GEOTECHNICAL MANUAL FOR SLOPES

GEOTECHNICAL ENGINEERING OFFICE
Civil Engineering and Development Department
The Government of the Hong Kong Special Administrative Region

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PREFACE TO FIFTH REPRINT

Since the publication of the second edition of this Manual in 1984, there have been significant advances in local practice with regard to slope investigation, design, construction and maintenance. As a result, more up-to-date guidance is given in subsequent GEO publications to supersede or supplement that given in this Manual (see the Addendum at the end of the Manual). For this up-to-date guidance, readers are recommended to refer to the original publications cited in the Addendum.

R.K.S. Chan
Head, Geotechnical Engineering Office
January 2011

FOREWORD

The Geotechnical Manual for Slopes provides guidance for the standards of practice that should be adopted for the design, construction and maintenance of slopes and site formation works in Hong Kong.

The first edition of the Manual was published in draft form in November 1979 to promote discussion and comment from those who used the Manual during its four-year life. This second edition has been prepared to rectify some weaknesses in the first edition and to up-date the content to current geotechnical engineering standards and practice. It takes account of improvements in practice that have occurred since 1979.

This second edition has been prepared entirely within the Geotechnical Control Office, after wide consultation throughout both the public and private sectors of the profession. It is based heavily upon the first edition, and only small changes have been made to many of the original sections. It is acknowledged, therefore, that this new edition owes a great deal to the Members of the Steering Group under whose guidance the first edition was produced on the basis of a draft prepared for the Geotechnical Control Office by Binnie & Partners (Hong Kong). Particular mention must be made of Professor P. Lumb, Professor S. Mackey, Mr R.O. Maher, Mr P.S. Molyneux, Mr D.J. Sweeney, Mr P.J. Thompson and Mr A.J. Vail who freely contributed a great deal of time and effort to this earlier endeavour.

Because this Manual is not meant to fulfil the role of a text book, no attempt has been made to provide a completely balanced treatment of the subject matter. Material that is of common geotechnical engineering knowledge has generally been omitted, the emphasis being placed upon the subject matter that is peculiar to Hong Kong or that, it is felt, requires fairly full explanation.

Within the confines of a single brief volume, it is difficult to deal adequately with such a broad subject area as that covered by the Manual. It is envisaged, therefore, that the Geotechnical Control Office will in time replace the Manual by a series of Geoguides, each of which would give fuller coverage to the material presently dealt with in each Chapter of this volume.

It is necessary to emphasize that the Manual is a guidance document and, as such, its recommendations are not mandatory. It is likely that situations will sometimes arise for which the Manual provides inadequate or inappropriate guidance, and the designer must use alternative methods of approach. There will also be improvements in design and construction practice that will supersede specific recommendations of this edition during its lifetime. For proper recognition to be made of advances in our knowledge, it is therefore essential that practitioners continue to provide the Geotechnical Control Office with suggestions for improvements to the Manual.

E.W. Brand
Principal Government Geotechnical Engineer
May 1984

CONTENTS

1. GEOLOGY OF HONG KONG

1.1 ROCKS

Two major rock types of igneous origin occur in Hong Kong : granitic and volcanic. Coarse grained granite rocks underlie Kowloon, the central part of Hong Kong Island, the lower levels of Victoria Peak, Sha Tin, Tsuen Wan and Castle Peak, although in places these are covered with colluvium, alluvium and recent marine deposits. Generally, finer-grained volcanic rocks underlie the middle and upper levels of Victoria Peak, the southern part of Hong Kong Island and much of the New Territories. In addition, sedimentary rocks, some of which are metamorphosed, cover a small area in the north of the Territory. On north Lantau and Tsing Yi, a major swarm of feldspar porphyry dykes has intruded both the granite and volcanic rocks. In many low-lying areas, superficial recent marine and estuarine deposits are common.

A detailed geological description of the rocks and rock formations of Hong Kong is given in a report by Allen & Stephens (1971), and the distributions of all deposits are shown on the accompanying 1:50 000 scale map (see Chapter 12 for details on how to obtain this map). It should be noted that this map will eventually be superseded by the larger-scale maps of the current Geotechnical Control Office geological remapping programme.

The granite suite of rocks are younger than the volcanic and have been intruded into the latter in the form of a batholith, or stock, with irregular outlines. The granites are generally widely jointed with typical joint spacings between 0.5 m and 2 m. Sheet joints are often present near the surface.

Although varying somewhat in composition and colour, the granitic rocks are often visually similar and are normally composed of feldspar, quartz, hornblende and biotite. Towards the contact with the volcanic rocks, the granites are sometimes finer-grained and the contact is often sharp and well-defined.

A granodioritic rock, which is part of the granitic suite, has been intruded into the volcanic rocks along a northeast to southwest line passing through Tai Po. Granodiorite has also been intruded into the granitic rocks of Stanley Peninsula.

The volcanic suite of rocks contains a mixture of lithologies including recrystallized and welded tuffs (ignimbrites), fine tuffs, coarse tuffs, trachyandesitic and rhyolitic lava flows, some of which are metamorphosed. The tuffs are the most common rock type. The rocks are generally closely jointed with typical joint spacings between 50 mm and 200 mm, however, in the coarser-grained varieties, joint spacings up to 3 m may be found.

Dolerite dykes of varying width, but generally not exceeding several metres, have been intruded into both the granitic and the volcanic rocks. The dykes in the volcanic rocks are sometimes sheared or faulted along one or both margins while this is rarely noted in the granitic rocks. The dykes tend to follow regional trends and generally dip very steeply.

Many of the rocks mentioned above have been faulted and sheared in places, the faults following regional trends. Fault zones, which are variable in width but certainly up to many metres, can be vertical or slightly inclined and may be weathered to considerable depths. Adjacent to faults, the rocks may be comminuted or very closely-jointed. The faults shown on the geological map of Hong Kong (Allen & Stephens, 1971) were identified by aerial photograph interpretation. Many more faults and dykes than are shown on the map are known to exist.

1.2 SOILS

In Hong Kong, materials on natural slopes that may be regarded and treated as soils from an engineering viewpoint are derived as products of the processes of both chemical and physical weathering. Soil may thus consist of insitu decomposed rock (chemical weathering) or colluvial detritus (physical weathering) which has moved down and blanketed the slopes in recent or ancient times during periods when the ground was excessively wet. The thickness of colluvial soils seldom exceeds 30 m, while for decomposed materials, soil depths in excess of 60 m are not uncommon.

The weathering of granites in Hong Kong is discussed by Ruxton & Berry (1957) who provide an excellent introduction to the formation and nature of these important but complex materials. The authors proposed a four-fold zonal classification of the full weathering profile. An illustration of this concept, in modified form, is shown in Figure 1.1. This scheme is useful for preliminary field description of weathered granites but will seldom be adequate for engineering purposes or for the description of other rock types. This is discussed further in Section 5.3.2.

Examples of various zones of weathered granitic and volcanic rocks based on the Ruxton & Berry (1957) system, are shown in Plates 1.1 to 1.12.

One feature of the weathering of granites that is of major importance for engineering projects is that rock quality does not always improve with depth; on occasions, hard rock at the ground surface may be underlain by thick zones of soil. Care must be taken that granite corestones are not wrongly interpreted as pinnacles of bedrock.

The soils derived from granitic rocks are usually sandy, while those derived from volcanic rocks tend to be silty whether they are formed by insitu decomposition or by colluvial processes.

Many of the natural soil slopes show signs of instability, which usually takes the form of shallow, narrow translational failures. The resulting debris may travel long distances, particularly where it becomes concentrated along drainage lines.

Soil that is formed by insitu weathering of parent bedrock can be recognized by the presence of relict joint planes or the original crystal fabric of the rock. The depth over which decomposed material changes to fresh rock is extremely variable and is related to the joint pattern, the spacing of the joints and the position of the water table. In the more closely-jointed rock, the change from soil to almost fresh rock can occur over a distance of only a few hundred millimetres. Joint planes in the fresh volcanic rocks are often coated with a clay mineral which frequently remains along the lines of relict joints in the soil. During the dry season, the water table is often located within the underlying rock and not within the soil. Where the rock is moderately decomposed (the feldspars have been altered to clay minerals but the rock cannot be crumbled in the hand), the joints are sometimes slightly open and provide a zone which is frequently more permeable than either the overlying soil or the underlying fresh rock.

Chemical weathering of both the volcanic and granite rocks produces halloysitic and kaolinitic clay minerals, the latter particularly during advanced stages of decomposition (Parham, 1969; Lumb & Lee, 1975). Lumb (1965) outlines the engineering properties of the residual soils of Hong Kong.

Colluvium is a heterogeneous material that consists of fresh or variably decomposed rock fragments (varying in size from a few millimetres to several metres) in a matrix of clayey silty soil. It results from the physical disintegration and transportation downslope of the parent bedrock.

Alluvium, generally consisting of variable proportions of subrounded gravel, sand, silt and clay, is found in the flatter valley bottoms. Extensive deposits are found in the valleys of Shek Kong, Yuen Long and Sheung Shui.

Considerable difficulty is sometimes found in distinguishing between colluvium, weathered rocks, alluvium and fill, and this problem is discussed in detail by Huntley & Randall (1981). Typical examples of fill and colluvium are given in Plates 1.13 and 1.14.

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2. SITE INVESTIGATION

2.1 INTRODUCTION

There are two principal components of a site investigation for construction associated with slopes. These are :

(a) surface studies, and
(b) subsurface investigation.

Much useful information can be obtained from surface studies and from an examination of the construction records and performance of existing structures in the vicinity of the site. The surface studies should form the first phase of a site investigation, and the subsurface work should be planned only after assessing the results.

The examination of surface features can be separated into two main stages :

(a) desk studies, and
(b) field studies.

Desk studies should be carried out before detailed field studies, but the engineer should visit the site during the initial phase of the investigation. The planning of the field studies should be based on the findings of the desk studies with emphasis being placed on the potential problem areas.

The general requirements for site investigation for construction associated with slopes are given in Table 2.1. This Table relates the height of the slope, the angle at which it stands and the risk category of the site to the information and specialist advice required.

The investigations needed to fulfill the requirements set out in Table 2.1 are given in Table 2.2. These Tables are intended to provide guidance only.

References that provide valuable information and guidance on many aspects covered in this chapter are :

(a) Code of Practice for Site Investigation, BS 5930 (British Standards Institution, 1981), and

(b) Report of the International Association of Engineering Geology Commission on Site Investigation (International Association of Engineering Geology, 1981).

2.2 DESK STUDIES

2.2.1 Existing Maps and Plans

Topographic maps and plans can be used to identify geomorphological forms and drainage patterns, and these can give an indication of the materials to be found on the site. Maps and plans covering the whole territory of Hong Kong are published by the Lands Department at scales of 1:100 000; 1:50 000 and 1:20 000. Details of these and larger-scale maps are given in Chapter 12.

Geological maps can be used to obtain information on materials and geological structures that affect the site. Geological maps are extremely useful as part of the site investigation but they are often based on isolated exposures and boreholes so that much of their detail is conjecture rather than fact. This should be borne in mind by the user, and interpretation should be left to a geotechnical engineer. A useful discussion on the use and interpretation of geological maps is given in the Manual of Applied Geology for Engineers produced by the Institution of Civil Engineers (1976). Geological Maps at a scale of 1:50 000 that were compiled by Allen & Stephens (1971) supersede previous geological surveys. These maps are also available separately. Detailed geological survey maps are in preparation by the Geotechnical Control Office.

2.2.2 Documents and Records

In the preliminary stages of an investigation, valuable geotechnical information about a site can often be found in records of the development of the area. This will include information on site formation, site investigation, well boring, piling, foundations and previous stability of slopes. Records are generally held by the Geotechnical Control Office or the appropriate Office of the Lands & Works group of Departments for public development investigations. Records are also held by consultant architects or engineers for both public and private developments, although records for old developments may be scanty or non-existent. Site investigation contractors and Hong Kong University may hold useful information. Old newspapers may also be of use in providing information about specific sites (Chapter 12).

Details of particular instabilities can sometimes be obtained from local residents. However, though the qualitative description of such incidents may be reasonably accurate, the details of timing are often less reliable.

At any early stage in the planning of site investigations, requests should be made to the various utility companies for details of services in the area of interest. These may have to be temporarily or permanently rerouted in the planned works.

2.2.3 Aerial Photograph Interpretation

Aerial and terrestrial black and white, colour and colour infrared photographs are useful for engineering applications. In addition to these, there are several other types of remote sensing (including satellite imagery and radar) now available for engineering use (Institution of Civil Engineers, 1976). Of these, only black and white aerial photography is readily available in Hong Kong. Complete black and white photographic coverage of Hong Kong at 1:20 000 to 1:25 000 scale is given in surveys flown in 1964, 1967 and annually since 1973. There is also coverage of urban areas at various scales, generally 1:4 000 to 1:8 000. Some earlier photographs dating back to 1924 are also available but coverage is incomplete.

Photo-mosaics of the 1964 aerial photographs are available at 1:25 000 scale for the whole Territory and at 1:6 000 for the urban areas. All aerial and some terrestrial photographs are available from the Lands Department.

In addition to topographic and geomorphological information, which can be identified by the stereoscopic examination of aerial photographs, some geological information and site history can also be inferred. One of the most important uses of aerial photograph interpretation (API) in Hong Kong is the interpretation of aerial photographs, acquired over a period of time, to build up a construction sequence for a particular site or development area. This is especially important when investigating surface hydrology or delineating areas of cut and fill (including reclamation). In some cases, construction techniques may also be visible on the photographs.

Many geological features can be identified using API, for example rock type, structural discontinuities, superficial deposits (colluvium and man-made), and slope instability. The geomorphology and drainage patterns usually relate to the geology. Figure 2.1 is a geological map and aerial photograph of part of Hong Kong Island which shows some major structural and lithological features. The major photolineations are traces of structural discontinuities such as faults, dykes or zones of intense jointing. These normally indicate zones of weaker rock associated with deep weathering and groundwater concentrations. Colluvial deposits generally occur as fans or as lenticular bodies along drainage lines. Colluvium consisting of detrital material that has continued to be weathered after its deposition can often be identified by API. Colluvial deposits are often associated with unusual groundwater conditions. Old slips, drainage patterns and sometimes the presence of seepage can be identified by stereoscopic examination of the terrain. Colour infrared photographs are often more effective than black and white for this purpose.

Another use of API is the systematic classification of land into 'terrain units' based on attributes such as slope angle, geological material, terrain type, erosion and instability. An example of systematic terrain evaluation can be seen in Plate 2.1. This plate illustrates some of the terrain units identified by API. Examples of its use in Hong Kong are discussed by Brand et al (1982).

Identification of geological and geomorphological features aids planning and interpretation of subsurface investigation and design works. Further information on the application of API to geological and geomorphological investigation can be found by reference to Van Zuidam & Van Zuidam-Concelado (1979) and Cooke & Doornkamp (1974).

2.3 FIELD STUDIES

2.3.1 Visual Examination

While the site and its immediate environs will be the subject of detailed studies, the examination of regional patterns of topography and drainage can provide valuable information about the probable subsurface structure of the site.

Unnaturally regular topographic forms often indicate the presence of a man-made slope. Such features may include truncated valleys and ridges and planar slopes; of the latter, fill slopes will usually be inclined at between 30° and 40° and cut slopes between 50° and 60°. Slumping and settlement are often indications of the presence of fills on site. Old developments on steep hillsides will have been formed by excavating on one side of the site and filling on the other, although the overall affect on the topography may not be sufficiently great to allow these features to be identified as man-made slopes by reference to the topography alone. Vegetation, particularly on old slopes, can make the visual identification of those which are man-made difficult, if not impossible.

A wall constructed during previous site formation works may either be a skin over a cut face or may retain fill. The batter of a wall may indicate which form the wall takes. The condition of any wall should be noted, particularly if it forms part of, or is affected by the proposed new works.

2.3.2 Engineering Geological Mapping

For a large or difficult site, it is advisable to carry out full-scale surface investigation and geological mapping. The methods used when plotting the data collected should be based on the Geological Society of London Working Party Report (1972) on the Preparation of Maps and Plans in Terms of Engineering Geology, modified as necessary for Hong Kong conditions. Further information on engineering geological mapping is given by International Association of Engineering Geology (1976) and Institution of Civil Engineers (1976).

Although soil and rock exposures should be indicated on the geological map under generic names, they should be fully described on field data sheets for subsequent correlation with the results of subsurface investigations. The methods used for describing Hong Kong soils and rocks are explained in Section 2.3.3.

2.3.3 Soil and Rock Description

Good soil and rock description is one of the most important features of a competently executed site investigation, and this should be carried out only by suitably qualified and experienced persons.

Numerous schemes are currently employed for description, and reference may be made to the following publications for some of the most recent : International Association of Engineering Geology (1981), International Society for Rock Mechanics (1978; 1981), and British Standards Institution (1981). Unfortunately, these vary not only in the terms used but also in the definition of the same term, and it is therefore vitally important that the system or terms used for the description of soil and rock be defined in all geotechnical reports.

Certain aspects of material description, with particular reference to weathered rock in Hong Kong, are discussed by Ruxton & Berry (1957) and Hencher & Martin (1982). Methods for distinguishing between superficial deposits and the classification of colluvium are discussed by Huntley & Randall (1981) and the Geotechnical Control Office (1982a) respectively.

It is important to recognise the difference between material descriptions applicable to small, essentially uniform samples and mass descriptions applicable to layers or zones that may contain heterogenous materials. In rock masses, component materials may differ considerably and should be described separately and fully.

Weathering is the general term describing the process of deterioration of a rock mass to a soil and comprises the effects of both chemical decomposition and mechanical disintegration. Many of the rocks of Hong Kong are affected by weathering to considerable depths, and examples are shown in Plates 1.1 to 1.6. Weathering typically proceeds through the rock mass inwards from discontinuities and so adjacent parts of a rock mass can show quite different degrees of deterioration. It is important to recognise the distinction between the description of the degree of weathering as it affects a small sample and that of a larger-scale layer or zone of the rock mass of the types illustrated in Figure 1.1.

The key to good description is to work systematically, and a list of headings under which description can be made is given below.

Material descriptions may include :

(a) colour,

(b) grain size and other textural features,

(c) degree of decomposition,

(d) degree of microfracturing (disintegration),

(e) strength,

(f) soil or rock name, and

(g) other characteristics, such as slakeability.

Mass descriptions may include :

(a) size, angularity, percentage and distribution of harder fragments,

(b) spacing and nature of discontinuities, and

(c) geological structure.

Various index tests can be employed as an aid to objective soil and rock descriptions. Amongst the most useful are the Schmidt hammer test, the point load test (Broch, 1978; Brook, 1980) and the hand penetrometer. Slakeability is also a simple, useful test. It is important to remember, however, that the results of such index tests may be dependent upon moisture content, the presence of microfracturing or jointing and the skill of the operator in both selecting a representative sampling point and conducting the test.

For a number of specific rocks, classifications of material weathering are available. One such classification that is applicable to most granitic and volcanic rocks in Hong Kong is given in Table 2.3. This classification is based on that of Moye (1955) and has been further developed on the basis of field measurements in Hong Kong (Hencher & Martin, 1982). Examples of these material grades for volcanic and granitic drillcore are shown in Plates 2.2 to 2.26, and the corresponding drillhole logs are given in Figures 2.2 to 2.7.

Classifications of material weathering have not been published for the other rocks of Hong Kong. For these other rocks, it is recommended that similar terms to those in Table 2.3 are used but with definitions modified to suit the particular site or project conditions.

2.3.4 Joint Surveys

Discontinuities such as joints usually control the engineering properties of a rock mass, particularly in cuttings. Where there are surface exposures of the rock, a joint survey should be carried out. The results obtained can be used to assess the risk of joint controlled instability.

The methods and equipment used to carry out a joint survey are discussed by the International Society for Rock Mechanics (1978), and methods to analyse the results are described by Hoek & Bray (1981). During analysis, care must be taken that rare but critical joints are not overlooked by the usual statistical methods of sorting data (Beattie & Lam, 1977; Brand et al, 1983). Further discussion, together with examples of joint survey record forms and a sample stereoplot, are given in Section 2.6.

A joint survey should only be carried out by an experienced geotechnical engineer who should visit the site, not only to carry out the survey, but also after the corresponding analyses are completed, to examine in detail the nature of those discontinuities that have been identified as critical. The slope or exposure should be examined again during construction for the presence of joint sets not identified in the survey which could lead to the development of instability.

2.3.5 Surface Drainage

During the surface investigation all stream courses, channels, nullahs, ditches, catchpits and culverts should be mapped, and details of size and condition plotted on the geotechnical site plan. This information will prove useful when assessing surface drainage characteristics of the existing site, and how these existing surface drainage measures will have to be modified or improved to accommodate the proposed development.

2.4 SUBSURFACE INVESTIGATION TECHNIQUES

2.4.1 Requirements

The requirements of a subsurface investigation are :

(a) to find the extent of the materials forming and affecting the site,

(b) to obtain information on the relevant properties of these materials, and

(c) to study the groundwater regime of the site.

These requirements should be considered when planning the position of and instrumentation in each hole. The investigation should be carried out under the full-time supervision of an experienced inspector, or other competent person, closely supervised by an engineering geologist or geotechnical engineer. For a major investigation, it may be necessary to have the full-time supervision of an engineering geologist or geotechnical engineer.

2.4.2 Trial Pits and Trenches

Trial pits and trenches can be either small hand-dug pits or wide trenches excavated mechanically. They permit the soil to be examined insitu and allow undisturbed block samples to be obtained. Trial pits should always be supported to prevent collapse and water should never be allowed to accumulate in them. When the exposed strata have been logged and the investigation is complete, the pits should be backfilled in properly compacted layers.

2.4.3 Hand-excavated Caissons

Hand-excavated caissons are common in the construction industry in Hong Kong where it is necessary to found piers on rock at depths up to and often in excess of 30 m. The method can be adapted directly for site investigation purposes. The advantages of caissons are the same as those of pits and trenches. The technique is severely limited, if not dangerous, below the water table in residual soils, unless used in conjunction with an effective dewatering technique.

2.4.4 Slope Surface Stripping

Useful information can be obtained from careful examination of slope surfaces. Many such slopes are protected with a thin layer of chunam (a lean soil cement mix), and before surfaces can be logged it is necessary to strip away this protective layer. The strip should be at least 500 mm wide, extending to the maximum height of the slope, and located to pass through areas of different surface features such as boulders protruding from the face, or where seepage can be seen. Light scaffolding is usually necessary to enable stripping and logging to be carried out.

2.4.5 Dynamic Probing

A hand probe, known as the GCO Probe, is being used increasingly. This is essentially a larger version of the Mackintosh probe. It has a 10 kg hammer which is dropped 300 mm. The probe point is 25 mm diameter and has an apex angle of 45°. Blow counts for each 100 mm penetration are recorded and plotted. The driving energy is such that probes have been carried out to 25 m in completely weathered granite. For investigation of an existing slope, a large number of probes are put down initially to obtain a general indication of the subsurface profile. This information is used to assist in the location of subsequent trial pits and drillholes. The probe can also be used for indicating the state of compaction of buried fill and the thickness of fill layers. A weight correction should be applied for depths greater than 5 m. A smaller version of the probe has been developed for use as an aid to the identification of surface materials. This mini GCO probe delivers the same impact energy per unit area as the full-size probe.

2.4.6 Boring

Boring methods are suitable only for soils and soft rocks. The applications and limitations of the various methods are as follows :

(1) Hand augering. Hand augering is suitable only for shallow holes in loose soils above the water table and for holes in trial pits. It is unsuitable for soils containing coarse gravel, cobbles or boulders.

(2) Jet boring (Jetting). The jet boring method uses the force of a high velocity stream of water to form the borehole. Borehole casing can be installed using this method. Acceptable quality samples cannot usually be obtained by this method. The high water pressures employed make jet boring unsuitable for use in places where an increase in degree of saturation, or in water pressure, could cause slope instability. The method is most suitable for granular soils, where it can be used to install wells or piezometers or to locate hard strata. Progress is stopped by hard strata, boulders or cobbles. Jet boring is seldom used in Hong Kong.

(3) Wash boring. Wash boring is generally carried out using a winch and tripod drilling rig. The ground at the bottom of the hole is broken up by the percussive action of a chisel bit and washed up to the surface by water that is pumped down the drillrods at low pressure. Generally, casing is driven down to support the sides of the boreholes. The fragments of soil brought to the surface by the wash-water are not representative of the character and consistency of the strata that are being penetrated. Standard penetration tests can be carried out, and open-tube samples or piston samples can be taken. The water pressures used are lower than those used for jet boring; therefore, infiltration of water into the slope will be less likely to cause instability. The method is suitable for sands, silts and clays and is seldom used in Hong Kong. However, in combination with rotary drilling it is widely used to advance cased holes in soft ground.

