 *Introduction*

Three basic source survey scales make up the Large Scales Mapping covering Great Britain:

Urban Mapping	1:1 250 Scale Source Survey	1 cm to 12.5 m (50 inches to 1 mile)
Rural Mapping	1:2 500 Scale Source Survey	1 cm to 25 m (25 inches to 1 mile)
Mountain and Moorland Mapping	1:10 000 Scale Source Survey	1 cm to 100 m (6 inches to 1 mile)

The master copies of the above maps have been converted to digital data and all of the above mapping is also available in this form (i.e. computer readable).

B.1.1.2 *Traditional large scale graphic products*

Traditionally, the basic scales have all been available as:

Published Maps	Maps which have been litho-printed on chart paper.

Additional updated versions of 1:1 250 and 1:2 500 scale maps are available as follows:

SIM (Survey Information on Microfilm)	These are printouts at scale from microfilm copies of updated master survey documents.
SUSI (Supply of Unpublished Survey Information)	Diazo or dyeline copies made to order from the master survey document.

B.1.1.3 *Superplan — graphic plots from up-to-date digital map data*

The Superplan service offers the latest large scale map information together with the advantage of choice and flexibility including:

— Site-centred or National Grid format.

— Enlargement or reduction from source survey scale.

— Enlargement or suppression of map detail.

— Rotation of plot (e.g. to follow a road or include a site).

Large scale mapping through the Superplan service is available for the whole of Great Britain.

B.1.1.4 *Superplan — instant printout*

Available in all Superplan areas, an Instant Printout is an upgraded and improved microfilm-based printout on National Grid sheetlines. These maps are updated after approximately 20 units of map change have been recorded.

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B.1.1.5 *Small scale graphic products*

Ordnance Survey publishes small scale map series at the following scales:

Pathfinder, Outdoor Leisure and Explorer Maps	1:25 000 scale
Landranger Maps	1:50 000 scale
Travelmaster Maps	1:250 000 scale or 1:625 000 scale

B.1.2 *Digital map data products*

Ordnance Survey produces a wide range of map data products including the following.

B.1.2.1 *Land-line*

Land-line is the family name for a range of digital map data products

It is produced from the Ordnance Survey large scale topographic database, at one of the three basic source survey scales outlined in **B.1.1.1**.

It has a vector (point and line) data structure representing map detail as single coordinated points, or as strings (lines) of multiple points. These points and strings are data coded for feature identification. The Land-Line 93 product contains unique junction coordinates and is edge matched between adjacent map sheets.

It is available in NTF and DXF transfer formats, on a variety of media from CD-ROM to floppy disk and with a choice of supply options.

B.1.2.2 *Other digital map data products*

For those users who have special data requirements a Data Solutions service, including field data capture, is available.

Further information may be obtained from Ordnance Survey Customer Information[1)].

Land-Form PANORAMA	1:50 000 scale digital height data
Land-Form PROFILE	1:10 000 scale digital height data
ADDRESS-POINT	Digitally coordinated postal address gazetteer
OSCAR	Ordnance Survey Centre Alignment of Roads datasets
Boundary-Line	Administrative area polygon data
Urban-Areas.91	Urban area polygons data linking to 1981 and 1991 Censuses
1:10 000 Scale Black and White Raster	Raster data from 1:10 000 scale published mapping
1:50 000 Scale Gazetteer	Definitive names data from Landranger published mapping
Strategi	1:250 000 scale vector digital map data
Base Data GB	1:625 000 scale vector digital map data

B.2 Geological survey and soil surveys maps and memoirs

B.2.1 *Geological maps and memoirs*

B.2.1.1 *General*

The British Geological Survey is the national repository for geoscience data in the UK. It is the custodian of an extensive collection of maps, records, materials and data relating to the geology of the UK, its continental shelf and many countries overseas. Digital indexes to the collections have been established and selective geographical searches of data availability are carried out using a GIS based information retrieval system. The Survey operates Central Enquiry Desks in the Keyworth and Edinburgh offices, and an Information Office is also maintained at the Natural History Museum in London. Enquiries dealing with particular localities should preferably be submitted in writing, accompanied by a marked up copy of a map or by eight figure national grid references defining the area in question. Local libraries may also provide a source of geological information for the locality. The Geological Society at Burlington House, London has an extensive library for the use of its members and bona fide researchers.

B.2.1.2 *Maps*

The 1:10 000 and 1:10 560 (6" to 1 mile) geological maps comprise the Survey's standard large scale series for recording field survey information. Formal publication of these maps was discontinued many years ago and for most map sheets copies were made to order from fair drawn dyeline masters. With the introduction of digital cartographic techniques to the production of these maps, dyeline copying is being progressively withdrawn as high quality colour electrostatic plots become available. Regional reference sets of these maps are available for consultation at the Survey's offices.

[1)] Ordnance Survey Customer Information, Romsey Road, SOUTHAMPTON SO16 4GU. Tel: 01703 792912. Fax: 01703 792452.

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The main publication scale of geological maps is 1:50 000, which is replacing the earlier 1:63 360 (1" to 1 mile) scale. These 1:50 000 maps are now being produced digitally to an accelerated timescale and are available as high quality colour electrostatic plots produced on demand. The litho printed edition of the map is normally published within two years of release of the digital version. For certain areas of special geological interest warranting greater detail, published maps have been issued at an intermediate 1:25 000 scale.

A series of 1:250 000 scale maps covering the United Kingdom and continental shelf has been published. This series portrays solid geology on and offshore and Quaternary geology and seabed sediments offshore.

A range of applied geology maps covering new town and development areas, and engineering geology maps of special local or regional studies are available. These maps normally accompany open-file reports but may also be purchased separately. The Survey has also published hydrogeological maps of the whole of the UK and more detailed maps of major aquifer units, and geophysical maps of gravity and aeromagnetic anomalies covering the British Isles and continental shelf.

B.2.1.3 *Memoirs*

The 1:50 000 and 1:63 360 geological maps are accompanied by a series of explanatory sheet memoirs. They include coalfield and economic memoirs for selected areas of the country. The memoir series has been published over many years and the geological interpretations in the older editions may have been changed. Copies of out of print memoirs may be obtained from the Survey's Library at Keyworth, Nottingham.

B.2.1.4 *Regional guides and reports*

A series of 20 handbooks describing the geology of individual regions of the United Kingdom are published as British Regional Geology Guides. A new series of Offshore Regional Reports describing the offshore geology of the United Kingdom has been published to complement the Regional Geology Guides.

These publications are produced by the Stationary Office and can also be obtained from Government bookshops.

B.2.1.5 *Digital data*

Increasingly data are being made available in digital format for incorporation into user GISs or for further manipulation and analysis. The supply of digital data is subject to licensing and royalty arrangements determined individually, according to the particular application for which the data are being sought.

B.2.1.6 *Library services*

The Survey's Library at Keyworth offers a range of literature search services, commercial desk-top study facilities and other related services. Library facilities are also available at the Edinburgh office.

B.2.1.7 *National Geosciences Records Centre*

This national resource provides the following data facilities.

B.2.1.8 *Geological records*

An extensive collection of geological records and plans is available for inspection at its offices in Keyworth and Edinburgh. It includes over 600,000 borehole records as well as geophysical logs, site investigation reports, road reports, mine and quarry plans and sections, field notebooks and unpublished survey reports. The Survey has statutory rights to copies of records from mineral exploration boreholes deeper than 30 m, water bores and test records of water flow.

B.2.1.9 *Borehole core and specimens*

The archive includes over 175 kilometres of core and 3 million samples and specimens from on and offshore sites. This material can be inspected and samples taken for further analysis and testing.

Charges are levied for inspection, retrieval and supply of material from the Records Centre. The Survey welcomes donations of geological information acquired at excavations, trial or other boreholes, or site investigations.

A catalogue of printed geological maps and publications is available from the Survey's offices where further details of the information and services provided by the Survey can also be obtained.

Kingsley Dunham Centre Keyworth Nottingham NG12 5GG Tel: 0115 936 3100 (switchboard) 0115 936 3143 (enquiries)	Murchison House West Mains Road Edinburgh EH9 3LA Tel: 0131 667 1000	London Information Office British Geological Survey Earth Galleries Natural History Museum Exhibition Road London SW7 2DE Tel: 0171 589 4090

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B.2.2 ***Soil maps and memoirs***

Soils maps depict the distribution of soil types in the landscape and incorporate much information that is potentially useful for site investigations.

Soil are defined on the basis of the upper 1.2 m of material at the Earth's surface and take into account particle-size distribution, chemical characteristics, drainage and parent material. Certain soil types are defined in other ways, for example, peats are defined by their botanical composition and state of humification and can vary in depth from 0.5 to over 10 m. The occurrence of compressible materials, shrinkable clays, shallow depth to rock, unconsolidated sands and degrees of natural soil wetness and drainage can all be assessed from soil maps. Special maps showing peat distribution, groundwater vulnerability to pollution and the risk of erosion by water are also available. Soils information for England and Wales is held by the Soil Survey and Land Research Centre at Silsoe; the Macauley Land Use Research Institute at Aberdeen holds the information for Scotland and the Department of Agriculture for Northern Ireland in Belfast holds data for Northern Ireland.

B.2.3 ***Hydrogeological maps***

Publication lists, soil maps and reports for England and Wales are obtainable from:

Soil Survey and Land Research Centre
Cranfield University
Silsoe, Bedford, MK45 4DT

Tel: 01525 860428

Fax: 01525 861147

Publications lists, soil maps and reports for Scotland are obtainable from:

Resource Consultancy Unit
Macaulay Land Use Research Institute
Craigiebuckler, Aberdeen, AB9 2QJ

Tel: 01224 218611

Fax: 01224 208065

Hydrogeological maps are published at the 1:63 360, 1:50 000 and 1:25 000 scales for large areas of Great Britain and there is complete cover for England, Wales and Scotland at the 1:625 000 scale, and at 1:250 000 scale for Northern Ireland. Groundwater vulnerability maps of Scotland at a scale of 1:625 000 and Northern Ireland at a Scale of 1:250 000 are published by the British Geological Survey. A series of maps covering England and Wales at 1:100 000 is being produced for the Environment Agency and is available from the Stationery Office. In addition there is much unpublished archive data and surveys which may be consulted by arrangement with the appropriate institutes.

B.3 Marine information

B.3.1 ***Charts***

The Hydrographic Office of the Ministry of Defence publishes charts for nearly all the navigable tidal waterways of the world to various scales. The charts show high and low water lines and the levels of the sea and river beds with reference to a datum that is defined on the chart, together with certain other tidal information.

B.3.2 Tide Tables

Admiralty Tide Tables are published annually in three volumes, namely:

Vol 1: European waters (including the Mediterranean Sea);

Vol 2: The Atlantic and Indian Oceans;

Vol 3: The Pacific Ocean and adjacent seas.

Each of these consists of three parts:

Part 1 gives predictions of the times and heights of high and low water at Standard Ports;

Part II gives data for prediction at Secondary Ports;

Part III gives the harmonic constants for all Standard and most Secondary Ports.

B.3.3 ***Other Publications***

The Hydrographic Office also publishes books of sailing directions, general information on tides, and other navigational publications, which, together with the charts, are listed in the Catalogue of Admiralty Charts and other Hydrographic Publications (which also includes a list of agents), published annually.

Further information on Admiralty charts and hydrographic publications can be obtained direct from the Hydrographer of the Navy[2)] .