(4) Cable percussion boring. Cable percussion boring can be carried out using a winch and tripod or the winch of a rotary drilling rig. The hole is sunk by using a clay cutter in dry cohesive soils or a shell in granular strata or below the water table, and a chisel is used to penetrate hard strata or boulders. Casing is driven down to support the sides of the borehole below the water table and in soils liable to collapse. Disturbed samples brought to the surface by the clay cutter or shell are suitable for identification of the strata and for classification tests. Standard penetration tests can be carried out and open-tube or piston samples taken. The method is suitable for all soils and weathered rocks, but chiselling through boulders can be time-consuming and expensive.

(5) Rotary percussive boring. A pneumatically-powered drilling rig can be used to rapidly advance holes through any type of soil or rock. Water flush is used during drilling to clear the bottom of the hole and to keep the drill bit cool. Samples are not obtained, but strata changes can be identified by changes in the rate of penetration or the colour of the flushing water.

In conjunction with the Lutz electronic drilling parameter recording device, this method is an effective investigation technique being both rapid and inexpensive. Parameters such as percussion vibration and rate of penetration, which give a clear indication of materials penetrated, are recorded automatically against depth. Calibration of the results against drillholes or other direct methods of investigation is usually required.

Rotary percussive boring is generally used for grouting or blast holes, but it may be used to form holes for the installation of observation wells. It may also be used as a probe, without the Lutz parameter recorder, in conjunction with other control boreholes or drillholes to map out the base of a layer of fill or the surface of bedrock.

2.4.7 Drilling

By far the most commonly used investigation method in Hong Kong is rotary drilling for the recovery of cored samples. The holes are usually vertical but can be inclined up to 45° without major drilling difficulties. The drilling of long inclined and horizontal holes requires special consideration, particularly with regard to maintaining adequate directional control and minimising collapse problems. Inclined and horizontal boreholes up to 200 m long have been drilled in Hong Kong. McFeat-Smith (1982) outlines some of the practical problems involved. Borehole deviation is common, particularly in the vertical plane, because of the weight of drilling rods and variations in rock strength. This is minimised by use of drilling rig platforms with sufficient rigidity to satisfy the conditions specified in Section 2.4.8, and by the use of 3 m long core-barrels and continuous coring methods, as discussed by Coates et al (1977). For long orientated holes, regular monitoring of hole orientation, at about 20 m intervals, is recommended. Instrumentation available in Hong Kong for this purpose includes test tube/chemical, clockwork compass and borehole camera devices, of which only the latter two are suitable for holes orientated between the horizontal and 45° downwards.

The drilling rigs should preferably be of the hydraulic feed type, and the flushing medium can be water, air, air-foam or 'mud', although the latter, unless degradable, should not be used in holes in which permeability tests are to be carried out or instruments installed. The diameter of holes drilled will depend upon the tests, if any, which are to be carried out on the recovered core and the type of instruments to be installed in the holes. The sizes of the various casing, core-barrels and cores are shown in Table 2.4.

It is important to select the correct core-barrel, as the adoption of the wrong type can cause disturbance or damage to the cores. The types available and their limitations are as follows :

(a) A single-tube core-barrel rotates against the core which is not protected from the drilling fluid; core recovery is seldom satisfactory and it should not be used for site investigation.

(b) A double-tube core-barrel has an inner tube mounted on bearings so that it does not revolve with the drill string; it can normally be used in the fresh and the slightly to moderately weathered rocks.

(c) Triple-tube core-barrels may be used where other methods have been found ineffective and good core recovery is required. Triple-tube barrels have detachable liners within an inner barrel that partially protect the core from drilling fluid and from damage during extrusion and subsequent transit.

(d) Non-retractable triple-tube barrels are suitable for use in fresh to moderately weathered zones of the rock profile and some of the stronger highly weathered materials.

(e) Retractable barrels, where the inner barrel projects ahead of the bit when drilling through soft materials and retracts when the drilling pressure is increased in hard materials, are suitable for weaker highly weathered rocks and for all completely weathered rocks and residual soils.

When a hole is required only for the installation of instruments or insitu testing, non-coring methods may be adopted. This is most commonly done using a rotary drill, wash boring through soil, and double-tube coring through rock. Rock roller bits or rotary percussive techniques can be effectively used but are not common in Hong Kong. The hammer drill, which is driven by a heavy diesel hammer, is particularly useful in deposits such as fill or colluvium containing boulders and other obstructions.

Cobbles set in a matrix of softer material can cause difficulty with all drilling methods, as the cobbles tend to rotate with the drill bit, eroding the matrix and reducing the efficiency of the bit. However, this problem may be overcome by using air-foam as the flushing medium with a large diameter triple-tube core-barrel.

At present, the most commonly used core-barrel for sampling soils in Hong Kong is the Mazier triple-tube retractor barrel. The inner tube is a full circumference plastic liner encasing 73 mm diameter core which is compatible with standard laboratory triaxial testing apparatus. The Mazier core-barrel is usually used in conjunction with the double-tube Craelius T2-101 barrel when coring of rock is required. Details of the core-barrels and flushing mediums used, and their suitability to the various subsurface materials in Hong Kong are given by Brenner & Phillipson (1979), Forth & Platt-Higgins (1981) and Brand & Phillipson (1984).

In Hong Kong, there is an increasing awareness of the need for better quality core samples. This is leading to an improvement in specifications and to a general rise in standards, especially in the high quality core sampling of soft ground. Large diameter triple-tube barrels (100 mm) used in conjunction with air-foam as a flushing medium have proved successful in obtaining high quality core samples for identification and strength testing (Phillipson & Chipp, 1981).

2.4.8 Factors Influencing High Quality Drilling

As well as the optimum choice of core-barrel and flushing medium, there are many other aspects or factors relating to equipment and techniques that influence the success of high quality core sampling. The drilling rig should be of the hydraulic feed type and should have the capacity to drive a rotary tool tipped with diamonds or tungsten carbide in the sizes and to the depths required. The weight of each rig should be such that a force of about 12 kN can be applied to the drilling bit without movement of the rig. If the weight of the rig is insufficient for this purpose, the rig should be securely anchored down. The drill should be capable of providing stable drillstring rotation at speeds in the range 50 to 1250 rpm and have a minimum ram stroke length of 600 mm. A rigid rod, clearly graduated, should be permanently attached to and parallel with the hydraulic feed rams, in order to provide a means of measuring penetration and estimating penetration rates.

Casing should be used to stabilise caving ground. The size of casing and drillrods selected should be as specified for the size of core-barrel in use. All casing and drillrods should be straight and in good condition and should be thoroughly cleaned before use. Short lengths of drillrods should be available to enable continuous coring to be carried out with the potential for each core run to be one complete ram stroke. Short lengths of casing, not greater than the ram stroke length, should be available to enable casing to be advanced after each core run where necessary.

To provide consistency of measurements, and to ensure that the coring equipment is functioning as efficiently as possible, the following general drilling techniques should be adopted :

(a) Establish a permanent marker adjacent to the drillhole for referencing all depth measurements.

(b) Measure drillhole depth before and after each core run.

(c) Check drillhole depth after each core run by summing the total length of core barrel and drillrods.

(d) Ensure drillrods are always centralised within the chuck,

(e) Ensure drill position is maintained such that the drillrods remain central within the drillhole.

(f) When drilling in slightly to moderately decomposed rock, coring runs should normally be limited to lengths of 1.5 m. In more weathered material, restrict the drill run length to a maximum of one ram stroke. When less than 80% of the core is recovered from a run, steps should immediately be taken to increase the core recovery by such methods as reducing the length of run or changing the barrel. Explanations for core loss, if known, should be recorded.

(g) Immediately remove non-retractable core-barrel on change of materials from hard to soft and replace with retractable core-barrel.

(h) Remove drillrods and core-barrel from the drillhole carefully and without jarring.

(i) Wash out core-barrel and sediment tube after each core run.

(j) Casing, if needed, should be advanced by rotary action without surging and should not be advanced beyond the depth penetrated by the core-barrel.

(k) Ensure that all equipment and components are maintained in a good, clean, lubricated condition and otherwise maintained in accordance with manufacturer's instructions.

Ideally, the drilling crew should include a driller who has had previous experience of high quality drilling using the methods required. The crew should also include a member whose responsibilities include the taking and recording of all measurements required. The performance of the drilling crew in competently and conscientiously carrying out all stages of the drilling operations is all important to the success of high quality drilling.

2.4.9 Backfilling

All investigation holes must be backfilled as open holes allow ingress of water into the soil or rock with a consequent reduction in stability. Holes in rock should be backfilled with a grout such as cement-bentonite. In large diameter holes, fillers can be used with the grout. Holes in soil should normally be grouted, although under some circumstances they may be backfilled with well-tamped soil; loosely placed soil is not acceptable as the permeability of the hole will remain high. A few days after completion of backfilling, grouted holes should be checked for settlement of grout and refilled to the surface if necessary. Holes backfilled with soil may require long term maintenance.

2.4.10 Sampling

In defining the quality of the samples to be recovered during an investigation, the characteristics of the ground and the properties which are to be measured must be considered (Table 2.2) and appropriate sampling methods specified.

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

The sample quality classes required for various purposes are defined in Table 2.5 which also summarises the methods which can be used to recover samples of the required quality. Some soil types in Hong Kong, such as colluvium or fill, are difficult to sample, and in some cases the best samples which can be recovered are only of class 2 standard. Samples of classes 3, 4 and 5 are commonly regarded as disturbed samples.

(1) <u>Quality class 1 samples</u>. For fine-grained clayey soil, these can only be obtained using thin-walled samplers, preferably of the fixed-piston type, with an area ratio not exceeding 10%, inside clearance not exceeding 1% and length not exceeding eight sample diameters. (Area ratio and inside clearance are defined in Figure 2.8). Thin-walled samplers are seldom robust enough for Hong Kong's residual soils; therefore class 1 samples will not be obtained. But as the soils are not very sensitive, class 2 samples will give acceptable strength and compressibility parameters. Class 1 samples can, however, be taken in alluvial and marine deposits. For residual soils, class 1 samples can be obtained from block samples.

According to the trial drilling carried out on typical Hong Kong soil types to date, the highest quality rotary core samples can be obtained by large diameter triple-tube barrels (MLC) using air-foam flush. These are probably equivalent to class 1 samples.

(2) Quality class 2 samples. The second highest quality rotary core samples (class 2) are obtained by H or N size triple-tube (MLC) core-barrels or Mazier triple-tube barrels and T2-101 mm double-tube core barrels with water flush. This method is useful for recovering samples for geological logging and laboratory testing, particularly for insitu decomposed rock grades IV and V. These can only be obtained from soils exhibiting some cohesion.

Class 2 samples can also be recovered from fine-grained clayey soils by either thick or thin-walled samplers, but the end-area ratio should not exceed 25% and the inside clearance 2%. They should have some overdrive space and should have ports fitted with non-return values to permit escape of air and water during driving, and to create a vacuum above the sample during extraction. The minimum internal diameter of the sampler should be 38 mm, but preferably 76 mm, and should be driven by a down the hole hammer, or jarring link, rather than by a trip hammer at the top of the hole. Whenever the hole is below groundwater level, water balance (maintaining the water level in the hole just above groundwater level) should be used.

(3) Quality class 3 samples. Class 3 samples are suitable for fabric study. Samplers used for the recovery of class 3 samples should conform to the general requirements given for quality class 2 samples. The use of a trip hammer is acceptable, and water balance is required only in highly permeable soils. The liner samples obtained during standard penetration tests are classified as class 3 samples.

(4) Quality class 4 samples. These are badly disturbed samples in which the moisture content has been changed by the drilling or boring methods. Samples of this class may be taken from the spoil from holes and trial pits or from the sampler cutting shoes. Small disturbed samples should weigh at least 0.5 kg, while bulk samples should comprise at least 10 kg of material. If stored in rigid containers, the sample should fill the container, and if stored in flexible containers, as much air as possible should be excluded before sealing.

(5) Quality class 5 samples. These can be obtained from the flushings, by either air or water, from any borehole or drillhole.

All samples should be clearly marked both inside and outside the container with labels giving the following information, where relevant :

(a) name of contract,

(b) name or reference numbers of site,

(c) reference number, location and angle of hole,

(d) reference number of sample,

(e) date of sampling,

(f) brief description of sample,

(g) depth of top and bottom of the sample below ground level, and

(h) location and orientation of samples (from trial pits).

Sample containers should be free of air and watertight. The ends of undisturbed samples should be trimmed, the walls of the tube cleaned and dried with a cloth and the sample sealed with several thin coats of just molten wax. Microcrystalline wax does not shrink to the same extent as paraffin wax and therefore gives a better seal against the walls of the sample tube. As an alternative, 'O' ring seals can be used. Any space between the end of the sample and the end caps should be packed with sawdust, sand or other suitable material. Lids can be sealed using a waterproof tape or wax. Samples of soil and rock that are to go to a laboratory for testing should be carefully transferred as soon as possible. They should not be subjected to any sudden shocks and should be protected from the weather.

2.4.11 Core Handling and Storage

All stages of core handling including sealing, packing and transportation should be carried out with extreme care. The results of good drilling can very easily be lost without careful handling of core especially upon removal of core from the core barrel. If triple-tube barrels are being used, inner tubes should be hydraulically extruded. Extrusion of cores using air pressure is difficult to control and should be avoided. Breaking down the barrel, in particular the removal of the cutting shoe, should seldom require hammering as this will undoubtedly disturb the core. Dismantling should be done carefully with wrenches, preferably with the barrel firmly anchored in a tripod vice above ground level.

If a core-barrel has to be hammered to free wedged pieces, a leather mallet should be used. The core should be placed in the correct order in a corebox made for that size of core. Core boxes can be made of wood, galvanised metal or, preferably, of moulded plastic to take the split tubing. The size of core boxes used should be limited to that which can be conveniently handled.

Weathered core should be sleeved in polythene tubing or sheeting or aluminium foil and sealed before being placed in the core box. Spacer blocks showing the depth from which the core was recovered should be placed at the end of runs to prevent movement of the core in the box. If a sample is removed from the core box, it should be replaced by a wooden block of an appropriate length. Samples for testing should be selected as the core is extruded, removed and placed in airtight rigid containers.

Core boxes and each individual run of core if stored in plastic tubing, should be clearly labelled both inside and out with :

(a) name of contract,
(b) name or reference number of site,
(c) reference number, location and angle of hole,
(d) date and method of drilling, and
(e) depth of top and bottom of each run.

It is good practice to photograph all cores. This should be done as soon as is practicable after recovery of the core and before description, sampling or removal of specimens for testing.

Core boxes should be stored under cover and arranged so they can be easily located and removed for examination. Cores in split plastic tubes and polythene sleeves can be stored, as individual runs, on shelves made of corrugated sheeting.

2.4.12 Logging of Holes

Soils and rocks from investigation holes should be described as suggested in Section 2.3.3. Soil samples that are not required for testing can be extruded and split open to see the fabric. Fabric shows most clearly when the sample is partially airdried.

The driller's log should be taken into account when producing the final log of the hole, which should be prepared by a suitably experienced and qualified person. Examples of formats for trial pit, borehole and drillhole logs are discussed in Section 2.6.3.

For rock core, measurements of core recovery, rock quality designation and fracture frequency should be made. The positions of shear zones and other discontinuities should be noted, and the reduced levels of these features given on the logs. Other details such as orientation, infilling and roughness should also be noted. For projects were the nature of discontinuities is particularly important, it is worth considering the production of a detailed fracture log.

The following terms are used when describing cores :

(a) Total core recovery is defined as $\left(\frac{\text{core recovered}}{\text{length drilled}} \right) \times 100\%$, and any core in which recovery is less than 100% means that some material, generally the weakest, has been lost. Areas where losses have occurred should be located as precisely as possible and the relevant levels given.

(b) Solid core recovery is defined as the length of material which is recovered as solid core pieces at full diameter expressed as a percentage of the length of core run.

(c) Rock quality designation (RQD) is the length of core recovered in lengths greater than 100 mm expressed as a percentage of the length of core run and measured along the centre-line of the core. If all the core is recovered and is in lengths greater than 100 mm, then RQD% = 100%. Any fractures or deterioration of the core caused by drilling should be ignored.

(d) Fracture index is defined as the number of fractures per metre run measured over any arbitrary length, which a is generally taken as a core run. However, if there is a marked change in fracture frequency during a run, such as at a fault zone, the fracture index should be calculated for each part of the run separately. Fracture index may also be quoted by reference to the maximum, minimum and mean length of core pieces (Franklin et al, 1971). The definitions for core recovery and fracture indices are shown diagrammatically in Figure 2.9.

Methods of showing these indices on a log are given in Section 2.6 on records. Water level, drill water return, casing data and other information should also be given on the log.

2.4.13 Instrumentation

During the investigation stage of a project, instruments may be installed to measure pore pressure, stress or relative movement. The various types of instrument, including their applications, limitations and methods of installation, are described in Chapter 10. The provision of piezometers in all investigation holes, and the subsequent reading of water levels can yield valuable design information for only a small increase in the cost of the investigation.

Although installation of two or more piezometers in one hole is feasible, it is difficult to achieve successfully and is not recommended unless very careful supervision is provided.

2.5 FIELD TESTING TECHNIQUES

Field tests carried out during the site investigation stage can be used to assess strength, deformation properties and permeability of both soils and rocks. Details of many of the tests considered here are given by the United States Bureau of Reclamation (1974). Tests used for construction control are discussed in Chapter 9.

2.5.1 Standard Penetration Test

This test is most commonly used to give a rough relative measure of the density of granular soils. The procedure is described in BS 1377 (1975). The test and its interpretation have been reviewed recently by Nixon (1982). The results can be significantly affected by the testing technique, so while carrying out the test and interpreting the results the following points should be noted :

(a) The borehole casing should not be ahead of the borehole, and water balance should be maintained if carrying out the test below the water table.

(b) Large diameter rods (BW or equivalent) or smaller rods with rod supports should be used to reduce energy dissipation.

(c) An automatic trip hammer should be used to drive the sampler, as the accuracy of a monkey and slip winch is too dependent on the skill of the operator.

The N value is defined as the number of blows required to drive the standard split spoon sampler a distance of 300 mm. The sampler is initially driven 150 mm to penetrate the disturbed material at the bottom of the borehole before the test is carried out. The operator, having noted the number of blows required for each 75 mm advance of the seating, then notes the number of blows required for each 75 mm advance of the test drive.

It should be noted that the empirical relationships developed for transported soils between N value and foundation design indices, relative density and shear strength are not valid for weathered rocks. Corestones, for example, can give misleadingly high values that are unrepresentative of the mass.

In view of the present state of knowledge, SPT results in weathered rocks should be used only to give a crude indication of relative strength.

BS 1377 (1975) recommends that, where penetration of less than 300 mm is achieved for 50 blows, the test be halted and the penetration recorded. It is common practice in Hong Kong, however, to continue the test up to 300 mm penetration regardless of the number of blows required. This can damage equipment. Provided penetration is accurately recorded, approximate N values greater than 50 can be estimated from the penetration achieved for the first 50 blows.

2.5.2 Impression Packer Survey

An impression packer survey can be used for tracing the orientation and aperture of discontinuities. The device comprises a central supporting rod which passes through a section of pneumatically inflatable rubber packer material. Overlying the inflatable packer are two stainless steel shells that are backed by a resilient foam material wrapped with a thermoplastic film. When the packer is inflated, the stainless steel shells are forced onto the borehole wall and the thermoplastic film is forced into any voids or fissures in the borehole thereby deforming permanently (Hinds, 1974).

The survey is run by inserting the impression packer device into the drillhole at successively lower levels, expanding the packers and obtaining an impression of the drillhole wall onto thermoplastic film surrounding the packers. The survey is referenced to a known direction by means of a downhole compass that is chemically grouted into position when the impression is being taken.

2.5.3 Dutch Cone Test

The Dutch cone penetrometer comprises a 60° cone, 10 cm^2 in area, mounted on a sleeved rod. The cone is used to measure penetration resistance as it is pushed into the soil at a steady rate while the sleeve or friction mantle measures skin friction. Readings are usually taken at 200 mm intervals, although a continuous reading of both cone resistance and friction can be given by an electrical cone penetrating continuously. The procedure is described in BS 5930 (1981) and ASTM (1982e). The cone resistance can be used to calculate bearing capacity and density but the results are badly affected if the penetrometer impinges on particles larger than the cone. Therefore, the equipment is unsuitable for the weathered rocks of Hong Kong but is highly suitable for marine sediments.

2.5.4 Pressuremeter

The Menard pressuremeter can be used to obtain strength and deformation characteristics of soils and rocks (BS 5930, 1981). The equipment consists of a probe which, when placed in a borehole, can be inflated. The volume changes of the probe, the expansion of which is limited to that in the radial plane, can be measured by means of a surface volume meter to which the probe is connected. A pressure versus volume change graph can be plotted, and this is converted into a stress-strain curve. From the test results, a limit pressure, which reflects the ultimate bearing capacity, is determined. A deformation modulus may also be determined, from which a rapid estimation of settlement may be made.

Tests are normally carried out at one metre intervals in AX, BX or NX holes, and in granular soils a split casing is used to protect the probe from damage. If the seating area for the pressuremeter is oversized, or is the walls of the hole are not smooth, interpretation of results becomes difficult.

A self-boring pressuremeter for use in soils, known as the Camkometer, has been developed (Windle & Wroth, 1972), but at present it is not sufficiently robust for use in the weathered rocks of Hong Kong.

2.5.5 Plate Bearing Test

Plate bearing tests may be used to assess strength and deformation characteristics of soils and soft rocks and can be carried out in trial pits or large diameter boreholes. Standard test methods are described in BS 5930 (1981) and ASTM (1982a). The results of a plate bearing test can be badly affected by the presence of boulders immediately below the test area, and in any case the accuracy of the application of the results to the prediction of the behaviour of full-size structures is dependent upon the size of the plate used for the test. Usually a 300 mm diameter plate will be adequate, but a larger area may be required to test coarse materials. The load can be applied to the plate by kentledge or by jacking against a reaction beam. However, the equipment required is cumbersome and very difficult to use on steep slopes and is, therefore, likely to be used for foundation design in level areas only.

2.5.6 Vane Test

The vane is used to measure the undrained shear strength of soft to firm clays and silts. Standard test methods are described in BS 1377 (1975) and ASTM (1982b). Erratic results are obtained if the soil contains gravel or any other large particles, and in Hong Kong the use of the vane should be limited to the marine sediments.

2.5.7 Permeability Tests

A knowledge of the coefficient of permeability of a soil or rock mass is required for the design of subsurface drains and, when assessing slope stability, to estimate the depth of the wetting band resulting from rainfall. To determine where perched water tables may form, comparative permeabilities rather than absolute values are adequate.

Both the permeability of the intact soil or rock and that of the discontinuities contribute to the mass permeability of a material. The permeability of intact unweathered Hong Kong igneous rocks is negligible and flow is controlled solely by the discontinuities.

Intact residual soil and weathered rock are more permeable than unweathered rock and thus discontinuities in these materials exert less influence on the mass permeability. Laboratory tests (Chapter 3) can give accurate values of the permeability of the sample tested but, as these samples are usually of intact material that do not contain major discontinuities, they are unrepresentative of the soil or rock mass. The coefficient of permeability obtained for soils in the laboratory can be ten to one thousand times lower than that of the mass from which the sample tested was recovered. The laboratory determined values for the permeability of a rock sample usually bear no relationship to the mass permeability.

Carefully conducted field permeability tests carried out below the water table in boreholes give values of about the right order of magnitude. Above the water table, where the results have to be extrapolated to obtain an estimate of steady-state flow values, the accuracy is questionable. This is because the soil through which the water is flowing probably never becomes completely saturated. However, such tests may model more accurately the conditions which occur during infiltration.

In applying the permeabilities determined by either laboratory or field tests to the design of subsurface drainage works, or assessment of depth of wetting, it is normal to assume that the soil or rock mass is homogeneous. The presence of a thin impermeable layer, or a thin highly permeable discontinuity which did not appear in the permeability test area, can introduce serious errors into the calculations. To prevent this, recovered cores should be examined carefully for any such features, and if these are present, their position should be clearly indicated on the borehole log. The designs can then be suitably adjusted to take account of the effect of these features on the groundwater regime.

Lugeon values determined from packer tests, which are described in Section 2.5.9, are used for estimating probable grout takes and for assessing the consequent reduction in permeability. Variations of permeability within the rock mass can be found with the packer test, and under certain conditions absolute values of permeability can be obtained.

2.5.8 Permeability Tests in Soil

The coefficient of permeability (k), can be calculated from the results of rising-, falling- or constant-head tests carried out in boreholes or standpipe piezometers. The way in which the test is carried out can affect the natural permeability of the material being tested. In tests involving a flow of test water into the soil, fines from the water in the holes can be washed into the soil reducing the permeability. To minimise this effect, only clean water should be used. Less commonly, in tests where water flows into the borehole, fines can be removed from the soil and if the water level is reduced too far, piping can occur and the derived values of k will be too high.