B.4 Meteorological Information

B.4.1 ***Reports***

The Meteorological Office collects and publishes meteorological information in the United Kingdom in various forms:

a) **Monthly**: The *Monthly Weather Report* summarizes weather observations for about 600 stations in the United Kingdom. Principal data include air temperature, rainfall and sunshine. There are summaries of autographic records of wind from about 130 stations, and frequency tables of the occurrence of air temperatures between certain limits for about 20 stations. Additionally, an annual summary has frequency tables of sunshine, rainfall and wind speed. The *Monthly Weather Report* is normally published about 8 months in arrears to allow time for all the data to be collected quality controlled.

[2)] Hydrographic Office, Ministry of Defence, Taunton, Somerset. Tel: 01823 337900; Telex: 284077.

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b) **Annually**: Monthly and annual rainfall totals for about 6 000 stations, together with amounts and dates of maximum daily falls, are published in *Rainfall 19xx*. Monthly, annual and seasonal rainfall as a percentage of annual average, frequency of distributions of daily rainfall, amounts and spells of rainfall, and rainfall excess or deficiency are included for selected stations. Heavy falls of rain are also listed. *Rainfall 19xx* is normally published two to three years after the end of the year to which it relates.

c) **Other information**: Where up-to-date information is not available, local statutory bodies, such as water authorities, may be useful sources of data.

B.4.2 ***Statistics***

Averages and extremes for various elements have been published from time to time in the past, and averages for selected stations in the United Kingdom for the periods 1961 to 1990 are due to be published in 1994. *Tables of temperature, relative humidity and precipitation for the world*, first published in 1958 and all parts of which remain available [205], presents climatic tables for some 1 800 stations throughout the world. Data consist of means and extremes for varying periods depending on the station.

All enquiries concerning the availability and prices of historical weather data should be addressed to the Meteorological Office[3].

B.5 Hydrological information

B.5.1 Surface water run-off data is collected by water authorities, private water undertakings and occasionally by local authorities. "The Surface Water Year Book of Great Britain" [206] was published annually in water years (October to September) for selected gauging stations. From 1935 to 1937, it was published by the Stationery Office for the Ministry of Health and the Scottish Office; from 1937 to 1964, for the Ministry of Housing and Local Government and the Scottish Office; and from 1964 to 1970, for the Water Resources Board and the Scottish Development Department. The practice of publishing in water years was changed in 1966 into calendar years, and responsibility for publishing is now with the Water Data Unit Department of the Environment (DOE) since the replacement of the Water Resources Board in 1974.

B.5.2 Since 1985, both surface water run-off data and groundwater level data have been published jointly by the Institute of Hydrogeology and the British Geological Survey (Hydrogeology Group) annually in "Hydrological data UK" [207] with Yearbooks published for 1981 up to the present.

B.5.3 Evapotranspiration and soil moisture information is issued weekly by the Meteorological Office as part of the MORECS (Meteorological Office Rainfall and Evaporation Calculation System) service, which includes data on potential evapotranspiration over Great Britain. All enquiries concerning MORECS should be addressed to the Meteorological Office[4] .

B.6 Aerial photographs

There are many collections of aerial photographs for the United Kingdom extending back over several decades. These are not centrally archived and there is no complete index available. In order to establish availability for a particular area of the country, it may be necessary to contact several sources. In 1993, the National Association of Aerial Photographic Libraries (NAPLIB) published the first Directory of Aerial Photographic Collections in the United Kingdom. [208] It is expected that this will be updated periodically. The Directory alphabetically lists all organizations and agencies that hold collections of aerial photographs including aerial survey companies that can carry out new aerial photography on commission. Full contact addresses are given for each entry together with an indication of the extent of the holding. Copies of the Directory can be obtained from: Publications Department, Aslib, The Association for Information Management, Information House, 20–24 Old Street, London EC1V 9AP (Tel: 0171 253 4488; Fax: 0171 630 0514).

B.7 Other sources of information.

Other sources of information include:

a) the maps of the Second Land Utilization Survey of Britain;[5]

b) records of mines and mineral deposits (see **E.6**);

c) maps published by a number of individuals before the establishment of the Ordnance Survey. Copies of these may often be found in public libraries and local museums.

The Transport Research Laboratory have issued TRL Report 192 "Sources of information for site investigations in Britain" [209]. This report is an update of the previous TRL Report LR403, which is referenced in the current version of BS 5930. TRL 192 will be implemented as the main source reference in the Design Manual for Roads and Bridges for planning site investigations for Trunk Roads [210].

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[3] The Met Office, Room JG6, Johnson House, London Road, Bracknell, Berks. RG12 2SY.

[4] The Met Office, Room SG/5, Sutton House, London Road, Bracknell, Berks. RG12 2SY.

[5] The published maps, an index map, and the Land Use Survey Handbook, may be obtained from the Director, Kings College, Strand, London WC2; or from Edward Stanford Ltd., 12-14 Long Acre, London WC2

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B.8 Seismological information

Computer listings and maps of earthquakes occurring in the United Kingdom an elsewhere may be obtained from the Global Seismology Group of the British Geological Survey, Edinburgh.[6)] The information, which is accumulated from the World Network, and the seismographs and seismographic arrays in the United Kingdom, includes time of occurrence, epicentral distance, focal depth, and magnitude. The listing includes historical references from a wide range of sources from both the United Kingdom and elsewhere. On request, the data can be converted to a quantitative assessment of the seismic hazard at a site, including the probability of a particular ground acceleration being exceeded (per year).

Annex C (informative) Notes on site reconnaissance

C.1 Preparatory

a) Whenever possible, have the following available: site plan, district maps or charts, and geological maps and aerial photographs.

b) Ensure that permission to gain access has been obtained from both owner and occupier.

c) Where evidence is lacking at the site or some verification is needed on a particular matter, for example, flood levels or details of changes in site levels, reference should be made to sources of local information such as: Local Authority, engineer's and Surveyor's Offices, early records and local inhabitants (see annex B).

d) When undertaking site reconnaissance on potentially contaminated land, ensure that all likely hazards have been identified, that appropriate safety procedures are followed, and that necessary safety equipment is used.

C.2 General information

a) Traverse whole area, preferably on foot.

b) Set out proposed location of work on plans, where appropriate.

c) Observe and record differences and omissions on plans and maps; for example, boundaries, buildings, roads and transmission lines.

d) Inspect and record details of existing structures.

e) Observe and record obstructions; for example, transmission lines, ancient monuments, trees subject to preservation orders, gas and water pipes, electricity cables, sewers.

f) Check access, including the probable effects of construction traffic and heavy construction loads on existing roads, bridges and services.

g) Check and note water levels, direction and rate of flow in rivers, streams and canals, and also flood levels and tidal and other fluctuations, where relevant.

h) Observe and record adjacent property and the likelihood of its being affected by proposed works (see **6.1.2**), and any activities that may have led to contamination of the site under investigation.

i) Observe and record mine or quarry workings, old workings, old structures, and any other features that may be relevant.

j) Observe and record any obvious immediate hazards to public health and safety (including to trespassers) or the environment.

k) Observe and record any areas of discoloured soil, polluted water, distressed vegetation or significant odours.

l) Observe and record any evidence of gas production or underground combustion.

C.3 Ground information

a) Study and record surface features, on site and nearby, preferably in conjunction with geological maps and aerial photographs, and note the following.

1) Type and variability of surface conditions.

2) Comparison of surface lands and topography with previous map records to check for presence of fill, erosion, or cuttings.

3) Steps in surface, which may indicate geological faults or shatter zones. In mining areas, steps in the ground are probably the result of mining subsidence. Other evidence of mining subsidence should be looked for: compression and tensile damage in brickwork, buildings and roads; structures out of plumb; interference with drainage patterns.

4) Mounds and hummocks in more or less flat country which frequently indicate former glacial conditions; for example, till and glacial gravel.

5) Broken and terraced ground on hill slopes, which may be due to landslips; small steps and inclined tree trunks, can be evidence of creep.

6) Crater-like holes in chalk or limestone country, which usually indicate swallow holes filled with soft material.

7) Low-lying flat areas in hill country, which may be sites of former lakes and may indicate the presence of soft silty soils and peat.

b) Assess and record details of ground conditions in quarries, cuttings and escarpments, on site and nearby.

c) Assess and record, where relevant, ground water level or levels (often different from water course and lake levels), positions of wells and springs, and occurrence of artesian flow.

d) Study and note the nature of vegetation in relation to the soil type and to the wetness of the soil (all indications require confirmation by further investigation). Unusual green patches, reeds, rushes, willow trees and poplars usually indicate wet ground conditions.

e) Study embankments, buildings and other structures in the vicinity having a settlement history.

6) British Geological Survey, Global Seismology Group, Murchison House, West Mains Road, Edinburgh GH9 3LA.

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C.4 Site invespection for ground investigation

a) Inspect and record location and conditions of access to working sites.

b) Observe and record obstructions, such as power cables, telephone lines, boundary fences and trenches.

c) Locate and record areas for depot, offices, sample storage, field laboratories.

d) Ascertain and record ownership of working sites, where appropriate.

e) Consider liability to pay compensation for damage caused.

f) Locate a suitable water supply where applicable and record location and estimated flow.

g) Record particulars of lodgings and local labour, as appropriate.

h) Record particulars of local telephone, employment, transport and other services.

Annex D (informative)

Detailed information required for design and construction

D.1 Detailed land survey

a) Detailed survey of site and its boundaries, including levels referring to Ordnance datum, means of access, public and other services, and natural drainage network (see annex B).

b) Present and previous use of site and particulars of existing structures and obstructions, and whether they have to be maintained or demolished (see clause **7**).

c) Adjoining property and differences in ground levels with particulars of any adjacent structures including heights, floor levels, type of foundation and other details, and whether support is needed for these adjacent structures.

d) Location and depth, where known, of any underground obstructions, and features such as cavities and mine workings and tunnels with a full description (see annex E).

e) Location with co-ordinates of triangulation and traverse stations (Ordnance Survey and site) positions and levels of Ordnance Survey and site bench marks, true north points and date of survey.

f) Establish site bench marks and record their nature, location and description.

g) Whether easements are required.

D.2 Aerial photography

See clause **8** and **B.6**.

D.3 Ground conditions

Ground conditions, including the possibility of contaminated ground, are dealt with elsewhere in this code, e.g. section 2 and annex F.

D.4 Hydrography and hydraulic models

Structures in, adjoining, or near waterways require information on some or all of the following.

a) Requirements of statutory bodies controlling waterways, such as Port Authorities, Water Authorities, Planning Authorities and Fisheries.

b) Topographical and marine survey data to supplement, where appropriate, Ordnance Survey maps and Admiralty charts and publications (see annex B).

c) Detailed information about rivers, size and nature of catchment areas, tidal limits, flood levels and their relations to Ordnance datum.

d) Observations on tidal levels (referred to Ordnance datum) and the rate of tidal fluctuations, velocity and directions of currents, variations in depth, and wave data.

e) Information on scour and siltation, movement of foreshore material by drift; stability conditions of beaches, dunes, cliffs, breakwaters and training works.

f) Location and details of existing river and marine structures, wrecks and other obstructions above and below the water line. Include effect of obstructions and floating debris, etc., on permanent and temporary works, including clearances.

g) Observations on the condition of existing structures, such as attack by marine growth and borers, corrosion of metal work, disintegration of concrete and attrition by floating debris or bed movements.