When testing soils below the water table, the following formulae (Hvorslev, 1951) can be used to calculate the permeability, k :

(a) Rising- and falling-head tests :

$$k = \frac{A}{FT} \text{ or } k = \frac{A}{F(t_2 - t_1)} \log_e \frac{H_1}{H_2} \quad . \quad . \quad . \quad . \quad . \quad (2.1)$$

(b) Constant-head tests :

$$k = \frac{q}{FH_c} \quad . \quad . \quad . \quad . \quad . \quad . \quad (2.2)$$

where A = cross-sectional area of standpipe or casing. (If the hole is not vertical, the horizontal water area should be used).

F = shape factor, which depends on the conditions at the base of the hole. The factors and their applicability are defined in BS 5930 (1981).

H_1, H_2 = water heads above or below the standing groundwater level (H_0) at times t_1 and t_2 respectively.

q = rate of flow.

H_0 = water head above or below the standing groundwater level maintained during a constand-head test.

The definitions of these terms are shown in BS 5930 (1981), and specimen test sheets and calculations are given in Section 2.6.4. Constant-head tests are likely to give more accurate results than variable-head tests but variable tests are simpler to perform.

These are steady-state equations suitable for the calculation of permeability when the test is carried out below the water table. In Hong Kong, it is often necessary to measure permeability above the water table. In this case, the steady-state equations can only be used if the time over which the test is conducted becomes very long (i.e. approaches infinity). However, under these circumstances permeability can be assessed using the constant-head test in which the water in the borehole is maintained at a constant level, and the flow rate required to maintain this level is measured at different times. The flow rate (q) plotted against the reciprocal of t should give a straight line that can be extrapolated to find q when :

$$\frac{1}{\sqrt{t}} = 0 \text{ (i.e. } t = \text{infinity)}$$

The Hvorslev (1951) constant-head equation 2.2 can then be used to calculate k, the head (H) being measured from the centre of the piezometer filter zone or from the centre of the uncased borehole length under test.

2.5.9 Packer or Lugeon Test

The results of this test, used to determine the permeability of a rock mass, should, where the flow occurs in only a few fissures or joints, be quoted in Lugeon values. One Lugeon is defined as a water absorption of one litre per minute per metre of NX drillhole at a pressure of ten atmospheres (approximately 1 MPa) maintained for ten minutes. One Lugeon is approximately equal to a permeability of 1×10^{-7} m/sec.

The test is carried out in a drillhole using either a single or double packer which is inflated to seal off the length of drillhole to be tested. To minimise end effects, the test length generally should be at least ten hole diameters. Water is pumped into the test-section under pressure, and after allowing time for saturation of the ground, the steady flow-rate is recorded. The test is carried out at a series of pressures but should not exceed overburden pressure or hydraulic fracture may occur. Under some circumstances, vertical cracks in the soil or rock can develop at pressures much lower than the overburden pressure. This may be indicated by a discontinuity in the graph of flow against pressure that should be plotted for all tests.

Pearson & Money (1977) present improved techniques that make it possible to distinguish between test system faults, such as packer leakage and non-equilibrium effects due to the hydraulic properties of the rock mass. Careful calibration of the equipment to be used in a water pressure test is necessary to calculate pressure losses due to friction (Dick, 1975). Calibration must be carried out for each pressure testing arrangement (pump, packer, valves and bypass, pressure gauge and flow meter) with various lengths of drill rods and varying flow rates.

The q/H graph, plotted from the results of tests conducted at pressures considerably less than 1 MPa (often 50 to 500 kPa), has to be extrapolated to give the flow at 1 MPa. This extrapolation introduces errors arising from the differences in energy loss between laminar flow and the turbulent flow that occurs at higher test pressures. Low-head tests tend to over-estimate the Lugeon values. However, in Hong Kong there is often no alternative to using low pressures, and when this is done, the test range should be quoted with the Lugeon value. A change of hole diameter from NX has little effect on the Lugeon value.

If the rock jointing or discontinuity pattern is sufficiently close for the test section to be representative of the rock mass, a mass permeability can be calculated using the following formulae :

(a) If $L > 12r$:

$$k = \frac{q}{2\pi LH} \log e \frac{L}{r} \qquad \text{. . . . . . . (2.3)}$$

(b) If $10\,r > L > r$:

$$k = \frac{q}{2\pi LH} \sinh^{-1} \frac{L}{2r} \qquad \text{. . . . . . . (2.4)}$$

where k = permeability,

H = gradient of the flow versus pressure head graph,

L = length of test section, and

r = radius of hole.

2.5.10 Geophysical Methods

Geophysical survey methods are inferential. They rely on the assessment of differences in measurable properties to derive, by inference, the changes in the subsurface conditions that are of interest to the engineer. An experienced geophysicist is required to plan the survey and interpret the results.

All geophysical surveying should have borehole control to avoid errors in the interpretation of the results. The more borehole control there is available, the better the interpretation of the geophysical profile between the holes.

Seismic refraction has been used successfully to assess the depth of weathering over a large area in detail. It may not be very satisfactory because of core boulders and differential weathering on joints. The method is most successful when used on large sites to obtain profiles of soft soil underlàin by rock. Where it is used, its accuracy should be checked at an early stage of the survey and, if found to be inadequate, the method should be abandoned.

The seismic refraction method relies upon monitoring the earliest time of arrival of seismic waves, refracted at the interface between different strata, at a spread of geophones. These times are plotted against distance from the seismic source, and a series of straight lines of varying gradient are thus obtained. The gradients of the lines are used to determine the depth of each interface. Problems can arise when weak layers are overlain by stronger material or when layers exist that are thin relative to the overall depth being surveyed. Control boreholes can help to overcome some of these problems and a check should be made to establish whether a seismic method can be used.

In some cases, it may not be possible to overcome the noise or background vibrations that result from traffic flows and construction operations, but by using a signal enhancement seismograph and choosing working times carefully, an acceptable signal-to-noise ratio in urban areas may be obtained.

Resistivity methods and seismic reflection are sometimes used for engineering purposes but neither are suitable for foundation or slope investigations in Hong Kong. Other geophysical techniques, such as magnetic, electromagnetic, gravity, nuclear and thermal surveys, are mainly of value in mineral prospecting and have seldom found engineering application in Hong Kong. In their present stage of development, they should not be used for slope investigations. Further information on geophysical methods is given by the Institution of Civil Engineers (1976).

2.6 RECORDS

The keeping of good records during site investigation works is essential. This Section contains examples of proforma, logs and maps showing how information should be recorded. These are not intended as standard forms but as examples of good practice. These forms have been based on the three Geological Society Working Party Reports (1970, 1972, 1977) but have been varied where necessary for application to Hong Kong.

2.6.1 Rock Joint Surveys

When carrying out joint surveys (Section 2.3.4), data should be recorded on forms of the type shown in Figures 2.10 and 2.11. In addition to surveying dip angle and dip direction, field estimates of strength, joint spacing and, if applicable, grain size should be recorded using the legends printed on the form. The forms given in Figures 2.10 and 2.11 have been filled in to show how they should be used. The results of a joint survey are plotted on a stereoplot as described by Hoek & Bray (1981). A typical example is given in Figure 2.12.

2.6.2 Engineering Geology Maps

The information collected during the geotechnical mapping of an area can be plotted in map form. The method of presentation of information will depend upon the nature of the project and the choice of the engineering geologist or geotechnical engineer carrying out the survey. The information should be presented in such a way that the user can read the map with ease.

2.6.3 Trial Pit, Borehole and Drillhole Logs

A legend of symbols representing the soils and rocks of Hong Kong, and suitable for use on logs, is given in Figure 2.13. The symbols can be combined to illustrate mixed soil and rock types. Alternative symbols may be used provided the use is consistent and explained on a legend. Examples of trial pit and drillhole logs are given in Figures 2.14 to 2.18. They show various methods of recording the information collected during the site investigation. Some important points concerning logging are given in Table 2.6.

2.6.4 Permeability Tests

Field and calculation sheets for water absorption (packer) tests and rising- and falling-head permeability tests are shown in Figures 2.19 to 2.21.

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3. LABORATORY TESTING

3.1 INTRODUCTION

This chapter discusses the testing of soils and rocks in the laboratory. Where detailed descriptions of tests are given in other texts, these descriptions are not reproduced, and the discussion is limited to the problems peculiar to Hong Kong soils and rocks. Where suitable references are not readily available, tests are described in more detail.

It is important to ensure that laboratory tests are carried out on soils and rocks that are truly representative of the materials at the site. For this purpose, particular note should be taken of the description and classification of materials given in Section 2.3.3.

3.2 CLASSIFICATION TESTS ON SOILS

3.2.1 General

Classification tests, which include the determination of moisture content, liquid and plastic limits, specific gravity and particle size analysis, are so called because of their use in identifying a soil as belonging to a group that exhibits similar behaviour (United States Bureau of Reclamation, 1974). In the case of Hong Kong soils, liquid and plastic limits and specific gravity have limited application for soil classification, and more information can be obtained from the particle size analysis. Moisture content and specific gravity are also used in the calculation of other soil properties, such as compressibility, dry density and degree of saturation. Particle size analyses are required for the design of filters.

Special precautions, described under the individual tests, are required for soils containing certain minerals. In Hong Kong soils, the most common of these minerals is halloysite, but significant amounts are unlikely to occur.

3.2.2 Moisture Content

The standard method of moisture content determination is described in BS 1377 (1975)(test 1(A)). The subsidiary methods (tests 1(B) & 1(C)) are suitable only for site testing (see Chapter 9).

An alternative quick method for the measurement of moisture content is by means of a microwave oven. However, calibration against a moisture content test determined in accordance with test 1(A) will be necessary before this method can be used. Recalibration of the microwave oven must also be carried out at regular intervals.

Soils that contain halloysitic clays, gypsum or calcite tend to dehydrate or lose water of crystallisation if dried at the standard temperature of 105° to 110°C. If the presence of a significant amount of these minerals is suspected, the effect on the determination of moisture content can be assessed by drying at various temperatures.

In a humid atmosphere, oven-dried samples re-absorb water very easily.

It is therefore important that they are cooled in a desiccator before weighing.

3.2.3 Atterberg Limits

The Atterberg Limit tests are described in BS 1377 (1975). Two methods of determining the liquid limit of the soil fraction passing a 425 μm test sieve are described (tests 2(A) & 2(B)). The cone penetrometer method (test 2(A)) is preferable to the Casagrande method (test 2(B)) because it is easier to perform and less prone to operator error, but either method is acceptable. Very few correlation tests have been carried out for the two methods on Hong Kong soils. The one point method (test 2(C)) should not be used unless there is insufficient material available to carry out tests 2(A) or 2(B).

Soils containing significant proportions of halloysitic clays must be tested without previous drying and rewetting, because the results obtained from a dried sample differ from those obtained from the sample in its natural condition. Testing of samples without drying is preferable for all soils, and a test report must state if the sample was dried.

3.2.4 Specific Gravity

Test 6 of BS 1377 (1975) describes the standard method for determining specific gravity. The removal of air under a vacuum from an oven-dried sample is difficult when the soil contains silt- and clay-sized particles. Three variations to the standard method have been found to improve deairing :

(a) the test is carried out on a sample at natural moisture content instead of oven-dried,

(b) the soil and water mixture is boiled at atmospheric pressure and then under vacuum, or

(c) kerosene is used instead of distilled water (this method should not be combined with (a) or (b)).

When variation (a) is used, or when it is suspected that soil has been lost from the sample during deairing, the remaining soil sample must be carefully collected, oven-dried and accurately weighed at the end of the test.

3.2.5 Particle Size Distribution

The standard method of wet-sieving coarse-grained soils is described in test 7A of BS 1377 (1975). This test requires preparation of the sample by wet-sieving on a 63 μm BS test sieve to remove silt- and clay-sized particles, followed by dry-sieving of the remaining coarse material. Some soils have a large amount of clay-sized particles in the interstices of the large particles, and these need additional dispersion.

The method of dry-sieving (test 7(B)) is not recommended for Hong Kong soils because clay particles may adhere to larger-sized particles.

The pipette and hydrometer methods of particle size analysis for fine-grained soils are described in BS 1377 (1975) (tests 7(C) & 7(D)). Although in BS 1377 the pipette method is preferred, the hydrometer method (test 7(D)) is suitable for most Hong Kong soils.

3.3 CHEMICAL TESTS ON SOILS

3.3.1 Sulphate Content

Hong Kong soils contain negligible amounts of naturally occurring sulphate. Sulphate tests are therefore necessary only where pollution of groundwater from industrial effluent or other contaminants occurs, and where such pollution could result in sulphates attacking cement in either cement-stabilised soils or concrete. BS 1377 (1975) test 9 describes the determination of total sulphate content of soil, and test 10, the sulphate content of groundwater aqueous soil extracts. It should be noted that the Building Research Station Digests referred to in the Standard have now been superseded by BRE Digest 250 (Building Research Establishment, 1981), and reference should therefore be made to this more recent publication, particularly when determining water-soluble sulphate concentration.

Total sulphate contents of more than 0.2% by weight in soil and 300 ppm in groundwater are potentially aggressive (BRE Digest 250). It is important to note that sulphate content is subject to seasonal and other variations, and the results of the tests are applicable only for the particular time and conditions of sampling.

Water used for flushing during wash boring and rotary drilling may cause alteration in the chemical properties of groundwater. Sulphate tests on samples of groundwater taken when boring by such methods, may therefore give results that are not representative of the ground conditions.

3.3.2 Acidity

The acidity of soil and groundwater affects the rate of corrosion of metals and the deterioration of concrete.

The standard electrometric method of determining acidity using a pH meter is described in test 11A of BS 1377 (1975), and this can be applied either to the sampled soil in suspension in water or to samples of groundwater. The subsidiary calorimetric method of test 11B also gives acceptable results. The pH can alter if there is a delay between sampling and testing, so field measurement should be used.

The acidity of Hong Kong soils is negligible and, as in the case of sulphate tests, these tests are most likely to be carried out only on soils that are suspected of being contaminated.

3.4 COMPACTION TESTS

The relationship between dry density and moisture content for a soil used for fill is determined by the standard compaction test, as detailed in test 12 of BS 1377 (1975).

Many Hong Kong soils are susceptible to crushing during a compaction test. The extent to which this occurs can be checked by carrying out particle size distribution analyses on a sample before and after the test. The BS description of the test gives an alternative method for soils susceptible to crushing, and this should be followed where appropriate. Good practice would normally require the use of this method at all times.

3.5 PERMEABILITY TESTS

The permeability of a soil or rock sample can be measured by falling-head or constant-head tests. Details of the methods are given by Akroyd (1969).

Samples of undisturbed soil used in laboratory tests are usually small and tend to be intact samples rather than samples containing relict joints and other discontinuities. As a result, the measured values of permeability may be lower than the actual field values, sometimes by two or three orders of magnitude. The results obtained from field tests, if correctly conducted (Chapter 2), will generally be closer to the true permeability of the insitu material than the results of laboratory tests.

Groundwater flow through rock is generally only along discontinuities. Permeability tests in the laboratory are unfortunately usually carried out on intact rock, and the results bear no relation to the permeability of the rock mass. Field testing is the only method of obtaining a reasonable assessment of rock mass permeability (Chapter 2).

3.6 CONSOLIDATION TESTS

Consolidation tests are used to determine the compressibility and rate of consolidation of fine-grained soils, including fills, under applied load. The tests are usually carried out on undisturbed samples but recompacted samples of fill may be used.

The one-dimensional consolidation test on small specimens (BS 1377, 1975, test 17) is suitable only for fine-grained materials. Hong Kong soils which contain sands and gravels can be tested in a large oedometer or in a triaxial cell. The bulk modulus of the soil obtained from a triaxial consolidation test or from the consolidation stage of a triaxial test can be related satisfactorily to the one-dimensional coefficient of compressibility, m_v. The calculation of the coefficients of compressibility and consolidation from this information is described by Akroyd (1969) and Bishop & Henkel (1976).

3.7 MEASUREMENT OF SHEAR STRENGTH

3.7.1 General

Accurate determinations of representative shear strengths of the materials of a slope are essential to meaningful stability analyses. Although it is possible in some circumstances for satisfactory strength measurements to be made insitu, laboratory measurements of strength are by far the most common. However, the values of shear strength determined from laboratory tests are dependent upon many factors, particularly the quality of the test specimens, the size of the test specimens, and the method of testing.

The shear strength of a given soil is dependent on the degree of saturation, and this varies with time in the field. Because of the difficulties encountered in assessing test data from unsaturated specimens, it is recommended that laboratory test specimens should usually be saturated prior to shearing, in order to measure the minimum shear strengths. Unsaturated specimens should only be tested when it is possible to simulate in the laboratory the field saturation and loading conditions relevant to the design.

The shear strength of Hong Kong soils should be determined in terms of effective stresses. This can be achieved by carrying out drained shear tests, or by conducting undrained shear tests with accurate measurement of pore pressure at failure.

Shear strength envelopes for soils and rocks are generally not linear over a wide range of stress. Shear strength tests should therefore ideally be carried out to cover the range of stress relevant to the field design situation. For the shallow potential slip surfaces that exist in Hong Kong, however, the normal stresses are quite low, and laboratory shear tests carried out at these low stress levels present experimental problems. The results from triaxial tests conducted at low confining stresses are particularly prone to errors in interpretation.

Although the strength of a rock mass is to some extent determined by the strength of the intact material, the stability of rock slopes depends in most instances on the strength along discontinuities. In these circumstances, it is necessary for the shear strength along these discontinuities to be measured directly in the laboratory. Emphasis is therefore placed on this in Section 3.10.

Even with a soil, specimen orientation can be important when the sample contains discontinuties such as relict joints. In the laboratory, a soil specimen may fail on a plane which is not necessarily that along which failure would occur in the field. Where failure in the field could occur along discontinuities, account of this fact must be taken when orientating laboratory test specimens. In Hong Kong, relict joints in soils can contain silt or clay, and can therefore often be the weakest part of a soil mass.

3.7.2 Selection of Test Specimens

It is important that the tests are conducted on soil and rock specimens prepared from undisturbed samples that are as representative as possible of the insitu material. Block samples or good quality core samples should be used, and they should be obtained from the correct elevation in the field. Larger samples are more representative of the insitu material, and they generally result in less disturbance during sampling. They also facilitate the production of good quality test specimens.

Laboratory specimens used in shear tests should be large enough to minimise boundary effects. The minimum specimen dimension should usually be at least six times the size of the largest particle contained in the specimen. Triaxial test specimens should generally be at least 70 mm in diameter. For direct shear tests, 60 mm and 100 mm square specimens, 20 mm thick, are commonly used, but shear boxes of up to 300 mm square are available.

In the case of soils, the presence of gravel- or cobble-size particles often precludes the preparation of test specimens that are truly representative, and test specimens must then be made from the matrix material alone. This usually results in the measurement of the minimum strength of the overall material. It has been shown (Holtz & Gibbs, 1956; Holtz, 1960) that the mass strength increases with the proportion of large-size particles. On the other hand, the preparation of test specimens of very weak material often proves to be difficult, and the resulting test specimens tend to represent the stronger portions of the undisturbed samples.

3.7.3 Types of Shear Test

The triaxial test is the most commonly used method for shear strength measurements on Hong Kong soils. This permits control of the principal stresses applied to the specimen and of the drainage conditions, and accurate measurements of pore water pressure can be made on saturated specimens.

The direct shear test (shear box) is being increasingly used for shear strength measurements on soils in Hong Kong. This is a much simpler and more economical test than the triaxial test, but it permits only partial control over the drainage conditions, and pore pressures cannot be measured. In the lower stress range, however, the direct shear test is thought to have distinct advantages over the triaxial test.

The shear strength of discontinuities is determined by direct shear tests. Specimens of intact rock are most commonly tested by means of the unconfined compression test, although high pressure triaxial equipment is sometimes used where this is available.

3.8 TRIAXIAL TESTS ON SOILS

3.8.1 Test Procedure

The triaxial testing of soils is described in detail in the reference book by Bishop & Henkel (1976).

Shear strength envelopes can be determined for Hong Kong soils by means of :

(a) consolidated drained (CD) tests, or

(b) consolidated undrained (CU) tests with pore pressure measurement.

Undrained tests are generally preferred for routine testing, because they can be carried out more quickly than drained tests, and because useful information can be collected from the stress paths of the tests.

Single stage or multistage CU tests can be carried out. Multistage CD tests are not recommended because of the likelihood of overstraining specimens. Even for CU tests, significant errors in the assessment of shear strength will result if the strain to failure for each test stage is excessive.

Filter paper side drains should not be used in triaxial tests, because they can lead to errors in strength measurement and are generally unnecessary for the soils of Hong Kong. Membrane corrections must be made in the usual way.

Saturation by back pressure can only be obtained by applying a small effective stress to the specimen. Specimens that start with a very low degree of saturation can be difficult to saturate. In these cases, saturation can be carried out by first percolating deaired water under a small hydraulic gradient through the specimen until air stops bubbling from it. A back pressure can then be applied to complete the saturation.

The strain rate for drained tests with pore pressure monitoring should be such that the pore water pressure fluctuation is negligible, and in any case the fluctuation should be no greater than 5% of the effective confining pressure. For undrained tests, the rate should be selected so as to allow complete equalisation of pore water pressure throughout the specimen. It is desirable that the strain rate does not exceed 2% per hour.

For undrained tests, failure can be defined either as the maximum deviator stress or as the maximum obliquity (σ_1'/σ_3'). For fully-drained tests, these two criteria coincide.

3.8.2 Interpretation of Results

For ease of interpretation, it is recommended that the results of CU triaxial tests are plotted as p'-q stress paths (Figure 3.1), where $p' = (\sigma_1' + \sigma_3')/2$ and $q = (\sigma_1 - \sigma_3)/2$ (Lambe & Whitman, 1969). The shape of a stress path indicates the tendency for a specimen to compress or dilate during shear. The p'-q plots also enable the most sensible strength envelope to be drawn as the boundary to a family of stress paths.

For CD tests, the p'-q stress paths are of no significance. Actual volume changes during drained tests should be measured throughout the shear process.

Strength envelopes determined from triaxial tests will often not be linear, and they will sometimes exhibit an apparent break-point in the region of a definite ‘critical’ pressure. This is because the stress-strain behaviour of the material is dependent upon the confining pressure under which it is sheared. Specimens that are tested at low confining pressures in the triaxial test tend to dilate during shearing. At high confining pressures, specimens tend to compress. These different stress-strain behaviours are indicated clearly by the different shapes of the respective stress paths (Figure 3.1). In Hong Kong soils, the critical pressure can be considered to be analogous to the maximum past pressure for a sediment.

It is important to remember that, where a strength envelope is not linear, the portion of the envelope used for design purposes must be that for the correct design stress range.

In the interpretation of triaxial test data, especially in the low stress range, the following sources of error should be borne in mind :

(a) Test specimens tend to barrel at high strains, which leads to an over-estimation of the shear strength.

(b) The saturation process prior to shear can lead to specimen disturbance in the form of unintended volume change. Strong (dense) materials tend to swell during saturation, which frequently results in loss of strength. Very weak (loose) materials may occasionally compress, to give misleadingly high shear strengths.

Experimental evidence obtained on Hong Kong soils indicates that the triaxial test tends to 'even out' the measured strengths of materials to such an extent that it is not a very sensitive tool for discriminating between dense and loose materials. This is particularly true in the low stress range that is of interest in Hong Kong. In the majority of cases, triaxial tests on saturated specimens will underestimate the field shear strengths.

3.9 DIRECT SHEAR TESTS ON SOILS

3.9.1 Test Procedure

The direct shear test (shear box) is described in detail by Akroyd (1969), ASTM D3080-72 (1982d) and Head (1982).

The shear strength can be measured on any pre-determined plane in a soil mass by trimming specimens at the correct orientation.

Specimens for direct shear tests cannot be brought to full saturation, but a high degree of saturation can be achieved by immersing specimens in water for a sufficiently long period of time prior to testing. This soaking process probably represents most closely the conditions to which the material is subjected in the field under steady infiltration.

Where test specimens are to be soaked, these should be immersed in water in the shear box for a period determined on the basis of trials carried out on that particular type of material. It has been found that a period of twelve hours is usually adequate for decomposed granites (grades V and VI). Measurements of the volume change during soaking should be made to determine whether swelling or compression occurs.

The rate of horizontal displacement for the shear test should be slow enough to ensure that drained conditions prevail. A maximum rate of 0.08 mm per minute is considered appropriate for drained tests on 20 mm thick specimens of Hong Kong soils. Accurate measurements should be made throughout the test of the shear force, the relative displacement of the two halves of the box, and the vertical movement of the specimen top plate.

3.9.2 Interpretation of Results

For a number of test specimens of similar material, the strength envelope can be determined from the relationship between the measured shear strength and the applied normal stress. This relationship will often not be linear.

In the calculation of the shear stress and normal stress, corrections should be made for the change in area of the shear plane throughout the test.

3.10 STRENGTH TESTS ON ROCK JOINTS

The stability of rock slopes usually depends on the strength along discontinuities, and emphasis has therefore been placed on the testing of such discontinuities. Testing of intact rock can be important where shear surfaces are constrained by the geometry of the slope to pass through intact rock (see Section 3.11).

3.10.1 Direct Shear Equipment

Shear boxes of three main designs are used for determining the shear strength of rock discontinuities in Hong Kong and these are described below.

The first is a modified version of a laboratory soil shear box that can be used to determine shear strength parameters of rock discontinuities, and is suitable for the low normal stresses likely to be used for testing in Hong Kong. The standard shear box is capable of taking a normal load of up to 3 kN which, on a 60 mm square specimen, is approximately equivalent to the overburden pressure induced by 35 m of overlying rock. The soil shear box is modified by fixing two PTFE strips on either side of the top half of the box, and extending the goose neck to maintain correct alignment. This prevents rocking of the box when high points of the discontinuity are in contact, and it avoids the specimen catching on the edge of the box.

The second is a direct shear box, designed specifically by Golder & Associates for rock testing. It has been used successfully in Hong Kong and is described by Hencher & Richards (1982). One advantage of this box is that the normal stress is applied by a dead load system.

The third is the Robertson Research Ltd. shear box that was developed to provide index values rather than rigorous values for the shear strength of discontinuities in rocks (Ross-Brown & Walton, 1975; Hoek & Bray, 1981). At very low loads, the accuracy of the pressure gauges used to monitor shear and normal loads can be of the same magnitude as the imposed load, and this makes the value of the results obtained questionable. There are also major problems in ensuring that normal stress is kept constant particularly in the case of rough, dilating joints.

3.10.2 Specimen Preparation and Test Procedure

Rock shear testing is specified by the International Society for Rock Mechanics (1974) and more recently in the Canadian Pit Slope Manual (Gyenge & Herget, 1977) and it is recommended that these procedures be followed.

For interpretation in terms of field shear strength it is essential that comprehensive descriptions should be prepared both before and after failure. Surfaces should be photographed, preferably using low-angle lighting to emphasize relief, and annotated diagrams should be prepared to indicate the nature and mineralogy of surface coatings and post-shearing damage. Roughness profiles should be drawn to indicate the nature of the surfaces although these should not be used in calculations.

Tests should be fully documented including a complete record of normal and horizontal displacements. This is particularly important at the point when peak shear strength is measured.

Multistage tests, where a specimen is sheared under a series of increasing normal loads, can be carried out. This avoids the difficulty of obtaining mean shear strength parameters from tests on several individual specimens caused by differences in joint conditions (for example, degree of weathering or roughness).

Rock shear tests are never standard and always require interpretation. They should only be carried out and interpreted by experienced personnel.

3.10.3 Interpretation of Results

It is recommended that the method of interpretation of test data and hence the assessment of field shear strength follows that discussed by Hencher & Richards (1982) and Richards & Cowland (1982).

It is worth emphasizing that there are two distinct stages leading to the assessment of strength in the field. The first requires that the laboratory data be corrected for vertical movements of the upper block in order to provide a basic shear strength for essentially planar but naturally textured surfaces. All effects of major roughness are therefore taken out. The second stage involves the determination of the influence of roughness in the field. An assessment of field roughness is made on the basis of a roughness survey, the strength of the discontinuity surfaces and observations of damage in laboratory tests and is then added to the basic shear strength measured in the laboratory.

3.11 STRENGTH TESTS ON INTACT ROCK

Unconfined compression (uniaxial) and triaxial compression tests are the most suitable for standard testing, and only these two are discussed here. Information on these and other tests is given by Jaeger & Cook (1976).

3.11.1 Unconfined Compression Tests

Unconfined compression tests are carried out on rock to provide values for compressive strength, Young's modulus and Poisson's ratio. Tests should be carried out according to either the International Society for Rock Mechanics (1979) or the ASTM Standard (1982c). It should, however, be clearly stated to which standard the test is carried out. The conditions for testing are strict, particularly concerning the tolerance on preparation of the specimen.

Only specimens of intact rock should be tested. Results from samples which fail along discontinuities or other flaws should be disregarded.

3.11.2 Triaxial Compression Tests

The triaxial test used for rock is based on that used for soils. The principles are the same but the equipment is more rigid. The method for testing is discussed in detail by Gyenge & Herget (1977).

Light oil is used as the cell fluid for low pressure testing. The specimens to be tested must be smooth cylinders with end faces accurately machined flat and normal to the long axis.

Hoek triaxial cells are available for standard core sizes from E (21.5 mm) to N (60.8 mm) and are illustrated in Hoek & Bray (1981).

3.12 PRESENTATION OF TEST RESULTS

It is most important to present good records of both soil and rock tests. Appendix B of BS 1377 (1975) and Akroyd (1969) give typical data and calculation forms for the tests they describe.

A method of presenting the data from triaxial tests is shown in Figures 3.2, 3.3 and 3.4.

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4. GROUNDWATER

4.1 INTRODUCTION

The incidence of slope failure in Hong Kong during periods of intense rainfall indicates the degree to which rainfall and subsequent movement of groundwater affects slope stability. A knowledge of groundwater conditions is needed for the analysis and design of slopes. The groundwater regime is often the only natural parameter that can be economically changed to increase the stability of slopes. This chapter describes the methods whereby the influence of rainfall on groundwater can be assessed. Most of the chapter deals with groundwater flow in soils. Groundwater flow in rock is discussed in Section 4.4.5.

Water affects the stability of slopes in the following ways :

(a) by generating pore pressures, both positive and negative, which alter stress conditions,

(b) by changing the bulk density of the material forming the slope,

(c) by both internal and external erosion, and

(d) by changing the mineral constituents of the materials forming the slopes.

4.2 RUNOFF, INFILTRATION AND GROUNDWATER RECHARGE

4.2.1 Water Balance

Water, whether in the solid, liquid or gaseous (vapour) form is continually circulating by transforming its state and by its movement between land, sea and air. This continuous movement is the hydrological cycle (Figure 4.1).

In terms of the stability of Hong Kong slopes, the land-based portion of the hydrological cycle is of most interest, especially its operation in steeply-inclined catchment areas. Inflow to the system arrives as rainfall which can be extremely intense in Hong Kong. Outflow from the system can be as runoff, evapotranspiration, and subsurface underflow. Change of storage within the system is that part of the rainfall which becomes incorporated into the groundwater system as recharge. The above elements form the basis of the water balance equation, which is :

Rainfall = Evapotranspiration + Runoff + Subsurface underflow + Change in soil moisture + Change in groundwater storage

Changes in groundwater are critical to slope stability as it is these elements in the water balance equation that effectively alter the degree of saturation of the ground above the water table and the elevation of the water table itself.

Changes in groundwater storage may be positive or negative depending on the input and output elements of the water balance equation. Where, for example, rainfall is low and evapotranspiration exceeds inflow, a loss of soil moisture will occur. This eventually results in a soil moisture deficit. Soil moisture deficit is the quantity of water in millimetres required to restore the soil to field capacity where field capacity is the maximum amount of water held by the soil before free drainage occurs. When rainfall exceeds outflow from the system, the soil moisture deficit is reduced until the soil becomes fully saturated. Thereafter, changes in groundwater storage become positive and recharge to groundwater occurs.

4.2.2 Runoff

Runoff is that proportion of rainfall that flows from a catchment into streams, lakes or the sea. It consists of surface runoff and groundwater runoff, where groundwater runoff is derived from rainfall that infiltrates into soil down to the water table and then percolates into stream channels. The amount of runoff in any given catchment depends on a variety of factors such as the condition and nature of the soil and bedrock, the intensity and duration of rainfall, the slope angle, the surface cover and the antecedent conditions within the catchment. The amount or depth of runoff may be calculated by gauging the flow in streams which drain the catchment. The runoff coefficient or runoff percentage is defined as the proportion of rainfall that flows from a catchment as a percentage of the total depth of rainfall over the catchment area.

In order to quantify the amount of rainfall running off catchments and therefore what proportion of rainfall is available for infiltration, the Water Supplies Department has analysed storm events for a number of catchment areas (Government of Hong Kong, 1968). A Φ (Phi) index may be defined to give an indication of the amount of rainfall falling on a catchment area above which further rainfall would appear as runoff. For short duration storms, the value of the Φ index is affected by catchment retention, the filling of natural depressions, infiltration and antecedent rainfall. As the duration of the storm increases, the effects of infiltration predominate until the soils become fully-saturated.

The Water Supplies Department has determined the Φ index for 43 storms in eight natural catchments and have established that there is a limiting Φ index of about 3 mm/hour for long duration storms (Figure 4.2). Various curve-fitting techniques were applied to the data, and from these, the runoff corresponding to the limiting Φ index was found to lie between 50% and 70%. Therefore, assuming that infiltration takes place at the limiting rate and that 50% of rainfall runs off, a rainfall intensity equal to twice the saturated permeability is necessary to create these conditions.

Whilst for long duration rainfall the Φ index drops to approximately 3 mm/hour, the Φ index for short duration rainfall can be as high as 80 mm/hour (Figure 4.2). Rainfalls of this intensity are not uncommon in Hong Kong.

The proportion of rainfall infiltrating or running off the catchment depends on a number of variables as stated above; however, the surface cover of any particular area will also affect the proportion of runoff. Vegetated soil slopes will impede surface flow and runoff far more than a chunamed slope. Similarly, in paved urban areas, runoff to drainage channels would be an extremely high proportion of rainfall.

4.2.3 Infiltration

Infiltration is defined as the movement of water from the ground surface into the soil or rock via the pores or interstices of the ground mass (i.e. the absorption of water by the soil). Infiltration can be further divided into that part which contributes to the water content of the unsaturated zone, and that part which recharges the saturated groundwater system. Some recharge to the saturated groundwater system may be lost as groundwater runoff, whilst recharge to the unsaturated zone may be lost by transpiration or evaporation. When an unsaturated zone exists in a soil, it is said to have a soil moisture deficit. Recharge to this zone reduces the deficit until the soil becomes fully saturated, at which time the soil moisture deficit is equal to zero.

4.2.4 Types of Groundwater Flow

Water flows through soil or rock in various ways depending on the nature of the ground. Water-transmitting soil or rock units are called aquifers. Different types of aquifer demonstrate different modes of groundwater flow, such as intergranular, fissure and conduit flow.

Intergranular flow is groundwater flow between the individual component grains that make up a soil or rock. This type of flow most closely follows the Darcy concept of flow through an homogeneous isotropic medium of uniform grain size. In practice, however, most water-bearing strata exhibit intergranular or homogeneous flow and path-preferential flow through fissures or conduits within the stratum. Groundwater flow tends to be most rapid through fissures, pipes and joints within a soil/rock mass and can have a significant effect on groundwater levels and, hence, slope stability. These features are common in the weathered rocks of Hong Kong.

An aquifer, therefore, can be simply defined as a permeable water-bearing stratum that transmits water under normal head or hydraulic gradients. An aquiclude is a stratum that may contain pore water but is not permeable enough to transmit water even under considerable hydraulic head. The term aquitard is used to indicate a stratum that shows limited water-transmitting capabilities.

Groundwater in aquifers not only exhibits intergranular and path-preferential flow characteristics related to the granular or fissured structure of the aquifer unit, but also exhibits confined or unconfined flow characteristics. In the former, groundwater is normally confined at the top by an impermeable stratum (aquiclude). The aquifer unit is therefore fully saturated, and the piezometric (hydraulic) head is above, or coincident with, the lower boundary of the confining medium. In unconfined flow situations, groundwater does not fully occupy the potential aquifer, and a free water surface (water table) exists within the aquifer. In Hong Kong, aquifers are usually unconfined.

Water tables in aquifers can either be main water tables or perched water tables. The main water table in any aquifer is the surface of the zone of complete saturation where water flows laterally under gravity. In natural slopes the main water table generally follows the contours of the ground surface. Above the main water table, the soil or rock is not normally fully saturated. The main water table is usually recharged when infiltration occurs through the soil or rock above the phreatic surface, and usually discharges to streams or the sea.

Perched water tables exist above the main water table where a localised reduction in basal permeability occurs in conjunction with recharge from above. Perched water tables may be transient, developing rapidly in response to heavy rainfall and dissipating equally quickly, or permanent, responding to seasonal variations in rainfall level.

Groundwater flow in soil may be markedly different to that in rock. This is not necessarily a function of the differences in lithology between soil and rock, but rather the differences between the water-bearing interstices contained therein. Two adjacent rock or soil units with different lithologies, but with similar permeability characteristics, will act as one aquifer unit. Water flowing through an aquifer makes no differentiation between soil or rock units with the same hydraulic characteristics. Flow in soil is more usually of an intergranular type. However, fissure or conduit flow can occur in soils through fissures, pipes and drying cracks. Flow in rock can be intergranular but, in most cases, occurs through joints and other discontinuities which vary in width, form and orientation. Flow along fault shatter-zones can be significant. Measurement of groundwater characteristics in fissured-rock is difficult as conditions can vary rapidly with depth and areal extent (Section 4.4.5). Where piezometers intercept heavily-jointed rock, groundwater heads and velocity of flow may be large, whilst in adjacent unfissured rock, permeabilities may be extremely low. Joints in rock may become full of water during heavy rainfall, exerting high hydraulic pressures on the rock mass and adversely influencing stability.

4.3 PORE PRESSURES

Subsurface water may be divided into zones of positive and negative pore pressure. The dividing line is the groundwater table where the pressure is equal to atmospheric pressure. The groundwater table is generally determined from the level of water in an open standpipe. The changes in pore pressures in the two zones affect slope stability in different ways. A schematic diagram of changes in pore pressure in the two zones as a result of rainfall is shown in Figure 4.3.

4.3.1 Positive Pore Pressure

Atmospheric pressure is usually taken as the zero pressure datum and so a positive pore pressure zone exists below the water table. If there is no groundwater flow, the pore pressure is hydrostatic and the water level measured by a piezometer at any depth within the positive pore pressure zone will coincide with the water table. Pore pressures are no longer hydrostatic if there is any flow (steady or transient flow), and water levels measured by a piezometer in this zone will not coincide with the water table. Increases in positive pore pressure will result in reducing effective stresses and consequently will reduce available shear strength (see Chapter 5). Increases in positive pore pressure can be rapid after a period of heavy rain. The rate of increase, however, depends on many factors including the rate of rainfall, the nature of the ground surface, the catchment area and the soil permeability.

4.3.2 Negative Pore Pressure

The negative pore pressure zone is above the water table, and the pressure in this zone is less than atmospheric pressure. Water is retained in the soil mass above the water table by capillarity. Immediately above the water table, the soil is saturated up to the level of the saturation capillary head (i.e. the zone of continuous capillary saturation). In this zone, the negative pore pressure is hydrostatic in the no flow situation and variation with depth is linear. Above this zone the soil is partially saturated and the negative pore pressure does not necessarily vary linearly with depth, see Figure 4.3. The uppermost zone in contact with air is the hygroscopic zone where pore air is continuous and at atmospheric pressure.

The height of saturation capillary head in soils, which may govern the magnitude of the largest negative pore pressure, relates largely to grain size. With the exception of clays, the finer the soil particles, the larger the saturation capillary head and the higher the negative pore pressure. High negative pore pressures give rise to greater effective stress in the soil mass and hence an improvement in stability. High values of negative pore pressure have been measured in the laboratory in undisturbed samples of Hong Kong soil (Wong, 1970). Rainfall infiltration from the ground surface may rapidly reduce the magnitude of negative pore pressure. Pore pressure reductions to zero have been measured at depths exceeding ten metres in the Mid-levels area.

4.3.3 Measurement of Pore Pressure

Positive pore pressure can be measured in open standpipes and in piezometers installed in predrilled holes. An understanding of the subsoil conditions at a site is important when interpreting the observed water levels in standpipes. It is often difficult to assess whether the observed levels indicate the main water table or a perched water table. The observed water levels in many standpipes and piezometers, installed at different depths, can be used to assess the main water table and whether the groundwater is in a hydrostatic or flowing condition.

Field observation of seepages out of the face of a slope, during and after rainfall, provide useful information on the position of the water table. However, care must be taken to distinguish between seepage of groundwater and runoff from the slope surface.

Measurement of negative pore pressure is not as common as the measurement of positive pore pressure. The instruments used for measuring negative pore pressures are tensiometers. The use of tensiometers has proved to be successful in Hong Kong. Information on these and other pore pressure measuring devices can be found in Chapter 10.

The response of the groundwater regime to rainfall varies widely from slope to slope, ranging from no response to large and immediate responses. Installation of piezometers and subsequent regular monitoring, especially during and after heavy rainfall, is recommended for an accurate assessment of the maximum possible rises in pore pressure at a particular location.

4.3.4 Determination of Pore Pressure Distribution

Field measurement using piezometers is a direct means to determine the existing pore pressure distribution. In many cases, extensive instrumentation is not possible and another approach has to be used to assess overall pore pressure distribution from just a few observations. There are a number of techniques that may be used, including analytical, numerical, analogue and graphical methods.

Flownet construction by sketching is a commonly used graphical method for the assessment of steady flow conditions. It is a rather crude method that assumes homogeneity of the soil mass but it can give a useful picture of the overall situation. Flownet construction is explained in many text books, for example Todd (1980) and Cedergren (1977). This basic method is applicable to two-dimensional homogeneous soil conditions which are not the conditions usually found in Hong Kong soils. However, flownet construction is considered to be a useful first step in the assessment of pore pressure distribution. The flownet method is applicable only in the positive pore pressure zone and ignores the effect of infiltration into the water table from above.

Various physical analogue methods are available for determing flownets and pore pressure distributions in slopes. These are also described by Todd (1980). These methods were originally devised for a different kind of problem, and they therefore have different applications and limitations. Some of them can sensibly handle two-dimensional problems only, and some are limited to steady-state groundwater flow. For slopes, electrical analogue models and viscous fluid models have some application. The simplest electrical analogue method employs conducting paper, but this is limited to steady-state, two-dimensional flow. It is, however, extremely useful for obtaining initial design information in a quick and inexpensive manner. Analogue models are all site specific and, with the exception of the conducting paper technique, they could not be justified for general use.

Numerical techniques, using finite differences or finite elements, provide powerful means for obtaining pore water distributions in slopes, and they are the only means by which transient flow situations can be fully modelled. If adequate computer facilities and standard programmes are available, these can provide useful design information.

4.4 WATER LEVELS FOR DESIGN

4.4.1 General

The extent to which infiltration from rainfall reduces the stability of slopes is dependent on a number of factors including the original position of the water table, the intensity and duration of the rainfall, the antecedent rainfall within the groundwater catchment, the geology, porosity and degree of saturation, the topography and the land use.

It is recommended that slopes in Hong Kong are designed for the groundwater conditions that would result from a ten year return period rainfall. There are two principal ways in which the water levels due to that ten year event can be estimated. The first involves the analysis of piezometric data, and therefore depends on the availability of good field records of water levels before, during and after rainfall. The second involves the solution of an equation that describes the formation of a wetting band (zone of 100% saturation) and is dependent on the porosity, permeability and initial and final degrees of saturation of the soil forming the slope, and the percentage of the anticipated ten-year return period rainfall that will infiltrate into the ground directly above or behind the slope.

Various methods are available for the determination of water levels from piezometric records. These include the statistical correlation of groundwater response with rainfall, groundwater modelling of the aquifer system and the extrapolation of observed piezometric responses. Each of these methods is discussed by the Geotechnical Control Office (1982a).

Different aquifer systems demonstrate markedly different responses to rainfall depending on their storage characteristics. Aquifers may display rapid response to intense rainfall (storm response) or a gradual rise in water level during the wet season (seasonal response). The storm component rises and falls in response to each significant storm. The seasonal component is generally lowest at the beginning of the wet season reaching a peak at some time after the end of the wet season. Typical piezometer responses are shown in Figure 4.4.

Because of this variability in aquifer response, it is recommended that the groundwater conditions to be used in a stability analysis are based on water levels measured in the field by piezometers. For sites where there are no piezometric data available, the wetting band approach can be used to give a rough estimate of groundwater levels. These two methods of water level prediction are briefly outlined in Sections 4.4.2 and 4.4.3.

4.4.2 Extrapolation of Observed Piezometric Responses

The prediction of design water levels is most satisfactorily done by the extrapolation of observed piezometric responses. This method is described in detail in Geotechnical Control Office (1982a) to which reference should be made. Brief details only are given here.

In this method, the rise in water level resulting from a ten-year return period rainfall is determined and is added to a typical groundwater level occurring before such a ten-year event. The calculated groundwater level may then be used in slope stability calculations.

As shown in Figure 4.4, there are two ways in which piezometers respond to rainfall :

(a) storm response, and
(b) seasonal response.

For piezometers showing mainly storm response, a ten-year return period storm rise should be added to a typical groundwater level occurring before the storm. Different piezometers are sensitive to different lengths of storm event, this being a function of the delay before rises start, the rate of rise and the rate of decay. The designer should try to estimate a critical duration of storm for each piezometer.

For seasonally responding piezometers, a ten-year return period seasonal rise should be added to a typical groundwater level occurring at the beginning of the wet season.

Some piezometers exhibit both storm and seasonal response. In this case, either a ten-year return period storm rise would be added to a typical wet season water level or a typical storm rise would be added to a ten-year return period seasonal rise, whichever is the most appropriate.

Where permanent perched water tables occur, the procedure outlined above is followed. For transient perched water levels, the geological boundary on which the water is perched is taken as the base level.

It is clearly important to have complete records of piezometric responses for the successful prediction of water levels due to the ten-year return period rainfall (storm or seasonal). Water levels must have been recorded at sufficiently frequent intervals so that storm and seasonal responses can be distinguished. In situations where piezometers respond rapidly to storm events, monitoring would have to be continuous. The most satisfactory way of doing this is by a technician with a dipmeter. Where a technician cannot be provided piezometer buckets are a good compromise.

In order to develop an appropriate expression defining the relationship between rainfall and piezometric response in a particular aquifer, the response of that aquifer must have been monitored over a number of years. Clearly, the more complete the records of piezometer response, the more accurate the developed expression. Few sites, however, will have extensive piezometric data and so it is recommeded that water levels are monitored for as long as and as carefully as possible, ideally at least two wet seasons plus-the intervening dry period, before any prediction is made on the water levels likely to result from a ten-year return period rainfall.

The principal assumption in this method is that piezometric response is proportional to the total amount of rainfall. However, it is important to remember that there are a number of other rainfall and rainfall related factors that can influence piezometric levels. These include :

(a) storm duration,

(b) distribution of rainfall during the storm,

(c) antecedent groundwater conditions,

(d) additive effects of previous storms, and

(e) timing of the storm relative to any seasonal fluctuations.

4.4.3 Wetting Band Approach

If there are insufficient piezometer readings at the stability analysis stage of design, the wetting band approach may be used to give a rough estimate of groundwater levels. It should be appreciated that water levels predicted by this method will indeed be very approximate, because no account is taken of upslope infiltration and non-vertical flow within the slope and because the results are very dependent on the value of porosity, permeability, degree of saturation and runoff, which are usually unknown. It is essential, therefore, that piezometers are installed and monitored at an early stage to verify the design data provided by this approach.

This appraoch assumes that the wetting band descends vertically under the influence of gravity, even after the cessation of rain, until it reaches the main water table or until it reaches a zone of lower permeability, which may be very thin. Under the latter conditions, a perched water table will form above the zone of lower permeability, and pore pressure will become positive.

When the descending wetting band reaches the main water table, the surface of the main water table will rise with a consequent increase in pore pressure. The thickness of the perched water table or the rise in the main water table will be approximately equal to the thickness of the descending wetting band, reduced to allow for the extent, if any, to which the soil within the wetting band is assumed to be not fully saturated. The water level (perched or main) on which the descending wetting band is superimposed should be the highest level observed after an average wet season. In the absence of piezometric records, an estimate of this maximum level can be made from the highest seepage traces or stains observed on adjacent slopes or retaining structures.

It should be emphasised that this method is only applicable where the rise in groundwater level is due to rainfall infiltration and it takes no account of sloping ground, downslope flow and the differences in aquifer responses. If the intensity of rainfall is at least sufficient to cause infiltration at the limiting rate, the thickness of the wetting band will be dependent upon the duration of the storm.