D.5 Climate

Information on the following may be obtained from publications of the Meteorological Office (see **B.4**) and, where necessary, supplemented from local sources:

a) annual rainfall and seasonal distribution;

b) severity and incidence of storms;

c) direction and strength of prevailing and strongest winds with their seasonal distributions;

d) local air flow characteristics;

e) liability to fogs;

f) range of temperature, seasonal and daily;

g) humidity conditions.

D.6 Hydrology

Most sites are liable to flooding. Information on sources of published data are given in annex B. It can sometimes be advantageous to set up data collection, initially on-site, specifically orientated to the investigation, and then again later, at the construction stage. Parameters such as rainfall, wind, river and tide levels, maximum and minimum temperature and ground water levels can be measured, preferably on a regular daily or weekly basis. Where appropriate, the continuation of data collection may be possible with the liaison of the controlling statutory bodies. Even short-term data collection can provide a better understanding of the site conditions in the context of all previously recorded data.

Data is also available giving local and regional precipitation and flow data.

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D.7 Sources of materials for construction

a) topsoil;

b) fill for earthworks and reclamation;

c) road base and surfacing materials;

d) concrete aggregates;

e) stone for building, rip rap or pitching;

f) water;

g) effect on environment.

D.8 Disposal of waste and surplus materials

a) location and capacity of spoil tips, including those for surplus dredged materials;

b) requirements to safeguard nearby structures from ground movements and slips;

c) liquid waste and standards of pretreatment required;

d) solid waste;

e) access to spoil tips;

f) transport requirements;

g) effect on environment, particularly in respect of any contaminated waste materials.

Annex E (informative)

Site investigations in areas of mining, quarrying and natural cavities

E.1 Introduction

In many areas of the country, both underground mining and open cast quarrying have been carried out for the extraction of coal, ironstone, limestone, chalk, fire clay, tin, salt, and many other minerals. There are large areas covered by waste from mining and quarrying.

Many rock types contain natural cavities and these are widely distributed in the United Kingdom. The majority of these, however, occur due to dissolution of soluble rocks and are therefore associated with areas that contain limestone, chalk and evaporite deposits. Other processes can also form natural cavities.

Subsidence or instability may result directly from mining or quarrying, or may occur as a result of excavation or loading above areas already affected by mining or quarrying, or where natural cavities occur. Subsidence is accompanied by changes in surface slopes and by lateral strains. Instability includes ground disturbance and the formation of unstable slopes; there may be interference with the flow of underground water. Similarly, subsidence and instability may be associated with all other man-made cavities such as storage caverns, underground railways and civil defence structures etc.

Environmental and safety considerations in relation to areas of mining, quarrying and associated waste disposal or natural cavities may also include the possible presence of toxic gases, combustible materials, rising water levels due to mine abandonment, or mines and natural cavities acting as pollution pathways. In addition, a cavity of any sort may have achieved some level of metastability, which can easily be disturbed, for example, by changes in stress brought about by construction or even site investigation activities.

Careful consideration should be given to all these matters when undertaking a site investigation in areas of mining, quarrying and associated waste disposal or natural cavities.

E.2 Underground mining

E.2.1 *General*

It is convenient to consider the general aspects of underground mining first and then the potential hazards that might be expected from the underground mining of specific minerals. Site investigations should consider the following general aspects of underground mining when assessing potential hazards.

a) Old abandoned mine workings. These are often extremely old, completely obliterated on the surface and unrecorded. In addition, mining in the past was carried out by a variety of methods, often haphazard, and consequently it is rarely possible to estimate with any accuracy the effect such mining may have on any excavation or building undertaken above it. Progressive collapse of galleries can cause upward migration of cavities. The timescale of progressive collapse is normally considered to be unpredictable.

b) Modern mine workings. Mining today is planned in advance, and normally undertaken by mechanized methods, so that the areas that have been and are likely to be affected by it can be identified.

c) Abandoned mine shafts. The potential hazards associated with abandoned mine shafts present their own particular problems, these are dealt with in **E.2.3.3**.

E.2.2 *Particular types of underground mining*

E.2.2.1 *Coal mining*

In the past, coal mining has been carried out by a variety of methods. These include the driving of adits, frequently sloping, from the surface, usually known as "drift mines"; the sinking of shallow shafts and extracting around the base on a limited scale, or "bell pits"; partial extraction, leaving pillars at intervals to support the seam roof, called "room and pillar" workings; and also some mining by "total extraction". These old mine workings are on a limited scale and, because they are partially extracted, often remain open for many years. The pillars and supports in them gradually deteriorate and crumble, which can cause collapse and subsidence at any time or when excavations or building works are undertaken above them. Progressive collapse of the roof of mine openings results in the upward migration of the void space. The absence of records to identify the location of old coal mines, together with the variety of different mining methods, makes it extremely difficult to predict the effect their presence may have on works carried out in old coal mining areas.

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Most coal mining today is by mechanized longwall "total extraction" methods. Settlement is usually virtually complete within a few months after extraction, although the ground near the surface may be disturbed and the bearing capacity will be impaired where the mined out seam is shallow. In the total extraction of deep seams, surface disturbance, as distinct from subsidence, is insignificant, unless above these there are shallow seams that have been worked out in earlier years. It is still important to examine each case thoroughly. There are empirical procedures, based on statistics, from which reasonable estimates can be made of the form and amount of subsidence resulting from longwall total extraction of coal (see **E.2.3.1**). Subject to certain requirements concerning siting and the form of the structure, the Coal Authority or the present owner of active mines is required to pay compensation for damage caused by subsidence.

E.2.2.2 *Metalliferous and other minerals*

Metalliferous and other minerals occur and have been exploited in a wide variety of geological environments. Whilst many mineral deposits occur in conformable horizons, many others occur as non-conformable veins or massive ore bodies.

Shallow adit or shaft and gallery mining of chalk and associated flints, limestone, gypsum clay and other non-metalliferous minerals may present particular subsidence and instability problems. These are caused by haphazard methods of working, the weaker nature of the strata, upward migration of galleries, and solution collapse due to concentrated seepage of surface water. In chalk, not all mining was to extract chalk or flints; for example, deneholes are believed to have been sunk for storage.

Barytes and metallic ores (such as tin, lead, zinc and fluorspar) usually occur in tabular, irregular or massive orebodies, which are discordant with the geological sequence into which they have been placed. They are normally associated with hydrothermal processes that have occurred after igneous activity. Metallic ores can occur as tabular subvertical deposits in joints, fractures and faults. Such deposits are not therefore accessed by strata mining methods, which form regular boundaries, but by techniques resulting in very complex mine geometry. Surface instability may be caused by the collapse of adits or pillars between worked areas.

E.2.2.3 *Solution mining*

Uncontrolled pumping of brine to obtain salt was undertaken in the past, which produced cavities of unknown size and extent, and consequently caused widespread subsidence, for example in Cheshire, Staffordshire and Worcestershire. Modern solution mining for salt and potash is therefore controlled, in order to produce cavities of such size and spacing that subsidence or instability will not be caused, although problems may still occur.

E.2.3 ***Potential hazards***

Potential hazards in relation to site investigations in areas of underground mining include: surface instability above workings and adjacent to shafts; differential settlement; combustion of natural or waste materials left in the workings; gas escapes during excavation; and rising water tables in abandoned mines. These are dealt with separately as follows.

E.2.3.1 *Mining subsidence*

There is a large amount of literature dealing with the principles of mining subsidence; there is, however, no scientific procedure for computing the dimensions and form of subsidence resulting from the various methods of mining undertaken in the past. There are, though, empirical procedures based on statistics, from which good estimates can be made. This is especially true in respect of subsidence resulting from modern "total extraction" longwall mining of coal, which causes a more positive and predictable subsidence wave at the surface. Unexpected subsidence may still sometimes occur.

Details about the procedure for estimating subsidence caused by modern total extraction coal mining are given elsewhere [211] [212].

The Coal Authority normally assists persons proposing development on coal mining areas by providing information about past, current and proposed future mining and by estimating the extent of subsidence and its effect on structures. The investigation should explore such factors as depth, thickness and inclination of the mine seams. Consideration should also be given to the junction of underground mines with open cast workings or quarries. Underground workings that are at shallow depths and beneath backfilled, open cast workings present particular problems for site investigation. Investigations should be planned on a case-by-case basis.

E.2.3.2 *Void migration*

Where old mine workings are still open there is a risk of collapse at some time in the future. Such collapses can lead to voids migrating upwards, unless and until they are filled by the collapse material, because broken rock fills a greater volume than solid rock according to a bulking factor. If the workings are close to the surface the void can migrate to the surface and cause a collapse or crown hole. The vertical distance through which a void can migrate, before infilling itself by bulking and thus becoming stable, is difficult to assess. Made ground, or fill, is normally considered as having no inherent strength to resist void migration and the assumption is usually made that if a void reaches the base of any fill then it will reach the surface. Guidelines for ranges of values for the thickness of solid rock through which voids can migrate are available [213].

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E.2.3.3 *Abandoned mine shafts*

Abandoned mine shafts almost always present a foundation problem. The building of new structures over or near old shafts should be avoided, unless it can be shown that the shaft is inherently stable, or that there is adequate evidence that the shaft has been properly filled and compacted. It should also have been provided with an adequate capping to prevent the possibility of any further subsidence under the proposed structure.

Old abandoned mine shafts that may have looked as though they were filled to the surface were often only filled from some intermediate staging, leaving a void below. This staging could collapse at any time, causing subsidence or instability at the surface. In addition, shafts that are known to be filled from top to bottom have collapsed because the filling has moved into the working below. Where the upper part of the shaft passes through overburden, subsidence of the filling may bring about the inward collapse of the shaft lining, causing a crater to form at the surface. The site and diameter of this crater depends on the nature and depth of the overburden and the extent of subsidence of the filling within the shaft. Very large craters can form where the overburden is granular.

Under Section 151(1) of the Mines and Quarries Act 1954 [214] the owner of an abandoned mine is responsible for securing the shafts and the outlets of the mine to ensure that no person may accidentally fall down a shaft or enter an outlet. The owner of an abandoned mine in the vicinity of a proposed new structure should therefore be consulted about the method used for securing the shafts and outlets associated with the mine.

E.2.3.4 *Gas emissions*

Carbon dioxide and methane are the most common potentially hazardous gases found in old mineworkings. Methane can be explosive at certain concentrations while carbon dioxide is an asphyxiant. Other gases that may be present include hydrogen sulphide and carbon monoxide, both of which can be combustible and highly toxic. Deoxygenated air may also be present. Due to the different densities of the gases encountered stratification may occur. Gases emerge through porous rocks, joints, fractures and old mine openings. Gas migration paths are effectively unpredictable and locations up to several hundred metres away from a source can be at risk [215] [216] [217].

Radon gas has been linked with lung cancer and may occur in metalliferous mining districts. A report detailing the counties where the radon levels are highest has been published [218].

E.2.3.5 *Spontaneous combustion*

This occurs where air paths are available and the air flow through a coal seam is sufficient to start and maintain oxidation processes, but is not strong enough to disperse the resulting heat. It is therefore found most frequently in areas of active or abandoned mine workings, but may also occur in a virgin seam close to its outcrop. Coal seams vary in susceptibility to spontaneous combustion. The two main effects of spontaneous combustion are the production of combustion gases and the collapse of the coal, which occurs during and after burning.