The intensity adopted for the design storm should be equal to the saturated permeability of the soil forming the slope surface (Lumb, 1962) multiplied by two to take account of runoff. This factor of two, as derived in Section 4.2.2, is based on measurements for large catchments For smaller catchments, especially those that are steeply inclined, a runoff of 50% may not be appropriate (Nassif & Wilson, 1975). The duration of the storm of that average intensity, and of a selected return period, can then be obtained by reference to curves of probable maximum rainfall prepared by Bell & Chin (1968) and Peterson & Kwong (1981). Rainfall estimates are available for return periods up to one thousand years, for durations from instantaneous to 31 days. It should be noted that these curves are derived from observations made at the Royal Observatory in Tsim Sha Tsui. Recently, the Royal Observatory has advised that these curves may not apply at other locations, and they recommend that an independent analysis is carried out using data obtained from the nearest autographic raingauge to the site in question. Such an analysis would normally only be carried out for particularly high risk slopes. A map showing the distribution of raingauges in Hong Kong is given in Figure 8.3.

The wetting band thickness that forms as a result of rainfall is also inversely related to the difference between the initial and final degree of saturation of the soil mass. Thicker wetting bands are therefore more likely to occur after a series of heavy rainfall events, when the initial degree of saturation will be higher, than after dry spells.

The initial degree of saturation may be determined in the field. This will, however, depend upon the amount of rainfall that has occurred prior to sampling, and also upon the method adopted for obtaining the samples. Preferably these should be obtained from trial pits. They should be tested for moisture content immediately they are recovered, although evaporation during excavation and sampling could still be significant, and disturbance can affect the density measurements necessary for obtaining degree of saturation. An added complication is the variation in mositure content that will occur with depth. Degree of saturation should therefore be determined at frequent intervals over the first few metres of the soil profile.

The wetting band caused by heavy rainfall will extend downwards from the ground surface under the effects of gravity. The relationship between rainfall on unprotected slopes, infiltration and the depth of the wetting front is given by (Lumb, 1962, 1975) :

$$h = (Dt)^{0.5} + \frac{kt}{n(S_f - S_o)} \qquad \ldots\ldots\ldots \quad (4.1)$$

where h = depth of wetting front,

D = diffusion parameter,

k = coefficient of permeability,

n = porosity,

S_o = initial degree of saturation,

S_f = final degree of saturation, and

t = duration of rainfall

Lumb (1975) proposed that this expression be modified to a more approximate formula :

$$h = \frac{kt}{n(S_f - S_o)} \qquad \ldots\ldots\ldots \quad (4.2)$$

This simplification assumes that the diffusion term can be ignored after heavy and prolonged rainfall. The parameters required for estimating the thickness of the wetting band are rainfall intensity and duration, runoff, permeability and degree of saturation. Figure 4.5 gives examples of graphical solutions of this equation and shows the significant effects of variation in the value (S_f-S_o), the increase in degree of saturation. Figure 4.5 indicates that for lower values of permeability the thickness of the wetting band increases. The reason for this apparent paradox is that whilst the intensity of rainfall necessary to cause saturation is less for a soil of low permeability, the duration of rainfall of that intensity is longer for any given return period. This results in a greater depth of wetting band.

The value of permeability may be approximated by carrying out infiltration tests with a double ring driven into the soil at the bottom of a trial pit or caisson at shallow depth. Ideally, tests should be performed at successive depths to give a complete profile. Water is fed from graduated bottles to the exposed surface in the inner ring and to the annular space between the rings. The amount of water flowing out of the bottle is measured with time, using the calibrated graduations. The flow under steady conditions may be used to determine permeability. A suitable field infiltrometer is shown in Figure 4.6 and typical results are given in Figure 4.7.

The assumptions made when applying the approximate formula given above result in an extremely simplified model for infiltration. For example, if in the soil profile there exists a very thin band of material of permeability lower than the overlying and underlying materials, this will act as a throttle on infiltration. Above the wetting band, positive pore pressures will develop, while below the band full saturation is unlikely to be achieved. Also, if the surface permeability is lower than that of the underlying material, the surface acts as a throttle and no band of saturation can develop. Similar analytical difficulties are introduced as the degree of saturation changes throughout the soil profile.

4.4.4 Other Factors Affecting Groundwater Conditions

In localized areas, groundwater conditions can be significantly affected by factors other than rainfall. These other factors should be considered when interpreting piezometer data or estimating maximum groundwater levels for inclusion in stability analyses. It is known that leakage from water-bearing services occurs and, in addition, that cable ducts (e.g. for telephone or power cables) convey water from sources of leakage. This is of considerable importance in Hong Kong as such leakages cause a rise in the local groundwater level and hence affect the stability of slopes. Groundwater conditions may also be affected by pumping, dewatering, the construction of deep foundations and the formation of new cut slopes.

There are seven types of water bearing services normally found in Hong Kong :

(a) reservoirs,
(b) fresh water mains,
(c) flushing water mains,
(d) sewers,
(e) storm drains,
(f) catchwater channels, and
(g) water tunnels.

Leakages from these services can be detected by chemical analyses of groundwater samples (Geotechnical Control Office, 1982a). However, chemical analyses cannot establish the amount of leakage occurring or whether it significantly affects the stability of slopes.

The effects of groundwater pumping should also be carefully considered. Slope designs based upon groundwater levels determined in areas of active groundwater pumping can subsequently become inadequate if pumping is stopped and groundwater levels rise (e.g. in the situation of site redevelopment). When determining maximum groundwater levels for stability analysis, the likely drawdown effects of nearby wells should be assessed and it is recommended that the levels used in the analysis should be based on the assumption of no drawdown.

Groundwater levels can also be affected by construction, both by temporary dewatering and by the construction of sheet pile walls. In addition, a permanent foundation or underground basement may have a damming effect on the groundwater flow (Geotechnical Control Office, 1982a). Allowances should be made for these factors when interpreting piezometric data.

Surface protection also affects groundwater conditions. Uncracked chunam or sprayed concrete will generally protect the surface from direct infiltration but, at the same time, will reduce the rate of evaporation from the slope surface. The combined effects create a delay in changes of pore suction. The existing suction will reduce gradually during the wet season because of the surface protection, and it will build up slowly in dry spells because of the low rate of evaporation.

4.4.5 Groundwater in Rock

The permeability of rock masses is controlled by discontinuity geometry, which includes spacing, direction, discontinuity width and form, as well as the degree of infilling and the roughness of the discontinuity surfaces. These factors are all variable over small distances and make both observation and analysis difficult.

The normal technique for assessing groundwater levels in soils requires a piezometer to be installed in a short pocket of free-draining filter material. If such a technique is adopted for rocks, the pore pressures recorded will be those in the joints intercepted by the piezometer pocket and may not be the most critical for design.

Examination of cut faces may do little to help interpret the results of observations of standpipes as the rate of flow from relatively tight joints in which there are high hydraulic pressures may be so low that all seepage is removed by evaporation, giving the impression that the face is dry (Coates, 1970; Hoek & Bray, 1981). As the pressure response in the rock may be quite different to that in the overlying soil, it is often necessary to insert separate piezometers and standpipes in the soil and rock layers.

Despite these shortcomings, the standpipe system can yield useful information on the distribution of groundwater in a rock mass, although the interpretation in terms of fissure water pressure, as described above, is very difficult.

Open joints at shallow depths in rock slopes present a different problem. Very intense rainfall over a short period may cause the joint to fill up with water. For exposed rock faces, where open joints exist, it is reasonable to assume that these joints could become full of water to ground surface, and therefore necessary to analyse the stability of individual blocks accordingly (see Section 5.3.2 on predicted worst groundwater conditions).

4.5 SUBSURFACE DRAINAGE MEASURES

4.5.1 General

The factor of safety against failure on any potential slip surface which passes below the water table can be improved by subsurface drainage, thus reducing groundwater levels. This section discusses horizontal drains, drainage galleries, vertical drainage wells, cut-off and counterfort drains. All of these will lower water levels.

In any subsurface drainage system, monitoring is important. Piezometers should be installed to measure the pore pressure before construction, and should be read during and after construction to observe the effect of the drainage works. In the long term, piezometer readings can indicate impaired efficiency of the drainage system caused by siltation, deterioration of seals or breakdown of pumps.

The volume of water flowing from any drain is directly proportional to permeability and hydraulic gradient. When a drain is installed, the groundwater level will be lowered reducing the hydraulic gradient. The seepage will, therefore, progressively reduce from its initial value to a steady state value. This reduction in flow is not necessarily an indication of drain deterioration. In low permeability materials there may be no visible seepage from drains. In these cases, they will often be operating successfully as seepage evaporates as it occurs at the outlet.

In rock masses, the groundwater flow will generally be confined to joints, and therefore any drainage system must intersect these.

Where the drain is above the water table, consideration should be given to providing an impermeable lining so that the drains do not recharge the soil or rock. Where an unlined drain passes through partly saturated zones, seepage from the drain can decrease the pore suction in the vicinity, thereby decreasing stability.

In all gravity drains, the outlet to the collecting chambers should be lower than the invert of the drain to prevent water backing-up the drain.

4.5.2 Horizontal Drains

The main advantages of horizontal drains are that they are relatively quick and simple to install and they rely on gravity drainage. Holes are usually 75 mm to 100 mm diameter, drilled at gradients of 10% uphill and using a perforated or slotted casing. It is recommended that a filter be incorporated into the drain to prevent erosion or clogging. Before long drains are contemplated, the construction and subsequent maintenance must be considered. Choi (1977), Kenney et al (1977) and Prellwitz (1978) discuss methods for the design of horizontal drainage systems but it should be noted that most of these methods are based on ideal conditions (i.e. homogeneous, isotropic materials and steady-state drainage).

Piezometers should be used during construction to monitor the effect of the drains so that the spacing and number of drains can be adjusted to achieve the desired water table drawdown. Furthermore, the performance of the drainage system should be monitored to properly assess its continuing adequacy. Clogging must be regarded as a real possibility particularly with very long drains. Drains should be flushed, at regular intervals (possibly once a year) in order to avoid siltation.

Tong & Maher (1975) describe the design of horizontal drain installations that are grouted to obtain impermeable inverts.

4.5.3 Drainage Galleries

Drainage galleries (adits) are tunnels excavated behind a slope face for the purpose of lowering the groundwater table. Due to their high construction costs they are only used to control the water levels in large slopes where no viable alternative scheme exists.

The effective cross-sectional area of a drainage gallery is generally greater than the excavation itself due to the disturbance caused by construction. This effective cross-sectional area may be further increased, if required, by the provision of radial drains drilled from inside. In order to retain a degree of flexibility so that radial drains may be provided, it is necessary to consider this possibility from the outset and ensure that the size of the gallery provided is adequate to accommodate drilling equipment.

Where a gallery is constructed in highly-weathered rock, permanent support is required in the form of a reinforced concrete lining. In this case, the permanent lining should be surrounded with a properly designed drainage filter so that there is a good hydraulic connection with the material being drained. Weepholes then have to be provided through the lining in order to drain the filter.

A discussion on the optimum location and size of galleries for man-made slopes may be found in Sharp (1970) and Sharp et al (1972).

4.5.4 Vertical Wells

Vertical wells can be installed and drainage can begin before excavation takes place. Wherever possible, permanent wells should be drained under gravity through a system of horizontal drains or a drainage gallery. In some instances, wells will require pumping to maintain drawdown. It is important that standby generating equipment is available in case there is a power failure.

Because of the rise in the groundwater table between adjacent wells, the depth of hole required for overall slope dewatering may be greater than the slope height. Alternatively, several levels of wells and staged pumping facilities can be used for particularly high slopes or deep excavations.

To determine the position and spacing of wells around an excavation, the regional flow characteristics of the area should first be determined and the sources of recharge defined. The studies should consider the effect of well dewatering on the groundwater environment and, in extreme cases, some system of controlled recharge of the groundwater system outside the well line may be appropriate.

The design of wells, well screens and pumps is described by Johnson (1982).

4.5.5 Cut-off Drains

Cut-off drains are intended to intercept shallow groundwater flowing towards the slope. They are most effective if founded on an impermeable layer present at shallow depth.

An impermeable zone or membrane should be used as a cut-off downslope of the drain, and the top part of the trench should be backfilled with impermeable material. Runoff from the upper slopes should be collected in surface water channels designed to the requirements given in Chapter 8. The free-drainage material used to backfill the trench should be designed to conform with the filter criteria given in Section 4.6. The size of perforations in perforated pipes, slots in slotted pipe and width of joints in open-jointed pipe should be based upon the grain size of the filter material used as backfill to the trench (see Section 4.6.1 for details of the grading of the filter in relation to slot size).

Pipes that are perforated or slotted over only part of their circumference should be placed with the perforations or slots at the top of the sectional-area so that the amount of water flowing in the filter material below the pipe will be reduced. If porous pipes are used, they should be placed as tightly together as possible to prevent fine filter material being washed into the pipe. As a precaution, the stability of the slope should be checked assuming the drain forms a tension crack.

4.5.6 Counterfort Drains

The use of counterfort drains may be considered for soil or weathered rock slopes where only shallow drainage is required. A tentative basis for design is given by Hutchinson (1977). The trench should be backfilled with free-draining material that conforms to the criteria given in Section 4.6, and the trench should be sealed at the surface with impermeable material to prevent the drain collecting surface runoff. Porous, perforated or slotted pipes incorporated in the drains should conform to the criteria given in Section 4.5.5.

4.6 FILTERS

4.6.1 Granular Filters

A graded filter is a material that, when placed against a soil, is so graded that the migration of finer particles is prevented as water flows across the soil/filter interface. Filters should be more permeable than the protected soil and should be so graded that segregation does not occur during placing. Filters are used to prevent fine-grained particles from entering subsurface drains, the most erosion susceptible materials being coarse silts and fine sands.

Table 4.1 gives details of the normal filter design criteria applicable to soils in Hong Kong. The base soil particle size distribution used for the design of filters should be obtained as described in Chapter 3, but without the use of dispersants. Where the base soil contains a large percentage of gravel or larger-sized particles, the finer fraction should be used for the filter design.

While the United States Army Corps of Engineers (1953) criteria are applicable to silty soils, it has been found by experiment under relatively low heads that the U.S. Army Corps of Engineers concrete sand is suitable for use as a filter for all silts and finer soils. The grading of BS 882 zone 2 natural sand is very similar to that of the U.S. Army Corps of Engineers concrete sand. This is shown in Figure 4.9, which also shows the grading of a free-draining material that may be used in conjunction with this sand. The grading for BS 882 zone 2 sand produced from crushed stone can be unacceptable, as this standard permits 20% of the material to be finer than the 150 μm sieve (see Rule 6 in Table 4.1).

Where filter materials are used in conjunction with a coarser free-draining material, such as crushed rock, the grading of the coarser material should conform to the filter design criteria given in Table 4.1, to protect the filter from erosion. Where filters are to be provided between material of widely differing grain sizes, such as between decomposed volcanic material and rockfill, two or more filter zones may be required.

Problems also arise when designing filters to protect gap-graded soils. Where some particle sizes of a soil gradation are scarce or missing, the filter material should be designed on the basis of the finer soil particles only. This is also true for layered soils.

Calculations for the design of filter gradings are conveniently carried out in tabular form. An example of such a calculation is given in Table 4.2 and shows the grading envelope of a drainage material suitable for use with the U.S. Army Corps of Engineers concrete sand (Figure 4.9).

It should be noted that, although a filter grading envelope may be designed conforming to all of the filter design rules, this does not ensure that the filter produced also conforms to these rules. It is possible to produce a filter that falls entirely within the specified envelope but does not itself conform to Rule 7 in Table 4.1 (uniformity coefficient). This type of filter is a problem if its uniformity coefficient exceeds 20, as segregation is likely to occur. If such a filter is used, special attention must be paid to its placing in order to minimize such effects.

Material used for filters should consist of hard durable stone that when placed, should be well compacted.

The minimum width of a zone that is to act purely as a filter should be that which can be constructed without segregation of the material in the zone where the protected soil boundary is directly in contact with free-draining material. A minimum thickness of 450 mm is recommended for machine-placed material. Where a thickness less than 450 mm is to be placed, it may be necessary to hand-place the filter to ensure its integrity. Where the filter zone is also to act as a drain, the thickness of the zone should be sufficient to carry the maximum expected groundwater flow while allowing free drainage of the protected material.

Where perforated or slotted drains are provided within a filter material, the filter should be large enough not to enter the perforations, slots or open joints in the pipes. The ends of the pipes should also be closed to prevent the entry of the filter into the pipes (Spalding, 1970).

The criterion adopted by the United States Army Corps of Engineers (1955) for the grading of the filter in relation to slot or hole size is :

(a) For slots :

$$\frac{D85F}{\text{slot width}} > 1.2 \qquad . \ . \ . \ . \ . \ . \ . \ . \quad (4.3)$$

(b) For circular holes :

$$\frac{D85F}{\text{hole diameter}} > 1.0 \qquad . \ . \ . \ . \ . \ . \ . \ . \quad (4.4)$$

where D85F is the sieve size that allows 85% of the filter material to pass.

The criterion used by the United States Bureau of Reclamation (1973) for filter grain size in relation to pipe openings is :

$$\frac{\text{D85 of filter nearest pipe}}{\text{maximum opening of pipe drain}} \geqslant 2 \qquad . \ . \ . \ . \ . \quad (4.5)$$

The above equations have been found to represent a reasonable range over which satisfactory performance can be expected.

4.6.2 Filter Fabrics

As yet, there is little experience in Hong Kong with the long-term performance of filter fabrics for permanent drainage measures. As a result, it is recommended that filter fabrics are only used in negligible and low-risk situations, as defined in Table 5.2. The two exceptions to this recommendation are when the fabric is required to function for rare storm events only or when the performance of the fabric can be monitored for a period of at least five years and can be removed and replaced if it ceases to function properly.

The most common form of filter fabrics are made from synthetic materials and are classified into three types : Non-woven fabrics, woven fabrics and knitted fabrics. Problems that are associated with all fabrics are :

(a) deterioration on exposure to sun light and ultraviolet light,

(b) clogging,

(c) reduction in permeability due to compression,

(d) reaction with chemicals in soils causing decomposition,

(e) microbiological growth on fabrics, and

(f) the formation of planes of weakness in the works.

If these problems are overcome by attention to design, construction and quality control, filter fabrics can be very useful and can reduce construction time.

A summary of design criteria for filter fabrics is given by Rankilor (1981). It is important to remember that the fabric itself does not filter the soil directly. It simply supports the soil interface between two soils of different grain sizes, allowing an internal soil filter to build up.

Fabrics with an equivalent opening size less than 150 μm and thick non-woven fabrics may be prone to clogging. In some situations, fabrics can contribute to higher stability as they are able to sustain tensile stresses.

5. DESIGN OF SLOPES

5.1 INTRODUCTION

This chapter deals with the types of failure that occur in Hong Kong's soils and rocks, and describes suitable methods for their analysis. The design of cuttings and fill slopes and the treatment of unstable slopes are discussed. Construction methods, where they influence slope stability, are considered in Chapter 9.

It is common practice to define the stability of a slope in terms of a factor of safety obtained from a numerical stability analysis. It is well known, however, that any site investigation can quantify only approximately the factors that influence slope stability. This is particularly true in residual materials where material properties may be highly variable over a relatively small area and where pore pressures may vary rapidly during or after intense rainfall. Therefore, there are certain situations where an analytical approach may not yield satisfactory results. This is evidenced in Hong Kong by a number of slopes that have remained stable for many years even though numerical stability analyses indicate their factors of safety to be less than unity. In some circumstances, engineering judgement, based upon adequate knowledge and experience of Hong Kong conditions, is more important than the numerical results of 'classical' analysis. This applies particularly to natural slopes, but also to slopes and retaining structures that were constructed many years ago and that have remained completely stable through a number of severe rainstorms.

In determining the appropriate factor of safety against possible slope failure for a particular engineering or building project, an assessment should be made of all new, modified and existing slopes that could influence or be influenced by the new works. This will frequently require the assessment of slopes outside the immediate vicinity of the works. It is also important to consider the likely volume and distance of travel of debris resulting from a slope failure, and its effect on people and property.

For the stability assessment of existing slopes and for the design of new ones, it is recommended that stability analyses be carried out using one of the well known engineering methods in conjunction with quantified material properties and predicted pore pressure distributions. In some circumstances, a non-analytical approach to stability assessment might be thought to be more appropriate, but this should be applied only with the greatest caution, after a thorough examination of the slope in question, and with a knowledge of the slope's performance over a considerable time. Ideally, the analytical and non-analytical approaches should both be applied in all cases of stability assessment.

The rest of this chapter is concerned with the classical approach to the stability analysis of slopes in Hong Kong.

5.2 FACTORS OF SAFETY

A widely accepted definition of factor of safety against slope failure is the ratio of average available shear strength of the soil along the critical slip surface to that required to maintain equilibrium (Bishop, 1955).

5.2.1 New Slopes

An appropriate design factor of safety against the failure of a slope depends on the extent to which that failure could cause loss of life or economic loss. Table 5.1 shows recommended factors of safety against loss of life and against economic loss for new slopes. These factors of safety are for groundwater conditions resulting from a ten-year return period rainfall. There are three risk categories in each case (negligible, low and high). The risk-to-life category reflects the likelihood of loss of life in the event of failure. The economic risk category reflects the likely magnitude of economic loss in the event of failure. Typical examples of slope failure situations for each risk category are given in Tables 5.2 and 5.3.

It should be emphasised that the factors of safety against economic loss and the typical examples of slope failures in each economic risk category are for guidance only. These generalised statements do not cover every slope failure situation. It is essential that the designer chooses for himself an acceptable balance between the potential economic loss in the event of a failure and the increased costs of construction required to achieve a higher factor of safety.

Failures in the high risk-to-life category cannot be tolerated even in the event of rare groundwater conditions. In addition to a factor of safety of 1.4 for a ten-year return period rainfall, a slope in the high risk-to-life category should have a factor of safety of 1.1 for the predicted worst groundwater conditions. The designer should determine these conditions bearing in mind the factors outlined in Section 5.3.2.

In any borrow area or site formation project, the factors of safety adopted in the design of slopes should be in accordance with the future use of the area, with allowance made in the design for surcharge loadings expected from subsequent development. Where the future land-use cannot be determined, it should be assumed to be residential.

5.2.2 Existing Slopes

When analysing an existing slope to determine the extent of any remedial or preventive works required, the performance history of that slope can be of considerable assistance to the designer. There is, for example, an opportunity to examine the geology of the slope more closely than for an undeveloped site, and to obtain more realistic information on groundwater. The designer is therefore able to adopt with confidence factors of safety for proposed remedial or preventive works that are lower than those specified in Section 5.2.1 for new works. As long as rigorous geological and geotechnical investigations are conducted (which include a thorough examination of slope maintenance history, groundwater records, rainfall records and any slope monitoring records), the factors of safety given in Table 5.4 may be used for the design of remedial or preventive works, provided that the loading conditions, the basic form of the modified slope and the groundwater regime remain substantially the same as those of the existing slope. There will often be instances, however, where particular circumstances will lead the designer to adopt, for remedial and preventive works, the standards specified for new works (Section 5.2.1).

For the design of remedial or preventive works to a slope, it may be assumed that the existing slope has a minimum factor of safety of 1.0 for the worst known loading and groundwater conditions. A designer who chooses to use this assumption as the basis for a back-analysis approach to the design of remedial or preventive works is referred to Leroueil & Tavenas (1981) for an illustration of the pitfalls of such an approach. In the case of a failed or distressed slope, the causes of the failure or distress must be specifically identified and taken into account in the design of the remedial works.

Where an existing stable slope will be substantially modified, or where its stability will be affected by new works, the factors of safety given in Table 5.1 should be achieved.

5.2.3 Natural Slopes

Natural slopes are frequently close to limiting equilibrium over very large areas, and preventive works can be expensive and difficult. It is clearly not advisable to undertake extensive trimming-back of natural slopes in order to achieve what may only be marginal improvement in stability. In such cases, disturbance of natural slopes and vegetation and the need for costly preventive or protective measures may be avoided by siting structures away from areas that could be affected by landslide debris.

Natural slopes need not meet the factors of safety given in Table 5.1, provided that :

(a) the slope is undisturbed (e.g. has not been and will not be cut, stripped of vegetation, subjected to increased loading or subjected to increased infiltration by alteration of the natural drainage regime), and

(b) a careful examination is made to determine that there is no evidence of instability or severe surface erosion.

In assessing natural slopes, consideration should always be given to the possible presence of potentially unstable boulders.

5.2.4 Temporary Works

Factors of safety for temporary works should be the same as for new permanent works but with due regard for the risk-to-life category during construction and for the groundwater conditions likely to occur during the construction period.

5.3 STABILITY ANALYSES

5.3.1 Modes of Failure

When choosing a method of stability analysis for design, the probable mode of failure of the slope must be considered. The method chosen should model the failure mode.

The most common failures in Hong Kong's soils, fills and colluvium are very shallow, being controlled by depths of weathering and infiltration during rainstorms. The failure surfaces are often roughly planar or only slightly concave over a considerable portion of their area.

The structure of a material at less than its critical density can collapse on shearing and, if the material is saturated or nearly saturated, high pore pressures can develop very rapidly. There is rarely any prior warning of a failure occurring in this manner, and the resulting debris, if liquified, can travel very large distances at high speed, even on relatively flat surfaces. Materials of dry density less than critical can be formed by inadequate compaction of fill, by deposition of colluvium in a loose state or by weathering insitu.

Highly and completely weathered rocks may behave as soil in terms of their engineering properties, and slopes should therefore be assessed by analysing a wide range of potential failure surfaces throughout the mass. In such materials, failure along relict joints should also be considered. Failure in less weathered rock is always controlled by the joint system. Typical failure profiles in weathered rocks and soils in Hong Kong are shown in Figure 5.1.

5.3.2 Input Data

Details of topography, geology, shear strength, groundwater conditions and external loadings are required for the analysis of slope stability.

(1) Topography. An accurate site plan is required showing the positions of site investigation holes, joint survey areas and the location of cross-sections to be analysed. Cross-sections must be surveyed to a detail such that they can be drawn to a natural scale large enough to read off dimensions to an accuracy of about 0.1 m. A scale of 1 to 100 is usually suitable. A larger scale, 1 to 50 or 1 to 20, may be required to obtain accurate dimensions for the stability analysis of slopes that are less than ten metres high.

(2) Geology. The depth of weathering, presence of colluvium or fill and the structure of the fresh and weathered rock should be assessed from the results of the surface and subsurface investigations (Chapter 2).

For analysis, geological data must normally be interpreted in terms of layers or zones of materials of like engineering characteristics. When dealing with weathered rocks, one of the zonal schemes proposed by the Geological Society of London (1972), the International Association of Engineering Geology (1981) or the British Standards Institution (1981) may be appropriate for this purpose. However, there may be occasions when it is more useful to develop an individual scheme to classify zones of material for a specific site.

The details of site geology available for analysis are usually based on a small amount of data, which is often open to more than one interpretation, and a range of possibilities must be considered when carrying out stability analyses. Geological conditions should be continually assessed during construction, and designs modified if geological conditions differ from those assumed. The geological structure assumed for the design should be shown on the slope cross-section.

(3) Shear strength. Shear strengths for the slope-forming materials,

normally expressed in terms of the effective stress parameters c' and Ø', should be determined by testing representative samples of matrix materials (residual soil and weathered rock) and discontinuities. The samples should be tested at stresses comparable to those in the field, and should be saturated, unless direct evidence is obtained by insitu measurement or observation to indicate that the materials in the field do not become saturated or near-saturated under the design rainfall conditions. Reference should be made to Section 3.7.

The shear strength of an unsaturated material is usually substantially greater than that of the same material in the saturated condition. However, near-saturation may be achieved within both vegetated and surface protected slopes for a ten-year return period rainfall, unless the slopes are effectively sealed against both direct and indirect infiltration. Therefore, soil suction should not generally be relied upon in design as a factor contributing to long-term slope stability.

To remain effective, protective slope surfaces must be maintained. For environmental reasons, vegetation is to be preferred to hard surfacing materials (chunam and sprayed concrete) as a protective cover for the prevention of surface erosion.

(4) Groundwater. Groundwater conditions should be assessed during and following the site investigation by installing and reading piezometers, and by observing traces of seepage. The levels obtained during the observation period are unlikely to represent the peak levels that will occur during the design storm. Therefore, an estimate must be made of the extent to which water levels in the slope will increase in response to rainfall and other factors. Reference should be made to Section 4.4.

Slopes should be designed for the groundwater conditions resulting from a ten-year return period rainfall. In addition, slopes in the high risk-to-life category should be checked to determine the sensitivity of their stability to water levels above those predicted as a result of the ten year return period rainfall. This requires the designer to consider the predicted worst groundwater conditions. Amongst other causes, these worst conditions may be due to major leakage from services, the blockage of filters or drains, exceptionally heavy rain (return period greater than ten years) and the filling of tension cracks or open joints. The predicted groundwater levels to be used in stability analyses for both the ten year return period rainfall and the worst conditions should be shown on the slope cross-section.

The possibility of formation of transient perched water tables at the interface of layers of differing permeability must be considered from examination of the material profiles within a slope and the catchment above the slope. Perched water tables may form at the interface between colluvium or fill and the underlying soil, between zones of weathering, or within zones of weathering or colluvium. Such transient conditions normally form and dissipate relatively rapidly and may be very difficult to detect by observation of seepage or piezometric measurements.

For rock slopes, maximum water pressure may develop during heavy storms as a result of tension cracks or open joints becoming full of water. The fissure water pressure acting on the joint should be taken to be a maximum at the base of the tension crack, reducing nearly to zero where the joint daylights in the slope face. Water pressures can vary from joint to joint within a rock mass, and the pressures measured by piezometers are only relevant to the joints that intersect the filter surrounding the piezometer tip. They can be relied upon only if the filter intersects a single joint.

Leakage from services, such as sewers, stormwater drains and water mains, can cause both saturation of slopes and rising groundwater levels. Service trenches should normally be constructed as suggested in Chapter 9 but, where there are details of proposed services available, and where there is any possibility of leakage into the slope, this should be taken into account in the design.

(5) External loadings. Loadings from traffic, building foundations, retaining walls, spoil heaps, pylons, blasting, pile driving, etc. that can influence the stability of a slope must be included in the analysis, with an appropriate allowance made for any safety factors that may already have been incoporated into the loadings. If external loadings are to be considered, the method of analysis must allow for their inclusion.

Earthquake forces need not generally be included in slope stability analyses in Hong Kong.

5.3.3 Methods of Analysis

Many methods of stability analysis are available for the design of soil slopes. The majority of these are based on limit equilibrium, although some are based on plastic limit theory and some on deformation. Fewer methods are available for the analysis of rock slopes, and almost all of these are limit equilibrium methods. The better known methods for soils and rocks and water pressures are listed in Tables 5.5 and 5.6. The advantages and limitations are given, and recommendations are made as to their application. The references given in the Table are not necessarily the original references but are easily accessible in books or papers where the methods are given in enough detail for design office use.

Lumsdaine & Tang (1982) reviewed the methods of analysis that are commonly used in Hong Kong, described the sources of computational error that can occur and compared the results obtained from the simplified and rigorous methods.

5.3.4 Three-Dimensional Effects

The majority of stability analyses assume a slip of infinite width in a planar slope. This assumption is reasonable in the middle of a slip, but the ends of the slip are affected by shear on the end walls. The resulting underestimate of overall stability is difficult to quantify. Hovland (1977) takes the cases of a cylindrical slip with conical ends and of a wedge with two sliding planes. In the first case, he shows the three-dimensional effect to increase the factor of safety by 10% to 50% depending on the ratio of the width of cylinder to height of slope. The maximum difference occurs when the cylindrical section is non-existent. In the case of the wedge, the increase is again about 50% for soils with $c/\gamma H$ greater than one. However, for small values of cohesion the difference reduces and for cohesionless soils, there is a reduction of the factor of safety by about 5%. Hutchinson et al (1973), by considering the earth pressures on the sides of a slide in Etruria Marl, show an increase in factor of safety of 16% for the three-dimensional case.

Plan curvature of the slope can also affect the overall stability of a slope. Slopes with concave plans are theoretically more stable than those with convex plans. However, drainage of convex slopes is better and the lower pore pressures can reduce the difference in factor of safety. The effect is most marked in jointed rocks. Opencast mining experience (Piteau & Jennings, 1970; Hoek & Bray, 1981) has shown that for concave slopes with radii of curvature less than the slope height, the slope angle can be 10° steeper than that predicted by conventional stability analysis, while the convex slope should be 10° flatter. The influence of curvature becomes negligible when the radius is greater than twice the slope height.

Corestones and boulders in a soil slope can increase both the strength and density of the sliding mass (Holtz & Gibbs, 1956; Holtz, 1960). The effect on the factor of safety cannot usually be quantified for routine stability analyses.

5.3.5 Recommended Methods of Analysis

(1) Preliminary design and negligible risk slopes. For preliminary design or for slopes in the negligible risk category, a time consuming complex analysis is seldom justified, as the input data are often scanty. Hoek's charts (Hoek & Bray, 1981), infinite slope and sliding block analyses are most useful for a rapid assessment of the stability of soil slopes.

(2) Low and high risk slopes. Non-circular analytical methods, such as those by Janbu (1972) or Morgenstern & Price (1965), are recommended for most soil slopes in Hong Kong. However, a sliding block or Bishop (1955) circular analysis may occasionally be more appropriate.

The assumptions made in obtaining shear strength parameters and pore pressures for use in the calculations are often such that a more rigorous but complex method, such as that of Morgenstern & Price, is not justified for the analysis. The Janbu routine method is sufficiently accurate for most purposes and is recommended for general use, with the proviso that there are circumstances under which the method does not work well (see Table 5.5).

Where a more rigorous method is warranted, one of the more complex methods, such as the Janbu Rigorous method or the Morgenstern & Price method may be considered. Both produce results of similar accuracy, although the former experiences numerical difficulty due to the less rigorous formulation of the equations used in its solution. However, these difficulties can usually be overcome by alteration of the slices used in the analysis. The Janbu Rigorous method has the advantage of being far less demanding on computer resources than the method of Morgenstern & Price. It is also possible to carry out the analysis by hand calculation, unlike the latter method.

These two methods are useful for special cases; for example, where the shear surface passes through two or more very different materials, where high quality input data is available or when carrying out a back-analysis of a failure.

There are fewer methods of stability analysis available for rock slopes (see Table 5.6). The principal types of failure of rock slopes are rotational, translational and toppling modes, and combinations of these modes may also be possible. Rotational failure, either circular or non-circular, can occur in slopes of highly-jointed or shattered rock. In these cases, analysis can be carried out by the methods of slices, as used for analysis of soil slopes. Detailed methods of analysis of rock slopes are given in Hoek & Bray (1981).

5.3.6 Reliability of Stability Analyses

The uncertainty associated with the design values of soil strength, groundwater conditions and other input data, is not taken into account directly in the usual stability analysis. Because of the innate variability of all the parameters used in the analysis, and because of the inevitable sparsity of test data, the reliability of the final calculated safety factor may be low, and the actual safety of a design could be much less or much greater than that calculated.

In principle, it would be possible to estimate the probability of failure of a design by regarding all the parameters as random variables, and to assess the reliability in terms of this failure probability. In practice, there will rarely be sufficient data to justify a full probabilistic analysis but it will always be possible to estimate the mean value of the safety factor (F) and its standard deviation (S_F). From these, it is possible to determine a standardised reliability index (R_F), which is the difference between mean safety factor and a safety factor of unity divided by the standard deviation :

$$R_F = \frac{(F - 1)}{S_F} \quad . \; . \; . \; . \; . \; . \; . \quad (5.1)$$

The significance of the reliability index in terms of the probability of failure is shown in Figure 5.2.

Even without interpretation as a probability of failure, the reliability index is a useful measure, particularly when comparing alternative designs. If two designs have the same mean safety factor but different reliability indices, then the design with the larger R_F value will be the safer of the two. Conversely, two designs with different mean safety factors but an equal reliability index will be equally safe.

There are several ways of calculating the mean and standard deviation of a function of random variable (Lumb, 1974), but the simplest and most convenient for routine work is the Taylor series.

Normally, there are insufficient observational data to determine the range of possible pore water pressures. It is therefore preferable to determine the reliability index for a number of fixed values of pore water pressure and to examine these using engineering judgement to decide if the factors of safety determined for each water condition are adequate.

5.3.7 Sensitivity Analysis

As an alternative to accounting for variability of the soil forming the slope by the statistical methods described in Section 5.3.6, the slope stability analyses may be repeated using differing values of c' and Ø' and different groundwater conditions. The effect on factor of safety as a result of variations in these parameters can then be assessed and, if considered necessary, more or less conservative criteria may be adopted for design, based upon the frequency distribution of the values of the parameters obtained during testing and observation.

5.4 DESIGN OF CUT SLOPES

5.4.1 Slope Profile

Cut slopes should be designed to the factors of safety in Table 5.1. A slope may be cut at one angle for its full height, or alternatively, at angles that vary according to the material through which the slope is cut. In the case of soil overlying rock, the slope can be steeper through the rock than through the soil. Berms may be formed at regular intervals up the slope. If berms are provided, the stability of both the overall slope and the slopes between berms should be checked. In Hong Kong, failure frequently occurs in the slope above the top bench where the excavation is formed in the most heavily weathered material and where the slope is most susceptible to infiltration through the natural ground above the crest of the slope.

If slopes are to be formed in fresh or weathered rock so that joints that dip into the excavation are exposed, it may be safer and more economic to form these slopes on a continuous profile parallel to the joints, rather than providing berms on a slope that has intermediate steeper slopes (see Figure 5.3). Consideration should be given at the design stage to the method of forming the final face in rock excavations. Controlled blasting techniques should be used to achieve an acceptable standard of finish.

Berms, where used, must be at least 1.5 m wide and should generally be spaced at not more than 7.5 m vertical intervals. One of the main advantages of providing intermediate berms with drainage channels is the reduction in volume and velocity of runoff on the slope surface and the consequent reduction of erosion and infiltration. Wide berms can also catch debris from slips occurring higher up the slope, reducing the damage to structures at the toe of the slope. They also improve access for maintenance but can cause increased infiltration unless drains and surface protection are well maintained. Benches can also reduce stability in adversely jointed soils and rocks.

Excavation of soil and rock cuttings is discussed in Chapter 9.

5.4.2 Improvement of Stability

If the initial analysis shows that the stability of a proposed cut is inadequate, the designer should first establish whether a change in the geometry of the excavation is feasible to reduce the height or angle of the cutting. At an early stage in design, this may be the cheapest solution. Otherwise, stability may be improved by the provision of retaining structures, internal drainage to lower the permanent water table, anchors and rock bolts, or alternatively, by a combination of these methods.

The design of retaining structures and the various factors of safety required of them are discussed in Chapter 7. The construction methods are discussed in Chapter 9.

Horizontal drains or drainage galleries are effective only where the original groundwater level is high, relative to possible failure surfaces. Any such installation on which the safety of a slope depends will require monitoring throughout the life of the slope. The design of horizontal drains and galleries is discussed in Chapter 4 and their maintenance in Chapter 11.

Force may be applied to a potentially unstable rock or soil mass to increase the normal stress on a potential failure plane, or to tie the unstable mass to a stable one. The force may be applied directly, or through a retaining structure in both soils and rock. Ground anchors may be used to apply this force where the overall stability of a slope is to be ensured. Guidance on the design of anchors is given in the Model Specification for Prestressed Ground Anchors (Brian-Boys & Howells, 1984). Rock bolts may be used where localised areas of instability are to be remedial (see Sections 5.4.3 and 5.4.5).

5.4.3 Treatment of Rock Slopes

Most rock slopes, after bulk excavation, need some form of treatment to ensure continued stability. Table 5.7 gives the range of applications or various stabilisation measures, and Figure 5.4 shows typical situations in which these methods may be used.

(1) Scaling. Immediately after excavation, loose blocks or boulders should be removed from exposed rock faces. Potentially unstable blocks should be removed carefully, without blasting, to prevent further loosening of the face.

(2) Buttresses. Buttresses to support unstable rock masses may be concrete or masonry gravity structures, which can be anchored to improve stability. Drainage should be provided behind buttresses to prevent water pressure building up in covered fissures.

(3) Dentition. Bands of soft material that are exposed in a rock face should be trimmed-back from the face. The resulting slots should then be filled with a suitable filter material protected by masonry or reinforced concrete to prevent erosion of the soft material. In Hong Kong rocks, these soft seams will normally only occur where weathering has taken place along a joint, a fault or a dyke. This penetrative weathering is indicative of water flow. Weepholes should therefore be provided in the facing to ensure that the soft seam is adequately drained and that high water pressures do not develop. Cavities, overhangs and open joints can be treated in the same way as exposures of bands of soft material. If required, concrete or masonry facings should be dowelled into the harder rock in which the soft seam occurs.

(4) Sprayed concrete. Sprayed concrete (details are given in Chapter 9) can be used to provide surface protection for zones of weak or highly-fractured rock. Where concrete is required to span between rock bolts or other supports, it should be suitably reinforced with steel fabric that can be attached to the rock surface with dowels and bolts before spraying. Sufficient weepholes should be provided where necessary to prevent a build-up of water pressure behind the surface.

(5) Dowels. Dowels are untensioned steel bars, usually 25 mm to 32 mm

diameter and 1 m to 3 m long, grouted over their full length into holes drilled in the rock. They are used for reinforcing closely-jointed rock and for anchoring reinforcement, concrete or masonry and small blocks of rock. The design of dowels is discussed in Section 5.4.4.

(6) Rock bolts. Rock bolts are suitable for stabilising localised areas but should not be used as a major slope support system. They are generally tensioned steel bars that comprise a short anchorage zone in sound rock and an unbonded zone in which tension is developed. The head of the bolt is threaded and fitted with a nut. Tension should normally be applied by a jacking system. The load developed is applied to the face by means of a steel plate bearing onto the rock surface, although for weak or badly-fractured rocks, a concrete pad may also be required. Typical rock bolts are 25 mm to 40 mm in diameter, 3 m to 6 m long, and have a tensile working load of up to 100 kN. The design of rock bolts is discussed in Section 5.4.5.

5.4.4 Design of Dowels

Stress is induced in a dowel when movement occurs along the discontinuity being stabilised. In hard rock, dilation occurs as movement along the discontinuity causes riding-up on the asperities. Provided the bonded length is adequate on either side of the discontinuity, tension and shear are induced in the dowel. The magnitude of these forces is dependent on the roughness of the discontinuity, the orientation of the dowel to the discontinuity and the relative movement necessary to develop peak shear strength and peak dilation.

Simplified design methods for dowels are given by Sage (1977) and Bjurstrom (1974).

For mild and high-yield steel dowels, the maximum stress should be limited to 50% of the guaranteed ultimate tensile strength. An allowance of 2 mm sacrificial thickness on the radius should be made for corrosion, and an annulus of grout of 6 mm minimum thickness should be provided around the bar. It is inappropriate to use prestressing steel for dowels unless double corrosion protection is provided.

5.4.5 Design of Rock Bolts

Rock bolts provide positive support to the discontinuity being stabilised by increasing the normal stress and, hence, the mobilised shear strength.

The bolt anchorage in sound rock may be formed either by bonding or by use of a mechanical device such as a torque-set bolt. The bond length should be designed with a factor of safety of 2.0 against pullout. Mechanical anchorages, where used for permanent bolts, should be grouted after initial installation to provide the required bond length. Stresses of up to 50% of the guaranteed ultimate tensile stress may be used, and bolts should be proof-loaded to 1.5 times working load to demonstrate adequate performance of the installed bolt.

Where prestressing steel is used in a rock bolt, it should be provided with double corrosion protection. Single corrosion protection is considered adequate for high-yield or mild steel bars, provided an allowance for generalised corrosion of 2 mm on the radius is made. For permanent bolts, single corrosion protection of the free length should consist of grout, a grease-filled sheath, or other suitable protection method. The corrosion protection of the head should be carefully considered, and the quality of the protection provided should be consistent with that for the rest of the bolt. Particular attention should be paid to detailing of the corrosion protection system and to site supervision to ensure that the risk of corrosion is minimised. Where grout is used, a minimum cover of 6 mm should be provided to the bolt.

Where persistent discontinuities occur, ties may be required between rock bolts to prevent failure. These ties may be structural channel sections with slotted holes cut to accommodate variations in the spacing of the bolts or, alternatively, they may be cast insitu concrete beams.

The load required in a rock bolt to prevent sliding of a block on an inclined plane may be calculated by referring to Appendix 3 of Hoek & Bray (1981). The shear strength used in the calculation of bolt load should be appropriate to the joint being stabilised.

5.4.6 Boulder and Rockfall Control

Where potentially unstable boulders are identified as a threat to proposed development downslope, there are a number of possible measures to improve stability.

1) Removal of boulders. Before development proceeds at the base of a boulder strewn slope, unstable boulders should be removed or, if too large, broken into small fragments by blasting or mechanical means to assist removal.

(2) Breaking-up of boulders and repositioning. Where it is uneconomic to transport boulders from the slope, breaking-up followed by repositioning of the broken pieces on the slope may be a solution.

(3) Stabilisation of boulders insitu. A variety of techniques are available including concrete underpinning, grout injection of boulder rubble to form restraining buttresses, steel dowels and bolts, tying-back with a combination of steel mesh and sprayed concrete or with steel hawsers, and surface protection combined with drainage to prevent erosion of supporting soil from around boulders.

It may not be economically practical to eliminate all rock and boulder falls from cut faces and steep natural slopes. In these circumstances, precautions should be taken to reduce the danger that such falls present to life and property. Figure 5.5 shows some rockfall control methods suggested by Fookes & Sweeney (1976). Fences can be provided at the base of slopes, combined with energy-absorbing catchpits, to contain boulders within a designed limit (Ritchie, 1963). Alternatively, the energy of moving boulders can be dissipated by the use of deformable walls such as gabion structures. Where space allows, boulder trajectories can be diverted by protective embankments into areas where they cannot cause damage.

5.5 DESIGN OF FILL SLOPES

5.5.1 New Earthfill Slopes

Fill slopes should be designed to the factors of safety in Table 5.1 using shear strength parameters obtained from tests on samples of the proposed earthfill material compacted to the design density. If a fill slope is provided with berms, these should be at least 1.5 m wide and not more than 7.5 m apart vertically. Surface protection and drainage should be designed to prevent erosion (see Chapter 8 and 9).

The foundation for the fill should be carefully prepared by removal of vegetation, topsoil and any other unsuitable material. If appropriate, the foundation should be benched to key the fill into an existing slope, and any inadequately stable slope material should be removed.

A free-draining layer, conforming to the filter criteria given in Chapter 4, may be required between the fill and natural ground to eliminate the possibility of high pore pressures developing where they could cause slope instability. Where springs or seepage-traces are found during the formation of a fill, suitable drains should be provided to collect the flow from them and to discharge it outside the limits of the fill. Care should be taken to prevent drains acting as a source of infiltration.

5.5.2 Treatment of Existing Earthfill Slopes

Where a slope of loose fill is to be stabilised to eliminate the possibility of a flow-slide, the surface layers should be stripped to a vertical depth of not less than 3 m and replaced with fill compacted to a density of not less than 95% of British standard maximum dry density. A drainage system may be required between old and recompacted fill to prevent the development of water pressure behind the compacted face. Alternatively, in some circumstances the required insitu density may be achieved by the use of dynamic compaction techniques.

While the simplest method of restoring a failed slope is to remove the slipped material and reform the slope at a safe angle determined by stability analysis, this is not always possible because of the constraint of land availability. In such cases, retaining structures can be incorporated into the design to permit the adoption of flatter slopes and to improve overall stability without the slope encroaching onto surrounding land.

Where a slope must be returned to its original profile, the failed material should be excavated and any steep failure surfaces benched prior to receiving fill. Drainage measures should be incorporated to deal with any groundwater flow. Slopes may be reconstructed with compacted earthfill, or with cement stabilised soil, lean mix concrete, masonry or rockfill.

5.5.3 Rockfill Slopes

Slopes composed of well-graded, free-draining rockfill materials are not normally subject to saturation or groundwater rise during rainfall, and consequently the factors of safety given in Tables 5.1 and 5.4 are inappropriate for potential failure surfaces within the rockfill mass.

Rockfill does not easily lend itself to conventional shear strength testing, but a conservative estimate of the angle of repose of the material may be used as the safe slope angle, provided that it is properly graded and compacted, and provided the foundation is stable. Some triaxial test data on rockfill materials are given by Leps (1970).

A filter layer may be required under the rockfill to prevent undermining and the migration of fines into the fill. The preparation of the foundation should be as for earthfill.

6. FOUNDATIONS ON SLOPES

6.1 INTRODUCTION

The design of foundations for structures on or adjacent to slopes must take into account the interaction between the structure and the slope. Two criteria must be considered :

(a) the influence of the adjacent slope on the bearing capacity and settlement of the foundation, and

(b) the effect the foundation will have on the stability of the slope.

The first criterion recognises that there can be a significant reduction in bearing capacity (both horizontally and vertically) due to an adjacent slope (Vesic, 1975; Poulos, 1976; Schmidt, 1977) and the second criterion is important because the stability of a slope can be affected by excavation for the construction of foundations on or adjacent to the slope, the load imposed by foundations on or above the slope, or the temporary or permanent change in groundwater regime caused by construction of the foundation.

Typical foundation types in Hong Kong are spread footings, caissons (hand-dug and machine bored) and piles (percussion, bored, precast and cast-in-place). Spread footings and hand-dug caissons are the most common because heavy machinery is not required for construction, the operation of which is difficult on steep hillsides. Shallow foundations (spread footings) are used for light loads, and deeper foundations (piles) are used where the bearing stratum is at depth or where the stability of the slope would be impaired by any additional load from the foundation.