E.2.3.6 *Hydrogeological considerations*

Underground mining may affect the hydrogeology of an area in a number of ways. Groundwater levels in operating mines may have been reduced and flow paths altered due to pumping, in order to exploit minerals at depths below the natural groundwater levels. Pumping may have continued for a number of years or, in some cases, for hundreds of years. Where a mine has been abandoned and pumping stopped, groundwater levels begin to rise and springs and streams once dry may begin to flow again. In addition, collapsed workings or grouting to stabilize workings may change flow paths, causing water to emerge where previously it did not occur. As the groundwater levels rise the water may become contaminated by materials related to the particular product being mined. Examples of such materials are heavy metals in non-ferrous metalliferous mining areas and high acidity due to oxidation of pyrites in Coal Measure mines. Seepages of this contaminated groundwater at natural outlets or abandoned mine entries can create an environmental problem, which needs to be identified. Remedial action may need to be taken.

Rising groundwater levels may bring problematic gases, such as those discussed in **E.2.3.4**, closer to the ground surfaces and may concentrate these in mineshafts.

E.2.4 ***Site investigation procedures***

Thorough documentary research should be carried out using original records where possible. Old photographs and aerial photographs should be used when available. Several publications are available giving guidance on sources of information for mine related site investigations [211] [212] [213]. In addition British Geological Survey Technical Reports are available for selected areas of the country. The importance of this initial desk study cannot be overemphasized and should allow the scale of any potential hazards to be identified. This enables an early assessment of the available techniques, so that the best information is obtained during the ground investigation and precautions are taken where necessary during this work to ensure safety of site staff.

The site should be examined in considerable detail in order to search for evidence of surface working, demolished mine buildings, mining waste and for damage to existing structures. The surrounding area should also be examined. The underlying geology should be determined, in particular mineral outcrops and faults. Preliminary cored boreholes may be necessary to establish the geological sequence and the existence and level of workable minerals beneath the site.

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At an early stage in planning the investigation, it should be established whether structures could be re-sited at more favourable positions on the site. It is usually advisable to postpone the final layout of the project until the mining situation is resolved. Investigation should then establish the nature and geometry of any cavities, the nature of the cover, and ground conditions down to an appropriate level beneath the cavities. Earth-moving equipment may be necessary to firmly establish the presence and location of mineshafts.

Provided overburden is not more than about 3 m thick, the excavation of trial pits or trenches can be used to examine the upper part of the rock. These can also be used for exploration of drifts and the tops of mineshafts. Drilling rigs can be used for exploration of filled mineshafts. When working above mineshafts that are thought to be filled, it is essential that measures be taken to ensure the safety of men and equipment, in case of rapid collapse of the filling.

For the determination of underground cavities and seams, direct methods such as rotary percussive or rotary drilling using rock roller bits are normally used. Indirect methods such as geophysical and geochemical methods may also be used but it may be necessary to follow up the findings with direct methods to confirm the presence of workings. The merits of indirect and direct techniques are discussed in, for example [212] [213].

The number of holes required depends on the type and extent of the proposed structures, the site conditions and the foundation design envisaged. The maximum depth to which exploratory boring should be carried out is determined by the nature of the structures proposed. During the drilling of investigation holes drilling runs should be timed and flush returns checked, as these may give a good indication of the presence of cavities or infilled voids. When cavities are encountered, borehole television cameras or photographic cameras can be used to examine them and plan further exploration. Boreholes are usually placed on a grid, but when “room and pillar” type mining is suspected, the grid should be broken to prevent possible coincidence with pillars. Inclined borings may be required to investigate steeply dipping workings.

Guidelines on safe practices for site investigations can be found in [22] [28] [219].

These guidelines give advice on protective equipment to be worn when dealing with contaminated ground and water, and precautions to be taken when gases are encountered. It also recommends safe drilling practices to be carried out when working on sites with mineshafts and mine workings. Where safety conditions permit, it is preferable to gain access to the cavities so that they can be fully mapped. Experienced mining geologists or engineers should be employed for this work. Reference should also be made to annex E and annex F of this standard. Under the Health and Safety Regulations (1995) [222], it may also be necessary to inform the Health and Safety Executive when undertaking work in the area of old mine workings.

The geological sequence should be plotted on longitudinal sections, together with the formation or foundation level of the proposed structure. The depth to which significant additional stresses in the soil are created by the proposed structures should be assessed, and the existence within these depths of any mine workings, backfilled areas, or areas of instability created by mine workings should also be plotted. This study should include the possible effects of changes in groundwater flow and drainage. Flow rates and levels of gases and groundwater may be dependent on atmospheric conditions and it is therefore advisable to monitor the presence of these over a period of time, especially where projects are adjacent to rivers or in tidal areas. The possibility of future mining or quarrying should be ascertained and its effects assessed.

E.3 Opencast mining and quarrying

E.3.1 *Opencast mines and quarries*

Several stability problems may be recognized in the working of minerals in areas of proposed and existing development. Where buildings already exist, development consideration has to be given to the distance between the top of the quarry faces and the buildings. Where development is proposed adjacent to existing workings, similar criteria apply whether the workings are backfilled or remain unfilled.

Occasionally, development may be contemplated within worked-out quarries, particularly for industrial purposes. Blasting in hard rock quarries often causes loosening of material well behind the face, which may be acceptable in the mineral industry as a short-term risk, but not in general civil engineering, where long-term risks of slope failure or rock falls are of great importance.

E.3.2 *Backfilled quarries and opencast mines*

These are often filled with waste material and then sometimes landscaped, or have become overgrown and blend into the landscape, so that they are often difficult to distinguish from natural ground. The backfill materials can include domestic, trade and industrial waste, as well as non-productive soil and rock from the quarry workings. Many such backfills can include potential gas-producing materials. In many cases, the backfilling is not controlled and both the quality and the degree of compaction of the backfill can be extremely variable. Changes in groundwater conditions may have a very significant impact, for example on the first inundation of a backfill. Differential settlement is likely to occur across the edges of quarries and across areas where the base relief of the quarry varies rapidly.

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E.3.3 ***Site investigation procedures***

When development is proposed near open quarries, consideration should be given to the stability of the quarry faces and the distance of unstable ground behind. This distance is influenced by various factors: the materials being quarried; the potential mode of failure of the faces, which may depend on direction, spacing and angles of bedding and jointing; the hydrogeology; the depth and width of such workings and the slope of the faces. It should be noted that the limits of opencast workings shown on mine abandonment plans are sometimes those of the seam area extracted and not the limit of the pit. Site investigation should be carried out to obtain the necessary parameters for a study of long-term stability, taking into consideration possible changes in the groundwater conditions.

Construction may be considered in areas where workings have been backfilled [221]. Investigations should be carried out to determine the depth and extent of such workings together with the kinds of backfilling materials and their state of compaction.

Areas of opencast coal and ironstone working may be well documented, but the nature and state of compaction of the backfilling materials usually requires investigation. Smaller mineral workings may not be so well documented. Domestic, trade and industrial waste backfilling is often liable to long-term deterioration with large settlements. Industrial waste may contain obnoxious or corrosive chemicals, some of which may be poisonous and could form an industrial hazard if disturbed. Other chemicals may be highly aggressive to concrete in foundations and building services. Many backfills also have a propensity to generate gas. Where these conditions might occur, the guidance given in annex E and F should be followed.

The location of any "buried wall" beneath the backfill should be determined, because large differential settlements may occur in structures built over such areas. This may be determined from available records or by sinking boreholes. Some success has been achieved in this connection with the use of geophysical traversing.

E.4 Mine waste disposal areas

E.4.1 ***Tips***

The modern working of mines and quarries generates large quantities of waste materials in a solid form. Some waste materials in quarrying may be temporarily tipped for eventual return to the workings as backfill, whereas underground mining waste is normally disposed to tips. Tipping of mine and quarry waste is subject to stability requirements of the Mines and Quarries (Tips) Act and regulations [222] [223].

The mode of tipping influences stability, because wide variations in material characteristics may occur. Other factors influencing stability may include the effects of layered construction and wet weather working, together with long-term weathering of waste heap materials. Much attention has been given to the stability of waste, and a number of publications have been produced on the subject [224] [225] [226].

Fires may occur in tips by accident or by spontaneous combustion. The resulting fumes may be dangerous, as also may be cavities formed by combustion.

E.4.2 ***Mine waste***

Waste produced from mining and quarrying operations may generate contaminants especially during weathering processes. The contaminants that enter the environment are dependent on the geological horizons from which the minerals have been extracted and the techniques used during processing. This may result for example in the production of heavy metals or high acid waters or gases.

E.4.3 ***Slurry lagoons***

A number of waste products from underground and surface mineral workings are in a liquid form and are settled in lagoons. The enclosure of lagoons is sometimes carried out using bunds formed from solid waste fractions. On other occasions, slurry may be pumped into disused quarry workings. The slurry may consolidate with time, and a characteristic desiccated crust may form on the surface. However, the liquid waste often only solidifies on the surface, leaving a reservoir of liquid or semi- liquid slurry beneath it that may remain concealed for many years, forming a potential hazard to many types of development.

Slurry lagoons of liquid waste are subject to the same statutory requirements as tips of solid waste mentioned in **E.4.1**; and publications dealing with the stability of waste consider slurry lagoon stability ([222] to [227]). Consideration should also be given to the environmental hazards of the waste should it penetrate adjacent water courses or an underlying aquifer.

E.4.4 ***Site investigation procedures***

All waste heaps and slurry lagoons should be regarded as civil engineering structures, and appropriate site investigations carried out to ensure their stability [228]. Where investigations are confined to the stability of existing waste heaps and slurry lagoons, an initial inspection should be undertaken [229], but the searching of any existing records should precede a further survey. Where the existing tip contains industrial waste, a chemical study may be required (see annex F). Plans and maps may exist with quarrying concerns, showing the extent of pre-existing works; old Ordnance Survey and other maps may give useful information about the pre-existing ground surface, including evidence of ground slips and springs. Photographic records of tips and tipping may be available either as aerial photographs or as simple ground level views (see **E.6**)

Where waste heaps or lagoons are proposed on a new tipping site, the subsoil conditions should be investigated to determine their suitability for the proposed stress and seepage conditions. Department of Environment Waste Management Papers give guidance on the current legislation in relation to land filling wastes especially with regard to controlling landfill gases and leachate.

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Construction on slopes or on soft ground deserves particular attention; the hydrogeological regime and past or future mining subsidence can also influence waste heap behaviour. Consideration should be given to the influence of the proposed tipping on the existing ground water pattern. Where waste heaps adjacent to lagoons may be subject to possible flooding, the likely effects on stability should be anticipated in design, as should the effects of any accumulations of water. The grading, structure, shear strength and permeability of the waste materials that it is proposed to tip should be determined [230].

Lagoon bunds should be considered as earth dams and designed accordingly. The stability of the foundation should be investigated and seepage studies carried out.

The discharge of slurry into lagoons poses problems of differential sedimentation with segregation of coarse and fine soil fractions in different areas. The consolidation characteristics of these soil fractions vary and this should be examined. Any crust that forms on top of the slurry should be investigated to determine the stability of any future solid waste layer, which may be used to cap off the lagoon.

E.5 Natural cavities

E.5.1 *Geological occurrences and formational processes*

Natural cavities are formed by a variety of processes, which mainly affect sedimentary strata. These features are therefore not uniformly distributed throughout Great Britain. Consideration of the processes involved in cavity formation assists in assessing their influence on a particular project.

E.5.1.1 *Dissolution*

Dissolution usually affects limestone, chalk, rock salt and gypsum-bearing rocks and these lithologies contain the majority of natural cavities in Great Britain. Rocks are progressively dissolved on contact with water, forming a wide geometrical variety of cavities. This process, although more dominant in the upper zones of soluble rocks, can produce cavities at depth by dissolution along discontinuities.

Cavities formed by dissolution occur mainly in Cretaceous Chalk and Carboniferous Limestone. They also occur in other geological units containing limestone and evaporites, where they can be locally significant.

E.5.1.2 *Cambering*

Cambering normally occurs in a competent jointed lithology, such as limestone or sandstone, where it overlies a less competent clay, marl or mudstone on a slope. Natural cavities may form in the jointed stratum by tilting and vertical movements, which open up the joint planes to produce gulls or fractures. Natural cavities formed in this way occur most frequently in Jurassic and Lower Cretaceous strata.

E.5.1.3 *Marine erosion*

A combination of chemical weathering, corrosion, attrition and hydraulic action can progressively break down the lithologies, forming a cliffline, and can also produce natural cavities. This erosion occurs preferentially along discontinuities in competent rock, which then produces cave systems. Cavities formed by this process can be found in a wide range of geological units. The effects of marine erosion on any geological unit are influenced by structure, rock type and strength in relation to cavity formation.

E.5.1.4 *Other processes*

These include soil piping, scour hollows, fault movement and erosion and are normally considered to produce minor cavities. Cavities produced by these processes tend to occur in a wide range of geological units, where their characteristics such as structure, rock type and strength have a more dominant influence on cavity formation.

E.5.2 *Potential hazards*

Potential hazards in relation to site investigations in areas of natural cavities mainly include the possibility of ground surface subsidence and the preferential pathways for water and contaminants.

E.5.2.1 *Subsidence*

Progressive subsidence can occur due to consolidation of loose infill material within the cavity. More rapid subsidence may occur where air filled voids are destabilized and migrate to the surface, or where natural cavities encounter mines and the infilling deposit is allowed to flow into the open workings. Water flows or lowering of groundwater levels are the main triggers for subsidence and these can be caused by human activities. These activities can include the construction of soakaways, leaking water services and the undertaking of groundwater lowering during pumping tests or exclusion works. In addition static or dynamic loading of the surface during construction works may cause instability of cavities due to overstressing of the ground.

E.5.2.2 *Water and contaminant pathways*

Natural cavities may provide preferential pathways and in turn increase local permeability in important aquifers within Great Britain. This can cause concern due to the rapid movement of contaminants in aquifer protection zones. Natural cavities below waste disposal sites could result in damage to the containment system and consequent migration of leachates through the pathway they provide into groundwater.

Where undetected and untreated, natural cavities may also result in leakage from surface water reservoirs, should they occur below them.

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E.5.2.3 *Mining and tunnelling*

Natural cavities may introduce groundwater and gas into mines and tunnels, presenting a health and safety problem to construction workers. Instability problems may also occur.

E.5.3 *Site investigation procedures*

While assisting in identifying the geological horizon and topography of a site, Ordnance Survey and geological maps may not help in locating natural cavities. More information may be gained from aerial photographs and previous site investigations in the site locality. A database exists of recorded occurrences of natural cavities and this could be of assistance [233].

Site reconnaissance or geomorphological mapping, to identify surface evidence of underlying natural cavities, is essential. This helps to identify areas of potential hazard where further ground investigation can be targeted. It should be borne in mind, however, that some natural cavities have no surface expression. Local pot-holing enthusiasts could also be of considerable assistance.

The guidelines given in **E.2.4** concerning site investigation procedures also apply in areas of natural cavities. Probing methods may not provide direct evidence of voids and can require further work to ensure a correct interpretation of the results produced. Further details on the techniques available for natural cavity detection are available.

E.6 Locations and sources of information

E.6.1 *General — Mining*

The maps and memoirs of the British Geological Survey (previously the Institute of Geological Sciences) show where minerals may occur (see annex B). The survey's National Geological Records Centre holds an archive of unpublished information giving details of the occurrence and location of shafts and mine workings. Early Ordnance Survey maps (see annex B) show known shafts and refuse tips associated with old mine workings, but many shafts were sunk, abandoned, and covered over long before such maps were made. Frequently, old shafts and refuse tips not visible on the surface may be revealed by air photographs.

The Catalogue of Abandoned Mines published by HMSO in several regional volumes lists many abandoned mines by map references. Although the Catalogue went out of print in 1929 and is no longer generally available, it is kept up to date in respect of coal mines by the Coal Authority and in respect of all other mines by the Health and Safety Commission in its Plans Record Office (see following text for details). Records of old abandoned mines are by no means comprehensive, as there was no statutory requirement for owners to deposit plans of abandoned mines with the Inspectorate of Mines and Quarries until 1872.

The Department of the Environment has published a "Review of Mining Instability in Great Britain" to assist planning authorities and developers [212].

This provides ten regional reports with maps detailing the extent of mining in England, Scotland and Wales. Technical reports and case studies are also provided.

Local enquiries to mining and quarry firms, mining consultants, mineral agents, estate agents, local authorities, or even individuals can often produce information about abandoned mines and quarries that is unobtainable elsewhere.

The current statutory requirements relating to the plans of abandoned mines are set out in Section 20 of the Mines and Quarries Act 1954 [214]. This Act requires the owner of an abandoned mine to deposit plans of the mine with the Inspector for the District, and requires arrangements to be made for their preservation. Plans of abandoned mines are preserved in the following places:

Coal mines	Plans Record Office of the Coal Authority in Bretby, Derbyshire.
Oil shale mines	Plans Record Office of the Coal Authority in Edinburgh.
All other mines	Plans Record Office of the Health and Safety Commission[7)] Copies of plans of abandoned mines in Cornwall are now sent to the County Records Office, Truro.

Similar statutory requirements for the depositing and preservation of plans of refuse tips associated with abandoned mines and quarries have existed since 1969 and are set out in Section 7 of the Mines and Quarries (Tips) Act 1969 [223].

Sectional Mineral Valuers of the Department of Inland Revenue at Newcastle, Leeds, Birmingham, London, and Cardiff, or the Chief Valuer at Edinburgh may be able to give some information on the possibility of mining subsidence or instability at sites in their areas.

There are three annual publications that give general information on current mining and quarrying operations. The first lists coal, stratified ironstone, shale, and fireclay mines; the second lists all other mines; and the third lists currently operating quarries.

E.6.2 *Particular — Mining*

Further sources of information on, and locations of some particular minerals are given below, but these are by no means comprehensive. Many other minerals are mined and quarried, but the distribution of their workings and records, if any, are too widespread and numerous to be listed.

a) Coal and associated minerals (stratified ironstone, shale). The Coal Authority has detailed information on current coal mining operations. In addition to the material that is statutorily preserved by it, the Authority often has additional information about

[7)] Plans Record Office, Health and Safety Commission, Thames House North, Millbank, London SW1.

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abandoned coal mines. It will provide a brief written report on the risk of subsidence or instability at any particular site from coal mining and make a nominal charge for such a report. Information on open cast coal quarries can be obtained from the local Area Office of the British Coal Open Cast Executive.

b) Salt. The Cheshire Brine Subsidence Compensation Board at Nantwich has detailed information about solution mining of salt in Cheshire.

c) Iron. The Ore Mining Division of the British Steel Corporation at Scunthorpe, and Ore Divisions of its various subsidiary companies throughout the country has detailed information on the mining and quarrying of iron ore.

E.6.3 *General — Natural cavities*

The Department of the Environment has published a review of instability due to natural underground cavities in Great Britain [212].

This document has been compiled to assist planning authorities and developers. Ten regional reports with maps have been produced detailing the extent of the occurrence of natural cavities in Great Britain. Technical reports detailing the nature and occurrence of natural cavities, a review of site investigation procedures and void treatment are also available.

Information may also be gained from British Geological Survey maps and memoirs, Ordnance Survey maps and aerial photographs.

Annex F (informative)
Site investigations on contaminated land

F.1 Introduction

F.1.1 *General*

Many site investigations are undertaken on land that has been disturbed or influenced by a previous or current use of the land. This annex advises on the additional requirements for the objectives, planning, performance and interpretation of such conditions.

Further information can be found in BS 6068 and BS 7755. In addition, DD 175 is currently being revised, which should provide further guidance on this subject.

F.1.2 *Definitions*

Contamination of land may be defined in a number of different ways. For the purpose of this code of practice, land in which the ground or groundwater has concentrations of one or more potentially harmful substances elevated above "normal background levels", or in which concentrations of toxic or explosive soil gases occur is deemed contaminated. The natural concentrations of some substances may be such as to present a hazard to some targets (e.g. salt marshes, metalliferous ore bodies, etc.). It is man-made contamination, however, that is of most concern, due to its "random" occurrence, diversity, and possible severity and instability.

Depending upon the past history of the site, and surrounding areas, the ground may be contaminated by organic or inorganic chemicals, hazardous gases, biological agents or radioactive elements. For the purpose of this code of practice, the terms "contamination" and "investigation" are taken to refer to all types of harmful substances.

F.1.3 *Sources of contamination*

Land may be subject to contamination arising from a wide range of activities carried out either on the land, or on adjacent areas. Some of the principal contaminative activities are:

— deposit or burial of industrial and domestic waste;

— spills or leaks of noxious liquids at the surface, or from underground tanks, pipes and drains;

— demolition of industrial structures and dispersal or burial of contaminated rubble and other materials;

— contaminated fill material imported to the site;

— spraying of agricultural chemicals (pesticides, herbicides etc.) using inappropriate procedures;

— stockpiling of materials such as road salt, mining waste, etc;

— burial of animal carcasses using inappropriate procedures.

Contamination has become ubiquitous within industrial and urban areas of the UK, but may also be found in rural areas. The potential presence of contamination should be considered at the desk study stage of all site investigations (see **6.2**).

F.1.4 *Types of sites*

The types of sites where contamination of the ground might occur include:

— landfill sites, and land surrounding these sites;

— former sites of heavy industry (e.g. steelworks, ship building etc.), and gas works sites;

— former or current chemical and manufacturing plants, particularly those using or storing bulk liquid chemicals or discharging significant quantities of effluent;

— sewage farms and sewage treatment plants;

— breakers' yards, timber treatment works, railway sidings;

— all works employing metal finishing processes (e.g. plating, paint spraying etc.);

— fuel storage facilities, garages and petrol forecourts;

— areas of filled ground;

— farm land;

— former mining sites (particularly mines for metal ores).

References [232] to [240] provide further information on the types of industrial sites of most concern.

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F.1.5 ***Nature of contamination***

Contamination of the ground may occur in one or more phases. It may be present in solid form (e.g. industrial residues or products, mining wastes etc.); it may occur in liquid form (e.g. oils, solvents, acids etc.) dissolved, entrained and/or floating on the groundwater, or within the partly saturated zone; or it may occur as gases (e.g. methane, volatile organic vapours, etc.) within the partly saturated zone, or entrained or dissolved in the groundwater. It is common to find that the groundwater is affected where a solid source of contamination is present, and conversely, chemical contamination may adsorb to the soil particles when liquid chemicals are split on the ground, or a plume of contaminated groundwater passes through the ground. Toxic or explosive soil gases may be present in the ground as a result of microbial decomposition of solid or liquid constituents of the ground, or by migration from adjacent areas.

Contamination may conveniently be grouped into the following categories:

— by metals;

— by salts;

— by acids and alkalis;

— by organic compounds and vapours;

— by fibrous materials (e.g. asbestos);

— by pathogens, viruses, bacteria etc.;

— by radioactive elements;

— by gases (e.g. methane, carbon dioxide, hydrogen sulphide, radon etc.) that either occur naturally, or arise from the biodegradation of chemical contamination in the ground.