6.2 SHALLOW FOUNDATIONS

6.2.1 Bearing Capacity and Settlement

The ultimate bearing capacity of a shallow foundation on a slope is lower than that for the same footing on level ground. A general expression for the ultimate bearing capacity of such foundations, which includes factors to allow for the slope of the ground, is given by Vesic (1975). Appropriate values for these ground-slope factors are given in Figure 6.1. This Figure is reproduced from Geoguide 1 (Geotechnical Control Office, 1982b) to which reference should be made :

(a) for guidance on the particular problem of estimating the bearing capacity of a foundation set-back from the crest of a slope, and

(b) for a discussion of other factors that can influence bearing capacity.

Where shallow foundations are constructed at more than one level on a slope, the foundations at the higher level may impose additional loading on the lower ones. This additional loading must be taken into account in the design.

In general, bearing capacity calculations do not allow for the fact that the soil forming the slope is already under stress, and so it is important to assess the overall stability of the slope under the influence of the loaded foundation. However, an acceptable factor of safety against slope failure obtained from a stability analysis that includes the influence of foundation loads, does not necessarily mean that the foundation is acceptable in terms of settlement.

6.2.2 Slope Stability

As a general rule, the stability of a slope affected by foundations should be checked if the slope angle is greater than $\phi'/2$ (Vesic, 1975). Where this is so, the foundation can be considered as an equivalent line load or a surcharge imposing horizontal and vertical loads and incorporated into the stability analysis.

The backfilling to a foundation may be poor, and so the stability analysis should consider the possibility of a tension crack forming on the upslope-edge of the foundation.

For shallow foundations on or above rock slopes, the stability analysis should take into account potential instability due to adversely orientated discontinuities. The analytical methods of Hoek & Bray (1981) are useful in this respect.

The stability of a slope can be impaired by excavation for the construction of shallow foundations on or adjacent to the slope and the demolition of structures supporting the toe of the slope. Both these effects should be considered during the analysis. In order to minimise the short term instability of a slope, excavations should be as small as possible and should be properly shored.

6.3 DEEP FOUNDATIONS

6.3.1 Lateral Loads

The horizontal stresses in a soil slope vary throughout the slope and, for deep foundations, the horizontal loading on the upslope side of the foundation is larger than on the downslope side. However, in a slope that has an acceptable factor of safety against failure, the difference in horizontal load is negligible (Schmidt, 1977) and need not be considered during the design of most deep foundations.

However, high lateral loading can be transferred to foundations in situations where there is significant ground movement (i.e. where the slope above or below the foundation fails or where the slope in front of the foundation is excavated) or where there is only a small ground movement (i.e. creep) but where the foundation is very stiff.

Various methods of analysis are available for the analysis of single piles subjected to lateral loading due to ground movement. Wang & Yen (1974) and Ito & Matsui (1977) use limiting equilibrium methods and suggest ways in which arching between closely-spaced piles can be considered. Poulos & Davis (1980) use finite difference methods; these, however, are very dependent on the correct definition of the stress-strain characteristics of the soil layers surrounding the piles (De Beer, 1977).

Where possible, lateral loads on deep foundations should be prevented. This can be achieved by either :

(a) stabilising potentially unstable slopes before construction of the foundation, or

(b) by the provision of an annular sleeve around the foundation.

An annular sleeve is a space of sufficient width between the foundation and the surrounding soil so that both can move without interaction. The space, which can be air filled or can contain a suitable compressible material, must be wide enough to accommodate the ground movements expected and the deflection of the foundation itself.

For air-filled spaces, when a lining has to be used to support the soil, the lining must prevent the ingress of groundwater or surface water into the space and must prevent the space filling up with soil. The plug at the top of the air-gap between the lining, and the foundation should be designed so that there is no load transmitted through it.

For spaces that are filled with a compressible material, the design width should take into account the compressibility of the material itself, especially as a result of the placement of wet concrete during construction.

In such circumstances, it may be appropriate to install the annulus eccentrically around the foundation with the centre of the annulus upslope of the centre of the foundation.

6.3.2 Slope Stability

Deep foundations can adversely affect the stability of slopes :

(a) by transmitting vertical or horizontal loads to the slope,

(b) by the removal of support during the excavation for construction of the foundation, or

(c) by the temporary or permanent change in the groundwater regime caused by the foundation.

To prevent the transmission of vertical loads onto a slope, the founding depth for a deep foundation should be below any potential failure plane within the slope. To prevent the transmission of horizontal loads, an annular sleeve should be provided. In this case, the annulus may be installed eccentrically around the foundation, but with the centre of the annulus downslope of the centre of the foundation. Where these measures are not possible and loads from the foundation are likely be transmitted to the slope, the stability of the slope under the influence of these loads should be assessed. However, this assessment is not easy as the methods usually adopted for stability analysis (limit equilibrium methods) are not compatible with those used to evaluate soil pressures (elastic methods).

When caissons or piles are closely spaced, there may be a reduction in overall permeability that would result in a rise in the groundwater level upslope of the foundation (Pope & Ho, 1982). This possibility must be considered in the design, and any assumptions made should be checked by the installation and monitoring of piezometers after the foundation is in place.

Piles and caissons can be used to support slopes, and methods for their design in these situations are given by Gould (1970) and Fukuoka (1977).

7. RETAINING STRUCTURES

7.1 INTRODUCTION

This chapter considers the forces exerted on retaining walls by soils. Many important aspects will be covered only briefly because greater detail is available in Geoguide 1 : Guide to Retaining Wall Design (Geotechnical Control Office, 1982b). The design of ground anchorages is not included in this chapter but guidance on many aspects of anchorage systems (e.g. installation, proof testing, factors of safety and monitoring) is given in the Model Specification for Prestressed Ground Anchors (Brian-Boys & Howells, 1984).

It is common practice to define the stability of a wall in terms of a factor of safety obtained from a numerical stability analysis. This chapter is concerned with this 'classical' analytical approach.

7.2 FORCES ON WALLS

7.2.1 Earth Pressures

Methods of calculating earth pressures are given in most textbooks, for example that by Terzaghi & Peck (1967). Some discussion on the limitations of these methods is given in Geoguide 1. Very large wall movements are required to mobilise the ultimate passive pressure; for design purposes, the passive pressure should be limited to not more than 50% of the calculated ultimate passive pressure. Alternatively, passive pressures may be determined by use of appropriate partial factors on shear strength and wall friction, as recommended in Geoguide 1 for sheet walls.

If relative movement can occur between a wall and the supported soil, the effects of wall friction may be taken into account when determining the pressure acting on the wall.

The magnitude of the forces acting on retaining structures is affected by the soil and structure interaction. Movements of walls necessary to generate different pressure conditions, proposed by Wu (1975), are quoted in Geoguide 1.

Design curves for active and passive earth pressures are given in several publications, notable NAVFAC DM-7 (1971). These design curves are reproduced in Geoguide 1.

The adoption of active conditions for retaining wall design assumes movement of the supported material. This may be unacceptable because of possible damage to adjacent existing structures or services (National Research Council of Canada, 1975). Under these circumstances, walls must be designed to prevent movement of the supported soil mass (see Section 7.3.6).

When utilizing the effects of passive pressure in retaining wall design, the possibility of excavation subsequently being carried out in the passive zone should be considered. If this can happen, the wall should be designed ignoring passive pressures.

There is little information available on the magnitude of the coefficient of earth pressure at rest for the insitu weathered rocks of Hong Kong. However, some guidance is given in Geoguide 1.

7.2.2 Water Pressures

The water pressure acting on a retaining wall behind which there is inadequate drainage can be considerably greater than the active pressure. Although, in practice, a drain is often used against the backface of the wall, the most effective way of preventing the development of these pressures is to provide an inclined drain between the backfill and the insitu soil (Cedergren, 1977).

If an inclined drain is provided that is capable of carrying all the flow, rainfall infiltrating from the surface flows vertically into the drain without producing pore pressures in the soil or water pressure on the wall. In contrast, for rain infiltrating into the platform above a wall to reach a vertical drain, there must be a horizontal component of flow and therefore positive pore pressures. Under these conditions, the pore pressure is zero in the drain but positive in the soil mass, thus reducing the shear strength and increasing the active pressure or reducing the passive pressure. Further details on this, including sketched flow nets, are given in Geoguide 1.

Allowance for the effects of these pore pressures should be made by including the total water pressure as another force acting on the trial wedge. Detailed guidance on this is given in Geoguide 1.

Compacted backfill of low permeability between the drain and the insitu soil may have serious damming effects on groundwater flow and, as a consequence, it may render a particular analysis and design of a wall inapplicable. If adequate drains are not to be provided behind walls, for example diaphragm, basement and sheet pile walls, the walls must be designed to resist any water pressures that can develop after construction. Water pressures acting on the back of a wall are accompanied by uplift pressures that act on the base. These must also be included in the analysis of stability because they reduce the normal stress and therefore the resistance to sliding.

Figure 7.1 shows typical arrangements for drainage behind walls. The long-term effectiveness of the drainage materials should be carefully considered when preparing the detailed design. A channel at the top of the wall prevents the ingress of runoff into cracks between the wall and the backfill, and an impervious base prevents infiltration from the drain into the foundation.

Services upslope of a wall should be installed as described in Chapter 9, as leakage entering the backfill can increase water pressure, and this has been known to cause retaining wall failures.

7.2.3 Surcharge Loads

The magnitude of surcharge loads due to traffic on adjacent highways in Hong Kong is given in Table 4 of Geoguide 1. Design methods for including these loads and those due to buildings, stock piles and construction plant are also given.

7.2.4 Construction Loads

Backfill to retaining walls should normally be compacted in thin layers using light compaction plant. This avoids imposing on the wall the high loads that are generated by heavy compaction plant. However, if heavy compaction plant is to be used, this must be taken into account when determining the loads for which the wall is to be designed. Lateral pressures produced by compaction are discussed in Chapter 3 of Geoguide 1, and guidance is given on methods of assessing lateral pressures produced by compaction plant. Heavy compaction of backfill can cause these to be much greater than at-rest pressures.

Wall designs should also take account of abnormal loads that may be imposed during construction. These may be caused by earthmoving plant, batching plants or cranes stationed above a wall.

7.3 STABILITY OF RETAINING WALLS

7.3.1 Base Friction

The value of the base friction angle depends upon the nature of the materials used to construct the wall and upon the methods adopted for construction. For walls without a key, the appropriate value of base friction is 2/3 ϕ'. However, it may be possible to justify a higher proportion of ϕ', but only in those cases where it can be assured that the excavation of the base will be carried out in the dry season, that disturbance and deterioration of the subsoil is prevented by the construction of an adequate blinding layer immediately after foundation exposure, and that the works will be professionally supervised.

7.3.2 Bearing Capacity

Closely related to the problem of overturning of retaining walls is the determination of bearing capacity. The forces acting on the back of a retaining wall produce a non-rectangular stress distribution below the base, and the maximum pressure should not exceed the allowable soil pressure derived from consideration of the bearing capacity and settlement. A number of methods for determining bearing capacity are available, but often widely varying values are obtained using the various methods. The method of Vesic (1975), which is discussed in Section 6.4 of Geoguide 1, takes account of slope angle, shape and depth of foundation and eccentricity of resultant load.

Geoguide 1 also includes guidance on allowable bearing pressures for jointed rock.

7.3.3 Factors of Safety

A retaining wall can fail by sliding, by rotation, as a result of bearing capacity failure, or as part of a larger-scale slope failure. Failure can also result from the structural failure of the wall itself, particularly in the case of a masonry wall.

The factors of safety adopted for design must be applicable to the mode of failure being considered. A definition of each of the factors of safety to be adopted is given in Geoguide 1 : Guide to Retaining Wall Design (Geotechnical Control Office, 1982b). For free-standing walls, which derive their resistance principally from gravity force and base friction, factors of safety should conform with Table 7.1. Recommended minimum values are given for the two separate cases :

(a) for the design of a new wall where design parameters are derived from geological and geotechnical investigation, and

(b) for the analysis of an existing wall and for the design of remedial and preventive works.

For the design of new retaining walls, reference should be made to the general guidance in respect of risk to life given in Chapter 5 for the design of slopes.

When analysing an existing gravity retaining wall to determine the extent of any remedial or preventive works required, the performance history of that wall can be of considerable assistance to the designer. There is, for example, an opportunity to examine the geological conditions surrounding the wall more closely than for an undeveloped site, and to obtain more realistic information on groundwater. The designer is therefore able to adopt with confidence factors of safety for proposed remedial or preventive works that are lower than those specified in Table 7.1 for new walls. As long as rigorous structural, geological and geotechnical investigations are conducted (which include a thorough examination of wall maintenance history, groundwater records, rainfall records and any wall monitoring records), the lower factors of safety given in Table 7.1 may be used for the design of remedial or preventive works, provided that the loading conditions, the basic form of the modified wall and the groundwater regime remain substantially the same as those of the existing wall. There will often be instances, however, where particular circumstances will lead the designer to adopt for remedial and preventive works the standards specified for new walls (see Table 7.1).

For the design of remedial or preventive works to a gravity wall, it may be assumed that the existing wall has a minimum factor of safety of 1.0 for the worst known loading and groundwater conditions. In the case of a failed or distressed wall, the causes of the failure or distress must be specifically identified and taken into account in the design of the remedial works.

7.3.4 Retaining Walls with Keys

Where possible, deep keys should be avoided because the process of constructing a shear key frequently loosens and softens the material on which reliance is placed for passive and shear resistance. Complex section walls, although requiring marginally less material, take longer to construct and are often more expensive than simple walls with base widths increased to provide sliding resistance.

When walls with shallow keys are used, Huntington (1957) suggests that they be analysed, using effective stress parameters, assuming sliding occurs on a horizontal plane through the soil under the key, and that both active and passive forces are increased by the increased wall depth.

7.3.5 Sheet Retaining Structures

The earth pressure that acts on an earth supporting structure is highly dependent on the amount of lateral deformation that occurs **in** the soil. The flexibility of the wall, the nature of the lateral supports (e.g. anchored or strutted) and the construction procedures themselves strongly influence the resulting earth pressures.

The design conditions assumed for such walls are dependent upon the way in which loads are induced at the support. Walls that impose loads on their supports because of wall deformation, whether the supports be in the form of floor slabs, struts or unstressed anchors, can be designed using the pressure distribution rules proposed by Peck (1969) for the design of braced excavations. These rules, which are reproduced in Geoguide 1, were based upon measurements of loads in struts during construction, and they supersede the rules previously published by Terzaghi & Peck (1967). They were not, however, determined for soils of the kind found in Hong Kong.

If load is induced at the supports (e.g. by prestressing anchors or struts) and the wall is forced against the retained material, the pressure on the wall is dependent upon the imposed support loads and can have a lower limit of active pressure and an upper limit of passive pressure. Normally, the load applied to such walls is designed to prevent distress of adjacent structures. Geoguide 1 provides guidance on the design of these walls.

7.3.6 Settlements outside Excavations

The design of permanent retaining walls to prevent settlement of adjacent properties is mentioned in Section 7.2.1, but unless the temporary works are designed and carried out with equal care, serious ground movements can still take place. Peck (1969) gives details of settlements that have occurred around excavations in various materials supported by systems using both unstressed and prestressed anchors and raking struts. O'Rourke et al (1976) have extended this work. Peck makes the point that movements depend to a large extent on the supported soil and that, if good workmanship is used in installing and removing a well-designed, well-constructed temporary bracing system, ground movements in dense sands and relatively stiff, cohesive granular materials can be negligible. Poor workmanship can make even the best design ineffective, and this requires that construction of temporary bracing works be closely supervised. The data provided by Peck (1969) and O'Rourke et al (1976) are reproduced in Geoguide 1; however, these are based on North American experience and should only be used for approximate guidance in the soils of Hong Kong.

The construction of the Mass Transit Railway in Hong Kong has provided some data for Hong Kong conditions, and a number of papers have been published. Geoguide 1 provides further discussion on these.

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8. SURFACE DRAINAGE AND SURFACE PROTECTION

8.1 INTRODUCTION

This chapter discusses the effects of surface water on slope stability and describes suitable methods for designing surface drainage systems and surface protection for slopes.

The main purpose of surface drainage and surface protection is to improve slope stability by reducing infiltration and erosion caused by heavy rain storms. The slope drainage system should collect runoff from both the slope and the catchment area upslope and lead it to convenient points of discharge beyond the limits of the slope. Surface protection should be applied to slopes formed in materials susceptible to rapid surface erosion or to weakening by infiltration. The two methods are often used together as part of a preventive or remedial works design.

With the exception of vegetative surface protection, the techniques used have been well established in Hong Kong for many years. The design methods, however, are largely empirical and should be used with care. There are no absolute guidelines for design, and knowledge of local precedents on similar slopes is an invaluable aid. It is particularly important that designs make due allowance for existing drainage works that affect the proposed site development.

8.2 CATCHMENT PARAMETERS

8.2.1 Runoff

Runoff from a catchment depends upon many factors which include :

(a) rainfall intensity,

(b) the area and shape of the catchment,

(c) the steepness and length or the slopes being drained,

(d) the nature and extent of vegetation or cultivation, and

(e) the condition of the surface and nature of the subsurface soils.

The determination of runoff by reference to unit hydrographs (Linsley et al, 1982) has been found to be successful in many countries. In Hong Kong, the method was used for the design of drainage systems for small catchments but was not found to offer any substantial advantage over design methods using empirical equations to represent the complex relationship between rainfall and peak runoff. The "Rational Method" is commonly adopted because it is both simple and straightforward to use and, for the relatively small catchments in Hong Kong, it yields satisfactory results (Tin, 1969). The formula is :

$$Q = \frac{KiA}{3600} \qquad \ldots\ldots\ldots \quad (8.1)$$

where Q = maximum runoff (litres/sec),

i = design mean intensity of rainfall (mm/hr) which is dependent upon the time of concentration,

A = area of catchment (m^2), and

K = runoff coefficient

The runoff coefficient cannot be determined precisely. The recommended value of K for slope drainage is 1.0. Use of this value will generally result in an overestimation of runoff, especially on naturally vegetated slopes, and hence lead to overdesign of drainage systems. However, this additional drain capacity is useful in that it makes some allowance for silting, which is a very common problem. Nevertheless, the drainage should be designed to minimise siltation and prevent the possibility of debris causing blockage.

8.2.2 Area of Catchment

The catchment area is determined by reference to contoured plans and is defined as that area draining to the point in the drainage system under design. Where natural catchments are considered, the boundaries will be defined by the topographic contours, overland flow taking place at right angles to the contours. Where the hydrology of a catchment has been affected by the construction of catchwaters or pipe discharges, the effects of these constructions must be considered. In the case of a catchwater, the catchment area draining to a stream downstream of the catchwater is normally reduced, although allowance should be made for the flow from any spillways to the catchwater provided on that stream. The construction of drainage discharges on the stream, by diverting flows from adjacent areas, may lead to substantial increases in the size of the catchment and this should be allowed for when determining catchment area.

In the case of the point in the drainage system under design having a number of subcatchments upstream, the maximum runoff should be determined by direct application of the Rational Formula to the total area of the whole catchment contributing to that point, and not by summation of the maximum runoffs from each of the subcatchments.

8.2.3 Time of Concentration

The time of concentration is defined as the maximum time taken by surface water to travel from the catchment boundary to the point in the drainage system under design. Usually this corresponds to the line of flow from the most remote part of the catchment, but for catchments that do not have a single well-defined remote point on the boundary, it is good practice to check the calculation for a number of different flow lines, since the time of concentration depends on the average slope gradient as well as the the length of the lines of flow. For natural catchments, the time of concentration is calculated using the following equation, which is a modified form of the original Bransby-Williams equation :

$$t = 0.14465 \left[\frac{L}{H^{0.2} A^{0.1}} \right] \quad . \; . \; . \; . \; . \; . \; . \; . \quad (8.2)$$

where t = time of concentration (min),

A = area of catchment (m^2),

H = average fall (m per 100 m) from the summit of catchment to the point of design, and

L = distance in metres measured on the line of natural flow between the design section and that point of the catchment from which water would take the longest time to reach the design section.

The nomogram in Figure 8.1 may be used for rapid solution of equation 8.2, an example of the use of the nomogram being given on the Figure. A minimum time of concentration of one minute should be used. The time of concentration is required for the determination of the design intensity of rainfall.

Special attention should be paid to catchments where the stream course has been channelled and straightened. In such cases, the time of concentration will be much shorter than that indicated by the Bransby-Williams equation; it should be calculated by adding the time of travel within the drainage channel during peak flow to the time of concentration calculated from the Bransby-Williams equation for the most remote subcatchment to the drainage channel.

For slopes constructed in urban areas, the determination of the peak flows from areas above the slope may be complex when the effects of existing drainage systems within the natural catchments are, as they must be, considered.

8.2.4 Design Intensity

Because mean rainfall intensity reduces with duration, the greatest runoff occurs when the duration of the storm is equal to the time of concentration. The maximum intensity for any given return period, which is a measure of the frequency of occurrence, may be determined by reference to intensity versus duration rainfall curves for the area considered. Figure 8.2 gives the intensity versus duration curves from Royal Observatory and King's Park data, as reported by Peterson & Kwong (1981) based upon original work by Cheng & Kwok (1966). For many years, these curves were considered to be representative of the whole territory. Recently, the Royal Observatory has advised that their validity in different locations should be confirmed by undertaking an independent analysis of data obtained from the nearest autographic raingauge station to the location of interest. A map of Royal Observatory and Geotechnical Control Office raingauge stations is given in Figure 8.3. At present, there is no information on the magnitude of deviations from Figure 8.2 for other locations in the Territory. For routine slope drainage design in small catchments, the additional work required to analyse the nearest raingauge data is not considered to be justified, and continued use of Figure 8.2 is recommended. However, for the design of major drainage structures such as culverts or nullahs, it is recommended that the procedure of analysing additional rainfall data from as close as possible to the site should be adopted.

The design of all drainage works on steep slopes, the stability of which could be affected if the drains cannot carry the runoff, should be based upon a two hundred-year return period storm. Temporary drainage for use during construction should be designed on the basis a ten-year return period storm although, if the construction period extends over several wet seasons, the design parameters should be greater.

8.3 DETAILED DESIGN OF DRAINAGE SYSTEMS

8.3.1 General Design Considerations

In view of the cost and maintenance considerations, the number and length of surface channels should be kept to a minimum for the design of new slopes. The requirement for surface drainage is heavily dependent on slope geometry. Wherever platforms or berms are incorporated in the design, surface drainage will be necessary to ensure that ponding and infiltration, or localised erosion at low points, is prevented. Benches cut in hard rock may be an exception to this rule, but problems of concentrated flow at low points causing damage further downslope should be carefully assessed before deciding to omit surface channels. When new channels are designed to discharge through existing outfalls, the capacity of the outfalls and the risk of erosion damage beyond the site limits should be checked as part of the design.

Trash grills and sand traps should not be provided above or on slopes. In the case of slopes susceptible to surface erosion, it is prudent to construct a sand trap at the slope toe or other locations convenient for inspection and maintenance. Runoff in the channel should then pass over the sand trap and through a trash grill to prevent material from entering and blocking any storm drains into which channels discharge. As trash grills can rapidly become choked by twigs and floating debris, grilled openings should be large enough to pass the maximum flow when partially blocked. For normal situations, 50% of the opening should be assumed to be blocked and, for slopes below areas covered with dense vegetation, 75% is probably more appropriate. Figure 8.4 shows a typical sand trap arrangement. It should be noted that strict adherence to design note 5 on Figure 8.4 may lead to sand traps of such large capacity as to be impractical in many cases. This note refers to an acceptable design capacity for average flow conditions and is for guidance only. For slopes in erodible materials in steep terrain, the land area available for constructing sand traps may severely restrict the capacity, in which case the need for regular maintenance should be stressed. If space is so severely restricted that provision of a sand trap is impossible, an alterative is to construct a sumped catchpit at the toe of the slope channel, which should be accessible for manual or mechanical de-silting.

Streams can carry large rocks or boulders during heavy rain, which may block or damage the slope drainage system. If the stream course is strewn with rocks and boulders, a rock trap (an example is shown in Figure 8.5) should be provided, together with suitable access for maintenance. Blockage or damage to channels can increase local infiltration which, if not checked in time, may bring about progressive failures to a large area.

Instructions regarding the inspection and maintenance of silt and rock traps and grills should be included in the designer's handing-over notes (see Chapter 11).

8.3.2 Layout of Slope Drainage

Runoff should be conveyed by the most direct route away from vulnerable areas of the slope, particularly from behind the top of slope. Runoff should be led down the larger slopes in several stepped channels and should not be concentrated into only one or two. Streams intercepted by a slope should be conveyed directly down the slope. Any change in direction needed to rejoin the stream course should occur at the toe of the slope.