The investigation of sites potentially contaminated by radioactive material, explosives or pathogens, etc., has special requirements beyond those adopted for other forms of chemical contamination. References [241] to [244] provide particular information on the precautions necessary when working on such sites.

F.1.6 ***Significance of ground contamination***

The actual (and potential) presence of ground contamination on a site affects all aspects of the planning, performance and reporting of a site investigation. The actual presence of contamination is also likely to affect the scope and method of ground works subsequently carried out on the site, and may affect the foundation design and intended use of the site.

The principal impacts of ground contamination on the site investigation process concern:

— health hazards that may arise during the execution of the field sampling and laboratory testing, requiring strict health and safety precautions to protect both site investigation personnel and the general public;

— the protection of surface and groundwater resources from the discharge of contaminated water arising from the investigation, or from the provision of pathways through an aquiclude;

— the possibility of wind-blown contaminated particles being deposited on land or surface water outside the boundaries of the site and thereby creating a hazard;

— the need to employ strict sampling, sample handling and testing protocols to ensure that the data collected are sufficient in type, quantity and quality so that the site can be categorized with the required degree of confidence;

— the need to employ qualified and suitably trained staff experienced in contaminated ground investigation to plan, direct, execute and monitor the work (see also clauses **17** and **F.5.2**).

In the remainder of this annex, advice on current good practice at each stage of the site investigation is given.

F.2 Planning the investigation

F.2.1 ***General***

The general considerations for the performance of a site investigation are described in section 1. This annex describes the additional considerations appertaining to the plan for the investigation of ground which might be, or is known to be, contaminated. The presence of contamination can have a major impact on the extent and type of investigation that is required. It also necessitates the imposition of special procedures to ensure the safety of personnel who are working on site, or who might be handling contaminated material in the laboratory.

F.2.2 ***Objectives***

At the outset, the client and the engineer, or other specialist who is responsible for the investigation, should clearly establish the objectives and strategy. This is particularly important for chemical site investigation, because the work may be required for a number of different reasons and this can affect the method, depth, extent and intensity of the investigation that is undertaken.

Investigations of contaminated ground may be required:

— in combination with a geotechnical investigation for a development scheme; or

— to assess the presence or absence of ground contamination of a site subject to a proposed property transaction; or

— to provide data for an environmental impact assessment.

In the case of a proposed building or civil engineering development, it is usually advantageous to undertake the geotechnical and chemical investigation at the same time. However, it may be necessary to carry out initial phase(s) of chemical investigation in advance of the main geotechnical ground investigation, in order to assess health and safety risks, or for another specific reason not involving geotechnical design.

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It should also be recognized that the sampling locations and techniques most appropriate for a geotechnical ground investigation of a site may not be able to provide useful samples for chemical testing, and vice versa (see **F.5.7**). The plan for the work must cover the requirements of both types of investigation.

F.2.3 ***Remote sensing techniques***

There are a number of remote sensing and geophysical techniques that can provide useful data on the potential contamination of certain sites. The use of these techniques should be considered at an early stage in the planning of the investigation, because the results can assist in the location of ground investigation positions and in some circumstances may substantially reduce the number of investigation points needed.

The following techniques are used.

— Infra-red thermographic surveys. These surveys, usually carried out by helicopter, detect temperature differences in the ground surface and can assist in the investigation of landfill sites and spoil tips.

— Infra-red photography. This type of photography detects differences in reflected energy, and can also highlight distressed vegetation, which may result from contaminated ground or landfill gases.

— Conductivity surveys. These may be used to interpret substantial variations in groundwater quality and the presence of buried metallic objects.

— Ground probing radar. This technique makes use of electromagnetic pulses emitted and received by equipment drawn across the surface of the ground, usually on a grid pattern. It can be useful for detecting buried tanks at shallow depth, but is not suitable in clay soils or water-saturated soils.

F.3 Desk study

F.3.1 ***General***

Whether the site investigation is being undertaken principally for geotechnical or chemical purposes, a desk study must be carried out as the first stage. Information that should be obtained and reviewed for the desk study is described in annex A, and sources of information are given in annex B. Specific guidance relating to contaminated sites is available in DOECLR3 [245].

The desk study provides the basis for the design of the subsequent ground investigation. This includes not only the number, type and depth of investigation points, permissions to carry out the work, access arrangements, etc., but also the health and safety requirements and sampling and sample handling protocols.

The assessment of the potential presence of contamination on a site from existing information should be part of the desk study of every site. The presence of contamination is usually associated with the former uses of the land or adjacent areas, although unusual geological conditions or random events (such as fly tipping, pipeline spills etc) may also result in contamination.

In addition to the identification of contamination sources, the desk study information should be assessed to identify pathways for contaminant movement, both present and potentially in the future, as well as sensitive targets. Short-term events that could affect these risks, such as flooding, should also be considered. Long-term events requiring consideration include the closure of local mines, developments on neighbouring sites, rising ground water and possible changes in sea level.

F.3.2 ***Principal sources of information***

The following sources of information are of particular importance in establishing the potential for ground contamination.

— Records held by current and previous landowners of the property. Such records may include plans, details of processes, chemicals used, and effluent characteristics, as well as previous site investigation data.

— Owners or employees of current or previous industrial users of the site. Invaluable information on previous site practice can sometimes be provided by interviewing such persons.

— Historical maps and aerial photographs. These records can assist in determining the different stages of development and redevelopment on a site, as well as identifying the locations of structures and other features on the site.

— Previous site investigation data, where this includes the results of chemical testing.

— Records held by the local authorities, in particular information held by waste disposal authorities, planning departments and development authorities.

— Records held by the Environment Agency in England and Wales and the Scottish Protection Agency in Scotland relating to groundwater and surface water quality in the area.

— Effluent discharge consents, issued by the water companies or former water authorities.

F.3.3 ***Evaluation of data***

The examination of available information and consultation with persons having a personal or regulatory interest in the site usually enables a qualitative assessment to be made of the risk of ground contamination, the principal types of contamination that may be present, and the most likely areas of contamination. A series of guidance notes produced by the Department of the Environment's Interdepartmental Committee on the Redevelopment of Contaminated Land (ICRCL) are useful in identifying contaminants related to specific industries and former uses ([232] to [239]).

The desk study usually reveals both the hydrogeological sensitivity of the site and the probable sequence of strata. However it is common for very little quantitative chemical test data to be available at the desk study stage. Where such data does exist it should be examined with due regard to the soil profile and the origin of the materials.

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From the evaluation of historical, geological, hydrogeological and chemical test information, and with regard to the results of the site reconnaissance visit (see **F.4**), the scope, method and phasing of subsequent ground investigations, together with the necessary safety precautions and sampling protocols, can be decided.

A comprehensive method statement should be prepared, documenting the following: methods of investigation; any special procedures required in order to protect the environment, e.g. disposal of contaminated soil and ground water; the responsibilities of the various parties involved in the investigation; the quality control procedures; and the safety measures for the investigation. The Site Investigation Steering Group have proposed a classification of potentially contaminated sites into three categories (red, yellow and green), according to the degree of perceived hazard to ground investigation personnel [28].

F.3.4 *Consultations*

If during the course of the desk study it is determined that there is a potential for contamination of the site, and it is intended to carry out a subsequent ground investigation, consultation should be undertaken with the Environment Agency in England and Wales and the Scottish Protection Agency in Scotland. The purpose of this consultation is to agree an acceptable method of investigation that will provide the necessary information without prejudicing groundwater or surface water resources.

Similarly, if investigations are to be undertaken on agricultural land, consultations should be held with the Ministry of Agriculture Food and Fisheries (MAFF) to determine whether there are any access restrictions.

Consultations are also required with the local water authority and the waste regulation authority if contaminated groundwater is to be directed to sewer, or contaminated soil is to be transported off site.

F.4 Site reconnaissance visit

A reconnaissance visit should be made to the site during the course of the desk study. The visit allows a visual inspection of the site and its immediate environs, and is undertaken for the following purposes:

— to validate background information on the site collected during the desk study;

— to collect additional information about the site;

— to assist in the planning of subsequent phases of site investigation.

The visit should be undertaken after the available background information has been collected and reviewed, and prior to the completion of the planning stage of any subsequent ground investigation. This is particularly important for potentially contaminated sites since special safety precautions may be necessary for persons carrying out the site reconnaissance.

General notes on the planning and execution of reconnaissance visits are given in annex C. For sites known to be contaminated, and for sites indicated by the desk study to be potentially contaminated, the validation and investigation of the following information is especially important:

— the presence of tanks or drums, whose contents and locations should be marked on plans;

— the actual locations of drains and other buried services (which may be sources or preferential pathways for contamination);

— the locations of raw material and waste storage areas, and any process operations that are being carried out on the site;

— indications of contamination such as odours, staining of the ground or paving, lack of (or abrupt changes of) vegetation, or preponderance of resilient species such as buddleia.

It is preferable for the person(s) who undertake the site reconnaissance to have been fully involved in the desk study. It is also useful to arrange to carry out the site reconnaissance in the company of someone familiar with the site layout and former use of the land (such as a plant manager or safety officer in the case of an industrial site).

A photographic record of the visit should be made including general views of the site and features of special interest. In the event that the site reconnaissance reveals health or environmental risks for which immediate action may be required, these matters should be drawn to the employer's attention.

F.5 Investigation of contaminated ground

F.5.1 *General*

An investigation of contaminated ground may be required for a number of different reasons which may, or may not, be associated with a geotechnical assessment of the site (see **F.2.2**). The particular sampling programme, the method and scope of the contaminated ground investigation, and the qualifications of the personnel who should supervise the field work are principally determined by the objective of the investigation and the expected ground condition. Factors such as degree of confidence, timescale and cost are also important. General guidance on the principles of ground investigation works where contamination may be present is given in the following subclauses.

F.5.2 *Site personnel*

All site personnel involved in the investigation should be fully briefed in advance of the works. This briefing should include:

— potential health and safety risks posed by the site;

— additional safety procedures to be followed;

— protective clothing to be worn;

— definition of personal responsibilities and actions to be taken in the event of accidents or unexpected conditions;

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— sampling and sample handling protocols to be followed;

— additional procedures to be adopted to avoid pollution of the environment, or harmful effects on people.

The site work should be supervised by an experienced and suitably qualified competent person, who, depending on the purpose of the investigation, may be a geotechnical engineer, geologist, hydrogeologist, chemist, or from a related discipline.

An environmental chemist should also be available (normally on a full-time basis) to provide professional advice on the identification of hazards, if the supervising person does not have relevant experience in this area.

Operatives of drilling and sampling equipment should be trained and experienced in the particular techniques and methods to be used (as well as in conventional site investigation works). Operatives not normally involved in site investigation activities, but assisting in the works (e.g. excavation plant operators) should be made fully aware of hazards and special procedures to be followed.

F.5.3 ***Sampling plan***

An appropriate sampling plan is a key aspect of any contaminated ground investigation. The plan should aim to:

— identify the types and concentrations of contaminants present;

— determine the lateral and vertical spread of contamination;

— identify the source(s) of contamination and the potential path/target combinations;

— provide sufficient data points to plan remediation measures (if necessary);

— identify the potential hazards and assess the risks.

A poorly designed sampling plan may fail to fully identify the contamination that is present. Conversely, oversampling and testing can be unnecessarily costly, and may still be ineffective. A staged investigation is usually most appropriate. In the case of water and gas sampling, it is also usually necessary to collect samples on several occasions over a period of time in order to obtain representative data on the conditions.

F.5.4 ***Sampling patterns***

The suitability of different sampling patterns for the investigation of contaminated land has been discussed by several authors ([246] to [250]). The two principal types of sampling pattern in use are:

a) judgmental sampling patterns;

b) regular sampling patterns.

Judgmental sampling takes account of the information gathered in the desk study and concentrates investigation points in areas where ground contamination is most likely to occur. Factors taken into account in selecting the investigation positions include:

— location of potential sources;

— previous site investigation data;

— topography;

— geology and hydrogeology;

— underground service runs, etc.

Judgmental sampling patterns are well suited to sites where sources of contamination are known, or suspected areas have been well defined. Such patterns are also useful where the purpose of the site investigation is to delineate the extent of contamination over particular parts of a site. Actual sampling positions may be a combination of targeted locations and localized grids. Because of the bias applied to the selection of investigation locations, care must be exercised in interpreting any statistical analysis of the analytical results.

Regular sampling patterns, such as rectilinear grids and herringbone patterns, are often used to provide coverage to areas when specific sources of contamination could not be identified in the desk study, or for validation of remedial works. Such patterns are normally easy to locate in the field and are well suited to sites with similar probability of contamination being present across their area. In cases where one part of the site is more likely to be contaminated than the remainder, the grid spacing is often reduced in that area.

An assessment of the probability of identifying localized hotspots of contamination with different sampling patterns was undertaken by Ferguson [250]. It was concluded that the herringbone pattern is the most effective and only very weakly influenced by the contaminant area orientation. A rectilinear grid has a similar efficiency to a herringbone grid for non-elongated shapes.

When deciding on the sampling pattern, each site should be considered individually and the distribution of sampling points selected accordingly. Whether judgmental or regular sampling patterns are used, the investigation should include several positions in unaffected areas of the site to assess background levels of contamination, and on the hydraulically up-gradient and down-gradient boundaries to assess changes in groundwater quality. In practice, a combination of judgmental and regular sampling is usually employed.

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F.5.5 ***Number of sampling positions***

An investigation of contaminated ground commonly uses several sampling techniques for different types of samples, and may involve several stages of investigation. Consequently, the application of rigid formulae relating the number of sampling positions solely to the size of the site is not recommended. The appropriate number of sampling positions depends on all of the following factors:

— degree of confidence required;

—area of site and extent of suspected contamination;

— nature and distribution of contamination;

— number of stages;

— proposed future land use;

— cost;

— availability of suitable equipment;

— timescale.

When possible the investigation should be undertaken in stages, as this offers greater flexibility and economy. Practical and time limitations often determine the number of stages. Initial stages of investigation may involve a preliminary soil–gas survey, the use of geophysical techniques or ground probing radar, the collection of representative surface samples of suspect materials, or the sampling of water from existing wells. Sometimes the initial sampling is combined with the reconnaissance visit, and the first stage of subsequent investigation involves a coarse grid of positions.

Ferguson [247] reports that 30 sampling positions have a 95 % probability of locating an individual area of contamination (hot-spot), covering 5 % of the plan area of the site.

F.5.6 ***Depth of samples***

Several samples should be taken from each sampling position. The number of samples should be selected according to site conditions, taking into account the likely sources of contamination, the proposed final levels and the prevailing ground conditions. Contamination usually occurs in the overlying ground and some investigators advocate sampling at specific depths, such as 0.15 m, 0.5 m, 1.0 m, 2.0 m and 3.0 m. This approach has the disadvantage that underlying or thin layers of contamination may not be sampled, or that an unnecessary number of samples are taken where low permeability natural materials are at shallow depth.

Samples should be taken in each different soil horizon; in particular from above and below, where there is any change from permeable to low permeability soils. Sampling should extend to the underlying uncontaminated strata. Where the soil structure is non-homogeneous composite samples can be taken. Samples of the surface deposits should be taken, as these pose an immediate threat to site workers, visitors or trespassers; a possible threat to surface water courses through run-off; and a threat to neighbours through wind action.

F.5.7 ***Sampling techniques***

The choice of sampling technique is site specific and influenced by the type and quality of sample to be retrieved, on-site access, the protection of water resources, geology, hydrogeology, and external factors such as cost and time. Techniques that minimize the exposure of the sample, cross-contamination, and arisings at the surface are favoured. Techniques in common use are as follows.

— Shallow trial pits (see **20.1**). Trial pits can be very useful for obtaining solid samples and are cost effective by comparison with other methods. On contaminated sites, however, there should be no physical entry into the pits, and careful control is needed of backfilling and covering to ensure materials are replaced in the sequence that they were removed. If necessary, clean cover should be provided to prevent dispersal of contaminants. Trial pits may also be used to obtain groundwater samples and are particularly useful in the identification and sampling of supernatant liquids.

— Continuous percussion sampling boreholes (see **22.9**). This type of drilling provides a complete stratigraphic section for sampling and visual inspection and minimizes cross-contamination. The technique is limited in depth to about 5 m to 8 m and is not suitable where obstructions may be present or the ground is water bearing. Also this method of drilling does not allow geotechnical borehole testing. Samples are normally class 2 or 3. Since the equipment is portable, percussion sampling is particularly suitable where there is restricted access.

— Light cable percussion boreholes (see **20.5**). This conventional method of geotechnical ground investigation may be used, although particular care should be taken in the casing-off of contaminated layers, the handling of spoil arisings, the use of water to advance the hole, and in backfilling.

— Mechanical augers (see **20.6**). Rotary drilling with a hollow stem flight auger is an acceptable method of forming a borehole in contaminated ground. The potential for cross-contamination is minimized by the continuous auger casing. The auger can generate heat, which might affect the composition of some contaminants, particularly if there are volatile components. Driven soil samples are obtained at selected depths from the base of the hole and geotechnical testing such as SPT may also be performed. Water and gas samples are normally taken from standpipes installed on completion of drilling.

— Rotary dry coring may be used to obtain soil samples at shallow depth, but this technique is not particularly suitable because high temperatures are usually generated, which can affect some contaminants. Rotary drilling with water flush is not suitable for contaminated ground.

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— Push-in groundwater and soil sampling probes (see **26.3**) are available from a number of specialist cone penetration test contractors, who have developed probes and sampling devices that may be used with conventional CPT equipment to provide high quality samples or measurements at discrete depths. Soil, groundwater and soil gas may all be sampled, and in-situ measurements made of pH, Redox potential, electrical conductivity and temperature. The principal advantages of these probes and sampling devices are the very low risks of cross-contamination and the absence of any arisings at the surface.

F.5.8 ***Sampling practices***

F.5.8.1 *General*

The essential requirement of the method adopted for sample collection should be to ensure that the samples are representative of the material under investigation. In particular, the sample should not be exposed to contamination before being analysed, and has to remain stable until analysed. In addition, sampling implements and containers should be constructed of materials appropriate to the analyses to be undertaken. This is particularly important for the determination of trace organics. Equipment should be thoroughly cleaned before re-use.

F.5.8.2 *Solid samples*

Solid samples should be taken using hand tools that are robust and preferably made of stainless steel, which is easy to clean and non-contaminating, or using driven steel sample tubes at the base of the borehole. Samples should normally be taken over a narrow depth range (say 100 mm to 150 mm) and should not be composited. It may sometimes be appropriate to take sub-samples across the stratum, in order to collect as representative a sample as possible. The method of sampling should be clearly recorded.

Sample sizes should be determined in consultation with the analyst and are subject to the analytical method requirements. In most cases a sample of around 1 kg is sufficient to enable a complete suite of analyses to be carried out. The sample containers should be sealable, watertight and made from a material that does not react with the sample. Polythene bags should not be used. For some samples such as those from gasworks sites, measures to ensure preservation are required when sulfides, cyanides or phenols are to be determined. The sample container should also be robust enough to avoid damage during transit. Any samples that are of a hazardous nature should be labelled clearly and suitably protected.

Particular care is necessary in the taking and handling of samples that may contain volatile organic compounds, as failure to observe best practices can result in the loss of most of the contaminants to atmosphere.

F.5.8.3 *Water samples*

Groundwater samples for chemical testing may be taken directly from a trial pit, from standpipe observation wells installed in completed boreholes (see **23.3**), or using discrete push-in samplers (see **F.5.7**). It is preferable to carry out repeat sampling and testing of groundwater over a period of several weeks or months to gain a good understanding of groundwater quality and its temporal variation. For this reason standpipe observation wells are often preferred. Such installations also allow the performance of in-situ permeability tests (see **23.4**). Sampling from standpipe observation wells is a three stage process as follows:

a) measurement of initial standing water level;

b) purging of the well so that it contains only water that has seeped in from the ground;

c) sampling.

Water (and any other liquids) seeping into an excavation should be sampled, preferably at the point of issue, by collection into a suitable clean sampling vessel. If samples are from the base of a trial pit or from an excavator bucket, solids ingress will probably be present. The sampling conditions are sometimes difficult to avoid and these should be noted in the trial pit log.

The most widely-used containers for water samples are polyethylene bottles, polypropylene bottles and borosilicate glass jars. Whatever material is used, the container should be opaque. The minimum volume of sample normally required is one litre. Containers should be thoroughly cleaned and dried before use. Special cleaning procedures may be required for certain parameters. The bottles should be rinsed three times and the fluid sampled before filling. The sample containers should be completely filled.

F.5.8.4 *Gas and vapour samples*

Samples for gas analysis in the laboratory may be obtained from boreholes (see Figure F.1), probes, structures or services. The sampling method chosen depends on the volume of gas available, its accessibility and the type of analysis required. Several types of airtight receptacle are available including gastight lockable syringes, bags with a non-stick coating and tubes constructed of plastic, glass, stainless steel, copper and aluminium. Due to their robust construction and ability to hold a large volume of gas in the compressed state, Gresham tubes are widely used. Stainless steel construction is preferred due to its relatively inert nature. All samples should be thoroughly purged with inert gas, i.e. nitrogen, and tested for leaks prior to use. Tubing through which samples are to be drawn should be made from butyl rubber. Silicon rubber tubing should be avoided, because it may adsorb some gases, particularly carbon dioxide.

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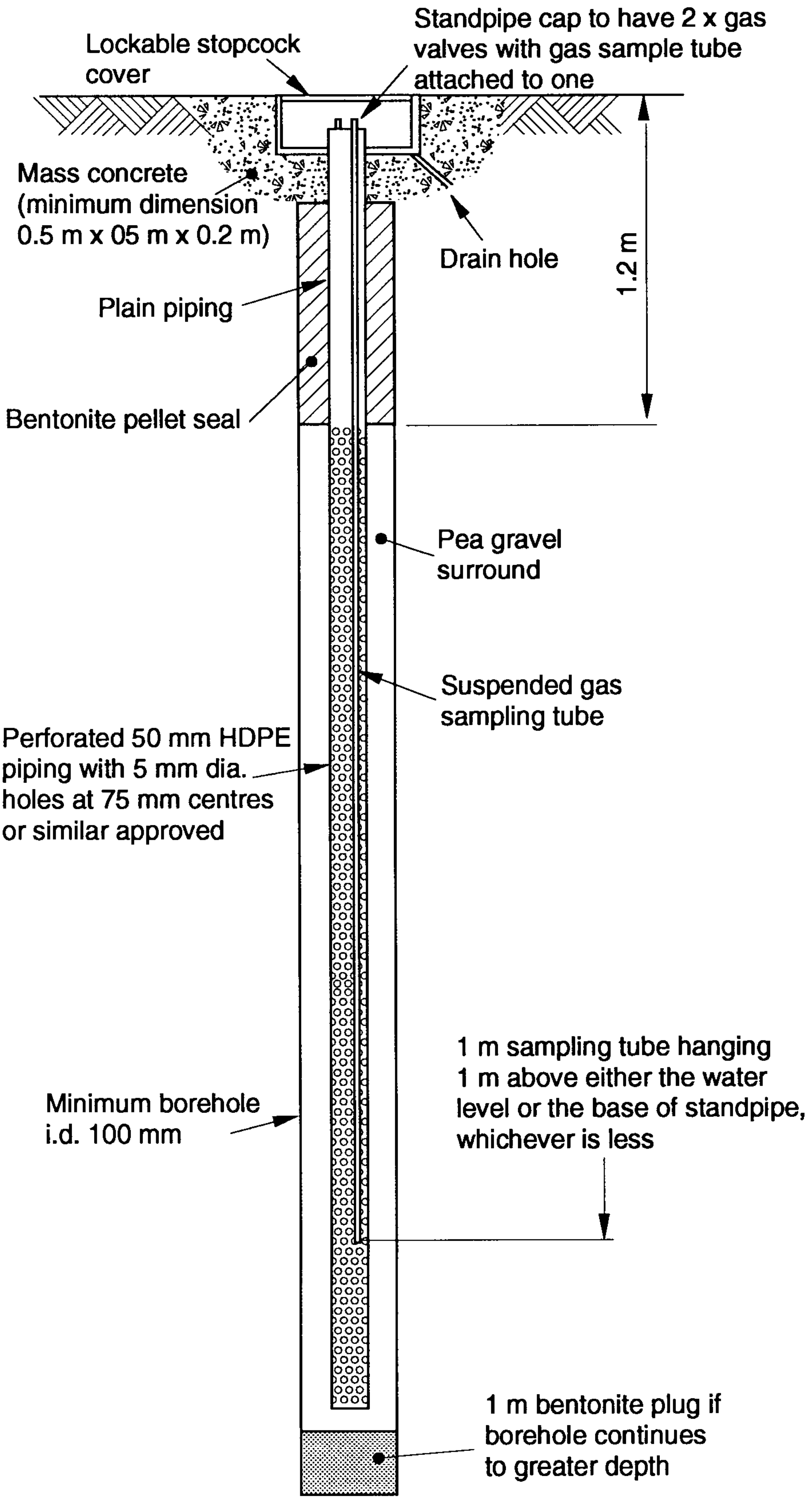

Figure F.1 — Gas sampling standpipe

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F.5.8.5 *Sample stability*

Some samples, especially liquids, may deteriorate rapidly after collection. Contaminants may be lost through volatilization, and may undergo chemical or biological changes, or there may be interaction with the container. To stop, or at least slow down, these reactions, a chemical preservative may be added to the sample, or the sample may be chilled or frozen. There is no universal preservative available, so when sampling it may be necessary to collect several samples at each point and, depending on the species to be determined, add a separate preservative to each. Some details of preservative and storage requirements are given in BS 6068. Some contaminants are not easily stabilized, i.e. volatile solvents, biological oxygen demand and chemical oxygen demand, and it is preferable to arrange for analysis to be undertaken immediately or soon after sampling. Bacteriological and water samples should be stored out of light and at a temperature between 2 °C and 4 °C. Analysis of bacteriological parameters should commence within eight hours of sampling.

F.5.9 ***On-site testing***

During some site investigations it may be advantageous to undertake testing on-site in order to:

a) ensure the health and safety of the site personnel by obtaining early indication of the presence of hazards, e.g. radioactivity, toxic vapours and acidic liquids, etc.;

b) allow the determination of contaminants that may change rapidly with time, i.e. pH, dissolved oxygen and sulfides;

c) speed up analysis of samples and enable a rapid analytical response.

A wide range of portable analytical equipment is available and some is limited in respect of specificity and accuracy. New equipment, however, is becoming available, which does not have these limitations. It is essential that all equipment has been properly checked and calibrated before being taken on-site and that the operators have had suitable training and experience.

On-site testing of gases and liquids is particularly appropriate as it lessens the chance of sample corruption and loss of determinant. It also reduces the need for preservatives, thereby reducing the complexity of sampling. For constituents such as pH, conductivity, dissolved oxygen and ammonia, the results given by electrochemical instruments on site are probably more reliable than samples that are transported to the laboratory and tested there.

F.6 Analysis of samples

F.6.1 ***General***

To obtain useful results from an investigation of contaminated ground, the analytical techniques employed need to be able to measure reliably the parameter of interest, have an appropriate detection limit and have a known response to possible interfering species. Detailed evaluation of available analytical techniques is not within the scope of this annex. Some of the critical aspects of sample analysis, however, include:

a) sample pre-treatment;

b) selection of analytical methods;

c) analytical quality assurance.

A comprehensive analysis of all the samples collected during an investigation can be time-consuming and might be uneconomic. On occasions, therefore, it may be advantageous to employ a phased analytical programme, which involves analysing a number of selected samples for a range of contaminants. The results of this initial analysis are used to design a more selective analytical programme on the remaining samples. An analysis can only be phased, though, if the samples do not deteriorate during storage.

F.6.2 ***Sample pre-treatment***

The majority of solid and liquid samples require some form of preparation prior to analysis. The method of preparation should be carefully chosen to avoid bias in the analytical results and to avoid the introduction of random errors. The elimination of bias is largely a matter of considering each step in the procedure in relation to the form of contaminant. Normally the procedures used for preparing samples for analysis depend on the stability of the contaminants. Drying causes changes, such as loss of volatile components, in the chemical and physical characteristics of a sample. The degree to which changes occur varies with temperature and time of drying. For this reason the procedure for drying samples should be standardized as far as possible. The methods chosen should be reported and the change in mass from the as-received sample noted.

A visual inspection of the sample should be made during the preparation stage and a note made of characteristics such as colour, odour and the presence of coloured, tarry or other non-soil material. When the sample is unstable and cannot be readily stabilized, it is important that preparation and analysis should be carried out as quickly as possible.

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F.6.3 ***Selection of analytical methods***

At present there are no officially recognized standard analytical methods for use during contaminated site investigations in the United Kingdom. For the majority of contaminants, a number of possible analytical methods is available and the most appropriate should be chosen by the analyst, in consultation with the site investigators. A range of analytical methods specifically developed for use on contaminated soils is available as part of the BS 7755 series. Analytical methods in the BS 6068 series are often appropriate for the analysis of water samples obtained in connection with the investigation of contaminated land, but should not be specified without reference to the analyst, because these may not be suitable for some concentrations and combinations of contaminants. The site investigators should ensure that the method selected meets the following criteria.

a) The results should be produced to the required accuracy and precision over the concentration range expected.

b) The effects of likely interferences and matrix effects should be minimized. This is particularly important for contaminated ground samples, which are often a heterogeneous mixture containing numerous pollutants in a complex matrix.

c) The technique should have its validity established by repeated testing.

d) The time taken for analysis should meet the time requirements of the site investigation.

e) The detection limit should be appropriate to the problem. This is particularly important when testing water samples.

The major consideration governing the choice of analytical method is the required accuracy of the results. This is influenced in turn by a number of factors, such as the existence of guidelines, by which the results are assessed, and data relating to contaminant levels to exposure and health effects. The relationship between the analytical results and the information requirements for the design of remedial works is also important. It should be noted, though, that the errors introduced by analysis are usually small compared to those associated with sampling and sub-sampling.

F.6.4 ***Analytical quality assurance***

At present the lack of standard methods of analysis for contaminated site investigations means that many different methods or variations of methods may be used. It is important that an analytical quality assurance programme is used to monitor the performance of the method employed and establish a comparison with any published criteria or an alternative laboratory. Several protocols have been specifically designed for chemical site investigations [195]. All the programmes involve verification work to ensure the technique is free from bias and to determine the method precision, repeatability and detection limit. The laboratory should perform routine quality control checks, for example by analysis of reference materials. An essential part of any analytical quality assurance system is a programme of inter-laboratory collaborative testing.

F.6.4.1 *Data interpretation and assessment*

Once a site investigation has been carried out, the significance of any contamination identified should be assessed in terms of the proposed use of the site. The questions that should be answered following investigations are set out in [232].

a) What hazards are likely to effect the proposed use of the site?

b) Which contaminants would give rise to these hazards?

c) Are these contaminants present and, if so, in what concentration and with what distribution?

d) Do any hazards exist? If so, how can these be removed or reduced?

e) Would the choice of a less sensitive land use be more effective in removing or reducing the hazards?

f) What remedial treatment is practicable and what monitoring is needed to enable the site to be used for the chosen purpose?

Several sets of guidelines to assist in the assessment of contaminant concentrations have been published including [35] [232] [234] [239] [240].

In addition to the above, information on background levels of contaminants may be used in the assessment of site investigation results. However, these comparisons can only be valid if the concepts of analysis are the same. Care should be taken when comparing data that are not produced by identical methods of extraction.

The ICRCL guidelines give tentative "trigger" values based on proposed land use (see references [231] to [239]). The trigger values are intended to assist in the selection of the most appropriate end use for a site and in deciding if any remedial treatment is required. It is important to recognize that the need for remedial treatment does not depend on the general level of contamination across the site as a whole, but on the maximum concentrations found in "hot spots" and the distribution of these areas of gross contamination. The purpose for which guidelines were developed should also be fully understood. It is essential that appropriate professional judgement is used in applying a set of guidelines to a particular site and circumstances.

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Additional recommended publications

NOTE The following publications are not referred to in the text.

HSE publications:

HSE. *Managing construction for health and safety.* Construction (Design and Management) Regulations 1994. Approved Code of Practice L54. London: HSE Books, 1995.

HSE. *Designing for health and safety in construction* London: HSE Books, 1995.

HSE. *A guide to managing health and safety in construction.* London: HSE Books, 1995.

BONE, S. Information on site safety for designers of smaller building projects. *HSE Contract Research Report 72*. London: HSE Books, 1995.

HSE publications are available by mail order from HSE Books, PO Box 1999, Sudbury, Suffolk CO10 6FS. Tel: 01787 881165. Fax: 01787 313995.

Construction Information Sheets:

No. 39 *The role of the client.*

No. 40 *The role of the planning supervisor.*

No. 41 *The role of the designer.*

No. 42 *The pre-tender stage health and safety plan.*

No. 43 *The health and safety plan during the construction phase.*

No. 44 *The health and safety file.*

These information sheets are available from local HSE area offices.

Other enquiries should be directed to HSE's Information Centre, Broad Lane, Sheffield S3 7HQ. Tel: 0114 2892345. Fax: 0114 2892333.

Publication:

JOYCE, R. *The CDM Regulations Explained.* London: Thomas Telford, 1995.

Safety documents produced by The Association of Geotechnical Specialists, 39 Upper Elmers Road, Beckenham, Kent BR3 3QY.

Guidelines for a Safety Manual for Investigation Sites

Essentially for management.

Safety Awareness on Investigation Sites.

This is a booklet for the use of all who work on investigation sites.

The AGS documents have drawn heavily upon:

The Construction Site Safety Notes.

Published by the Construction Industry Training Board (CITB) 1988-90;

Code of Safe Drilling Practice.

Published by the British Drilling Association (BDA), 1992.

Guidance Notes for the Safe Drilling of Landfills and Contaminated Land.

Published by the BDA, 1992.

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