On slopes susceptible to erosion, a system of chevron drains (as shown on Figure 8.6) is recommended. On rock, chunamed, stone-pitched or other slopes not susceptible to erosion, a system of berm and stepped channels should be used where berms are incorporated in the slope design. If such slopes are constructed without berms, surface drains are normally provided only at the crest and toe of the slope, although drains with irregular spacing and direction are often constructed on small benches on the slope face during remedial works to large failure scars.

8.3.3 Types of Channel

Channels for slope drainage should be open concrete-lined U-channels or half-round channels. Pipes should not be incorporated in slope drainage systems.

Concrete-lined channels backfilled with free-draining stone, or unlined trenches containing porous pipes surrounded with free-draining stone (known as French drains) should not be used. In the former case, the carrying capacity of the channel is seriously reduced by the presence of the stone. In the latter, the stone surround in the unlined trench creates a source of infiltration, the porous pipe not conveying any water until the ground in which the trench is excavated is incapable of accepting any more infiltrating water. In both cases, the surface of the stone eventually becomes blocked with organic debris and topsoil in which grass and other vegetation can take root. The system then becomes ineffective as a drain.

Cut-off drains can be used at the top of a slope to intercept some of the water flowing into the slope from infiltration further up the hillslide. These drains are most effective where there is a shallow impermeable layer. The drain should be excavated upslope of possible failure surfaces to avoid acting as a tension crack (see Chapter 4).

8.3.4 Channel Design

The minimum gradient for channels is determined by the velocity of flow sufficient to remove silt. The velocity should not be less than 1.3 m/sec for the peak flow occurring with a frequency of at least once in two years.

The channel size, which depends upon the gradient adopted, may be determined from Figure 8.7. Channels larger than 600 mm may be designed using the charts developed by Ackers (1969). Alternatively, assuming a maximum permissible velocity of 4 m/sec and a roughness factor of 0.013, Manning's formula can be used :

$$V = \frac{1}{n} [R^{0.67} S^{0.5}] \qquad \ldots\ldots\ldots \quad (8.3)$$

where V = velocity (m/sec),

n = roughness factor,

R = hydraulic mean depth = $\frac{A}{P}$,

A = wetted cross-sectional area (m^2),

P = wetted perimeter (m), and

S = gradient of channel

Stepped channels are not particularly effective as energy dissipators. However, there would seem to be no practicable alternative. The flow in stepped channels is turbulent, and sufficient freeboard must be allowed for splashing and aeration. The stepped channel details shown in Figure 8.8 make some allowance for splashing and are the most effective used in Hong Kong to date for reducing the velocity of flow. In the absence of any experimental data, the size of the stepped channel and gradient of the invert may be determined using Figure 8.7 by assuming a velocity of 5 m/sec through the minimum section (at the top of each step). At the top of slopes, the velocity is lower and the cross-sectional area of flow greater, but splashing and aeration is less. Therefore, the section adopted for the stepped channel may also be used to cross narrow berms.

8.3.5 Changes in Direction

At any change in direction, the pattern of flow in the channels is affected. Channels in which the velocity is approximately 2 m/sec should change direction through bends of radius not less than three times the width of the channel. This radius should be increased where the velocity is greater than 2 m/sec or, alternatively, sufficient freeboard should be provided to contain the superelevation of the water surface (Chow, 1959).

Where a stepped channel crosses a berm, a hydraulic jump may form which must be contained within the channel. The splash allowance provided for the stepped channel may therefore be extended across the berm.

8.3.6 Junctions of Channels

Junctions of channels pose the greatest problem when designing slope drainage. They inevitably cause turbulence and splashing, and any chamber constructed to contain this is vulnerable to blockage by debris. Avoidance of such chambers is recommended, except possibly at the base of the slope, where the deep channels required to contain the splashing would cause a hazard if left uncovered.

At junctions, the smaller channel or channels should be brought in at half the width of the main channel above the invert of the main channel. The channels should be deepened with an added freeboard allowance to contain the turbulence, splashing and backwater effects.

Where channels are to discharge into a stepped channel crossing a berm, they should be curved into the stepped channel.

If excessive splashing and turbulence is expected at a particular junction or change in direction, consideration should be given to providing a baffle wall, as shown on Figure 8.9, or a catchpit, as shown on Figure 8.10.

The tops of all channels should be flush with the slope surface (Figure 8.11). Where possible, an apron that drains towards the channel should be provided to return any splashing to the channel. This applies particularly to stepped channels (Figure 8.8).

All surface channels into which subsurface drains discharge should be designed to prevent the subsurface drainage outlets from becoming drowned. It this cannot be done, a separate system should be designed for the subsurface drainage outfalls.

8.4 VEGETATIVE PROTECTION MEASURES

8.4.1 General Design Considerations

Vegetation on man-made slopes aids erosion control and serves important landscape functions. Because of these attributes, the use of vegetation for slope protection is increasing in Hong Kong. The effect of vegetation on slope stability is a complex interaction of mechanical and hydrological factors that are difficult to quantify. As yet, there are no firm design rules, but in recent years there has been rapid growth in local experience with a variety of vegetation types, and this knowledge can be of valuable assistance to the designer.

There are certain conditions that make it difficult to establish and maintain an effective vegetative slope protection, and these must be carefully considered in design. Nevertheless, the designer should consider vegetation as the primary protection method for all engineered slopes in soil and weathered rock (material decomposition grades IV, V and VI). Rigid protective measures should only be employed if specific requirements for slope angle, location or maintaining low infiltration rates preclude the use of vegetation. Even in these instances, some combined rigid and vegetative treatment may be possible and advantageous.

8.4.2 Vegetation Succession

Recent use of vegetation on man-made slopes in Hong Kong has concentrated on grassing, rather than shrub or tree planting. As a general rule, a single species of vegetation should not be planted in isolation. On natural slopes in Hong Kong, vegetation exhibits a natural succession towards broadleaf mixed woodland as the climax condition. To achieve climax vegetation cover on engineered slopes in a relatively short time, shrubs and trees should be planted in addition to grass. Consideration should also be given to appropriate vegetation management techniques to assist the natural succession process.

8.4.3 The Effects of Vegetation on Slopes

It is widely appreciated that slope vegetation is beneficial in terms of erosion control and its contribution to landscape quality. However, the overall effect of vegetation as a slope stabilising mechanism cannot be easily categorised as adverse or beneficial. The effects of vegetation on slope stability are listed in Table 8.1 and are broadly classified as either hydrological or mechanical factors. It should be appreciated that the net effect of the hydrological factors is difficult to quantify precisely, as it depends on the complex interaction of many elements of the hydrological cycle on, above and below the ground surface (Kirkby, 1978).

The results of detailed research studies carried out elsewhere provide support for the use of vegetation as a net beneficial mechanism, primarily through the effect of root reinforcement (Prandini et al, 1977; Gray, 1978; Gray & Megahan, 1981; Gray & Leiser, 1982). The traditional view in Hong Kong has been that the need to limit infiltration capacity is the overriding consideration, and this has resulted in the widespread use of chunam or other rigid protection measures. With growing awareness of the unpleasant environmental aspects of chunam, it is clear that detailed local research on the slope stabilising effects of vegetation is required.

Until such time as further data from local studies are available, it is recommended that designs for new slopes that include vegetative surface protection should allow for direct infiltration while neglecting the beneficial effects provided by the vegetation. For stability analyses of existing slopes that have an existing shrub and tree cover, it may be appropriate to investigate and quantify the beneficial factors given in Table 8.1. Guidance on suitable methods for assessing these factors is given by Gray (1978), Schiechtl (1980) and Gray & Leiser (1982). Where it can be clearly demonstrated that direct surface infiltration is a critical control of stability (as opposed to infiltration in the catchment area above the slope), and stability cannot be improved by flattening the slope, then a rigid surface protection should be considered in order to reduce infiltration.

8.4.4 Limitations on Slope Vegetation

Successful establishment of vegetation on a newly-formed slope is governed by several factors related to the time of planting and the steepness, location and material composition of the slope.

The planting season in Hong Kong is generally related to higher temperature and rainfall, as shown in Figure 8.12. The optimum planting season for grass is March to May. However, planting between early February and late September will generally be more successful than planting done in the winter months. The optimum planting season for shrubs and trees is April to June, but this can similarly be extended from early March to late August if necessary. Should planting be required outside the planting season, provision should be made for frequent watering. Alternatively, the planting could be delayed until the following season, with temporary erosion control measures employed in the interim period.

The steepness of the slope has a major effect on the effort required to establish vegetation. Some general guidelines are given in Table 8.2. Where vegetation is planned on particularly steep slopes or difficult sites, advice should be sought from foresters, horticulturists or landscape architects. It may be advantageous to select tree species of low mature height for planting on the steeper slopes to reduce the risk of uprooting in typhoons. On slopes located in a restricted urban setting, it may be desirable to plant only grass and shrubs to limit the height of the vegetation.

Slope-forming materials may also have a direct effect on establishing slope vegetation, as the fertility, moisture availability and resistance to root penetration are all determined by the composition of the materials. As a general rule, routine planting techniques may be used for all types of vegetation in soil and completely decomposed rock. In less weathered material, establishment becomes increasingly difficult, especially for shrubs and trees, and consideration should be given to the use of special techniques such as planting in formed pockets or benches on the slope backfilled with soil. Trees should generally not be planted on steep rock slopes where root-wedging along joints could cause instability. All soils and rocks in Hong Kong should be regarded as relatively infertile, and appropriate fertilisers should be added at the time of planting, as discussed in Chapter 9.

With regard to slope location, protected north-facing slopes are generally easier to vegetate compared to the more severe south-facing exposures. Vegetation is adversely affected by exposure to salt water, severe air or water pollution, erosion from wave action or running water and pedestrian traffic. If present, these factors should be controlled or alternative types of surface protection should be considered.

8.4.5 Vegetation Species

Planting on newly-formed slopes should commence with grass, and a particular selection of species should be made to achieve a vigorous covering capable of controlling surface erosion. Low maintenance requirements, low fire hazard in the dry season and compatibility with other slope plantings are other desirable factors that should also be considered in species selection. As there are no universally applicable grass species for all Hong Kong slopes, expert advice is often required to achieve specific project objectives. A list of the most commonly used grass species is given in Table 8.3. Planting techniques are discussed in Chapter 9.

Shrub and tree planting should commence after the grass has become established, so that a stable vegetative cover can be attained. Planting should be carried out in the months of March to August after several days of wet weather (Figure 8.12). Considerable local experience with shrub and tree planting has been gained recently, and the most successful species are listed in Tables 8.4 and 8.5. While all the species listed have proved to be acceptable and successful on both cut and fill slopes in Hong Kong, there are considerable differences in their habits and tolerances to extreme site conditions, as shown in the Tables. Planting stock is available locally for all the species listed. The Tables are not intended to be exhaustive or exclusive, and other species may be suitable for particular sites.

Slope plantings should generally consist of both shrubs and trees, and include a mixture of species. Typically, a planting programme might consist of 75% trees and 25% shrubs, with particular species limited to not more than 30% of the mix. Trees are usually planted as seedlings or whips, as discussed in Chapter 9. Shrubs should generally be planted in a grouped pattern rather than randomly dispersed among the trees.

The assistance of a landscape specialist can be very helpful in drawing up a planting programme. On difficult or unusual sites, such as those in a marine environment or any site where planting design is important, the advice of landscape specialists should be sought early in the project. Some suggestions for such sources of information are given in Chapter 12. Further guidance may be found in the publications by Schiechtl (1980), Gray & Leiser (1982) and the Urban Services Department (1971, 1974, 1977).

8.5 RIGID PROTECTION MEASURES

8.5.1 General Design Considerations

The primary uses of rigid surface protection on slopes are to reduce rainwater infiltration and to prevent erosion of the slope-forming materials. By reducing infiltration, rigid surfacings can contribute to slope stability either by preserving soil suctions in partially saturated material or by restricting the development of positive pore water pressures on potential failure surfaces. To achieve these functions, the following three aspects of the surfacing material must be considered.

(1) Permeability. The lower the permeability of the surface protection, the greater will be its effect in minimising infiltration. The effect of rigid surface protection in limiting infiltration can be assessed by measuring the change in degree of saturation before and after rainfall, which is dependent on the permeability of the surfacing material relative to the underlying soil. This aspect is considered in more detail in Chapter 4.

(2) Durability. The effectiveness of the protection can be reduced considerably by cracking and spalling of the surfacing material due to temperature changes, wetting and drying effects or poor bonding of the protection to the underlying material. There is, however, very little information on the design lives of various types of rigid surfacing. Much depends on the thickness of surfacing as well as the type of material and quality of workmanship during construction.

(3) Strength. The strength of the surfacing material is important in resisting erosion by running water. It also indirectly affects durability and permeability. This property is most commonly adopted in the material specification because it is easy to measure.

In view of the unsightly effect of large areas of rigid surfacing on the surrounding scenery, it is recommended that the most satisfactory type of surface protection is a vegetative cover, and this should be adopted whenever possible. In cases where a rigid surfacing is required, its adverse impact on the environment can be reduced by planting trees in rings formed on the surface, as described in Chapter 9. As a general rule, the area of rings for planting should be limited to a small percentage of the slope area to maintain the integrity of the surface cover and to minimise infiltration. The rings should be spaced fairly evenly over the whole area of the slope. Trees with shallow, spreading root systems which may cause cracking of the rigid cover should generally be avoided. Specialist advice should be sought for particular planting design requirements.

8.5.2 Chunam

Chunam is a common type of surface protection used on slopes constructed in Hong Kong. It is now declining in popularity as a permanent slope protection measure for environmental reasons, but is still used extensively for certain slopes such as temporary slopes on construction sites and steep existing slopes.

The typical thickness of chunam surfacing is 40 to 50 mm for permanent works but may be slighly reduced for temporary measures. It should cover the entire slope surface to be protected except where openings are required for drainage weepholes or vegetation planting beds (see Chapter 9). It is good practice to extend the chunam beyond the slope crest for a short distance to join up with a surface channel designed to intercept runoff from above. This will reduce surface infiltration immediately behind the chunam surfacing and provide safe working space for the maintenance of the surface channel.

Where water is observed seeping from the surface to be protected, or where water seepage may be expected to develop as a result of heavy rainfall (e.g. below valleys truncated by the cut slope), appropriate drainage measures should be provided to prevent the build-up of water pressures behind the chunam. The current general practice is to provide weepholes of 50 mm diameter at 1.2 m centres in each direction. It is, however, recommended that such drainage measures should be located by inspection of the slope surface prior to chunaming, and not applied indiscriminately.

It should be recognised that the effectiveness of chunam in preventing infiltration decreases with age and, for high risk-to-life slopes, the surface cover should be inspected periodically for signs of deterioration and replaced as necessary (see Chapter 11).

8.5.3 Sprayed Concrete (Shotcrete)

Slope protection applied by spraying mortar onto the surface of a slope is an alternative to chunam plaster and, for large areas of slope surface, application of sprayed concrete may be quicker and cheaper than chunam. It has been reported that shotcrete with a low water/cement ratio is relatively impermeable if properly applied. However, gunite (shotcrete containing fine aggregate) is less permeable than conventionally-placed mortar (Tynes & McCleese, 1974).

Design considerations for spray concrete are similar to chunam. In Hong Kong, shotcrete is most commonly used on rock cut slopes. In such cases, it is particularly important that drainage weepholes should be located at rock joints showing evidence of seepage.

8.5.4 Masonry

Various methods of slope surface protection incorporating masonry blocks are widely used in Hong Kong. These range from dry random rubble blocks to mortared coursed masonry. The most common type is a mortared random rubble facing (stone pitching) with typical thicknesses of 200 to 300 mm. A thin layer of concrete is often provided as a backing to the stone blocks. These are the most durable and effective type of surfacings for erosion control. Recent studies of masonry retaining structures by the Geotechnical Control Office have indicated that good-quality stone pitching acts as a retaining wall and therefore helps to maintain slope stability by providing a measure of slope support. However, this effect has not been studied in detail and should not be considered in the design of new slopes or the assessment of stability of existing slopes with stone pitched covers. It is good practice to bed the masonry on a layer of free-draining material and to provide weepholes, both at the base of the structure and between blocks at points where seepage is observed or suspected.

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9. CONSTRUCTION

This chapter deals with those aspects of construction that influence the safety of slopes, or that are peculiar to and affect the safety of structures that maintain slope stability.

9.1 CONSTRUCTION CONTROL

The importance of the designer being adequately represented on site throughout the construction period cannot be overstated. This is particularly true for geotechnical engineering, the success of which depends largely upon the experience of the geotechnical engineer or engineering geologist supervising the works on site. The designer is failing in his duty if he does not advise his client that it is inadequate to rely purely upon the obligations placed upon the contractor, under the terms of the contract, to guarantee that the quality of the permanent works and safety of the temporary works meet the intent of the designer. The extent to which the designer is represented on site will depend upon the size and complexity of the particular project. Site staff may range from one inspector of works on a small project to a resident engineer aided by several assistant engineers and other supervisory staff on a major project. Where fill is to be placed, the site staff should include persons experienced in carrying out the necessary field and laboratory testing.

Site staff should be familiar with the conditions assumed in the design. Should the actual conditions vary markedly from the assumed conditions, the attention of the designer must be drawn immediately to the changes. The designer should also arrange to visit the site frequently to ensure that conditions remain constant.

Site staff should keep detailed records of progress and of the conditions encountered when carrying out the work. Weekly progress photographs should be taken from positions agreed with the designer before work commences and should be supplemented as the need arises. Photographs should be dated, and descriptions of weather conditions, time of taking photographs and points of interest, particularly if not part of a series of routine progress photographs, should be attached. All tests that are carried out should be given a serial number, and details of the location, weather conditions, date and time of testing should be recorded. As-built drawings should be prepared as the work proceeds.

9.2 TEMPORARY WORKS

The Po Shan Road failure (Government of Hong Kong, 1972) demonstrated the critical effect that constructional activity can have on slope stability. Clearly, temporary works should be designed to minimise their effects on adjacent slopes and structures. In designing temporary works, due regard must be paid to conditions that may arise during their life. If works extend into the wet season, the design must include facilities for adequate drainage of the site and must make allowance for changes in groundwater level (e.g. the effects that changes in groundwater level have on the loads on strutting to excavations). The works must be carried out in a manner to ensure that the effects on adjacent slopes and structures are kept to a minimum.

The design and construction of temporary works must not significantly alter the conditions assumed in the design of the permanent works. If conditions are changed, the design of the permanent works must be amended to take account of the new circumstances.

Factors of safety adopted in the design of temporary work should reflect the risk posed to the public, the work force and the structure. If, for example, an occupied building could be threatened by failure of a temporary embankment, that embankment should be designed with the same factor of safety against slope failure as a permanent embankment. Where failure of a temporary embankment involves no risk to the public, the embankment may be designed with a lower factor of safety (see Table 5.1, Chapter 5). The consequent costs of failure should be weighed against the savings made in working to the lower factor of safety. A greater responsibility for the regular monitoring of the performance of the works and, when required, for their maintenance is placed on those supervising the project when lower factors of safety are adopted.

Throughout the construction period, all temporary works should be subject to regular inspections by an experienced engineer or technician. Signs of distress in any structure or slope should be recorded and steps taken to alleviate the distress. Where conditions are more severe than those assumed for the design of the temporary works, the design should be reviewed and the works modified accordingly.

9.3 SITE FORMATION AND BORROW AREAS

The formation of large flat areas on hillsides may cause a large increase in the infiltration of rain water into the ground. Before development, surface water tends to be shed rapidly from steep slopes so that infiltration lasts for a period approximately equal to the duration of the rainstorm. On flat surfaces, water ponding will increase both the amount and the duration of seepage into the ground. This, in turn, could lead to a rise in groundwater flow producing instability close to, or some distance away from, the point of infiltration.

The greatest risk occurs where a platform is left open without any surface seal or drainage, pending future development. Once development is complete, a large part of the formation is likely to be paved or protected so that infiltration is kept to a minimum. Where no immediate development is proposed, the surface should be sealed and graded towards drains to promote surface flow away from the surrounding slopes. Also, temporary bunds should be provided along access roads and at the edge of platforms. Thus, the design of these areas requires the consideration of local stability of the excavated slopes and the capacity and adequacy of drainage at all stages of development.

The factors of safety to be adopted in design should be decided in accordance with the future use of the area formed. When the responsible authority is unable to advise on the future use of such areas, it should be assumed that platforms adjacent to slopes are intended for residential purposes. If the future scheduled use is for recreation, a lower factor of safety may be appropriate since the area is unlikely to be in use during or immediately following heavy rain.

Areas of filling constructed as part of the formation or its access roads should be properly placed and compacted to the standards given in Section 9.5.

The importance of the visual impact of site formations on the environment cannot be over-emphasised. Landscaping proposals should constitute part of the design for the area and include a consideration of formation levels, landform, nature of the ground, drainage, surface protection and their combined effects on slope stability and the adjacent land. Vegetation should be used in preference to other forms of protective cover for slopes (Chapter 8).

The slopes should be closely inspected during construction to provide the necessary data and records for verifying the design assumptions. Piezometers should be monitored after completion of a site formation in order to provide post construction water levels.

The slopes and drainage system should be regularly monitored, inspected and maintained after the site formation is completed. Where responsibility for any area passes to another Office or Department, a report containing site investigation, laboratory testing, design and construction data should be formally handed over together with a clear statement of the extent and purpose of the monitoring to be maintained.

9.4 EXCAVATIONS

9.4.1 Programme

The programming of works involving excavations must take into account Hong Kong's heavy seasonal rainfall, which usually begins in April and ends in October. Excavations that result in high slopes and the associated drainage and surface protection should preferably start in October and be completed by April. If excavations are carried out during the wet season, the surfaces of the slopes formed must be protected and drained as excavation proceeds, drainage around the crest of the proposed excavation having been provided as a preliminary operation. Where necessary, temporary conduits should be provided to carry the discharge from those drains already completed. If trenches on or above slopes have to be excavated during the wet season, this should be done with extreme care. Ideally, they should be excavated and backfilled in short sections. Precautions should always be taken to prevent water collecting in the trench.

9.4.2 Methods

Excavation in the residual soil and completely weathered rock zones may be carried out using conventional bulk excavation methods, but care must be taken to avoid loosening the finished surface which may lead to severe surface erosion and siltation. Trimming should be carried out with light earthmoving equipment or by hand as appropriate.

In highly weathered granite, core boulders will be encountered. If these have to be removed, this should be done as excavation proceeds. They should not be left protruding from the exposed face until bulk excavation is completed, when access will be difficult and when their removal may threaten any development in progress below the formed slope. Boulders may be trimmed using plugs and feathers or hydraulic splitters. If blasting is to be used to trim them to the required slope profile, the amount of explosive should not be excessive, as over-blasting can loosen boulders sufficiently to allow them to be subsequently dislodged from the face of the excavation.

Where boulders threatening existing developments have to be trimmed, precautions must be taken to protect these developments during the trimming operations. This may be accomplished by means of wire nets or bamboo protective barriers, but their design must take account of the trajectories and impact of falling debris.

Excavation in the slightly weathered to fresh zones normally requires blasting which, if not properly controlled, can lead to shattering of the rock mass and to a general loosening and opening-up of joints. To avoid blast damage, rock may be presplit (Langefors & Kihlstorm, 1978; Hoek & Bray, 1981). Presplitting is accomplished by drilling a row of closely spaced and usually small diameter holes along the line of the final face. The height of a presplit face between benches should not normally exceed 15 m in order to maintain drilling accuracy. In Hong Kong rocks, a hole diameter of about 75 mm at a spacing of 700 mm has been found effective for presplitting, and this could be taken as a first approximation, but tests to determine the most effective system for a particular location should be carried out. These holes should be lightly charged with a cartridged explosive with a diameter less than half that of the hole drilled. The row of presplit holes should be detonated without any delay period between them and before the main excavation charge is fired. This should form a clean fracture plane along the final face to which the main excavation can break. Due regard should be given to ground vibrations in designing the size of the blasts. To further limit the possibility of damaging the final face, it may be necessary to reduce the burden, spacing, and exposive charge in the line of blast holes of the main excavation which are adjacent to the presplit face. The presplit fracture acts as a vent path for the explosion gases of subsequent blasting, but does not protect the rock behind the presplit row from vibration. Presplit blasting is not usually successful in well-jointed hard rocks where the joints are open and are inclined to the presplit line.

Smooth wall blasting or postsplit blasting, in which the line of holes is fired after the main blast, is an alternative method to presplit blasting. The method is often used to clean up faces that have been affected by heavy blasting but is generally less effective in producing a clean finished face than presplit blasting.

When excavation is complete, all rock faces should be scaled of loose blocks, even if blasted by either of the two methods described above.

During excavation, the designer must visit the site frequently to examine the exposed faces for signs of conditions that are more severe than those assumed in the design. Relict joints may be present in the weathered rock zones. Where these are seen in excavations, particularly if they are major joints dipping out of the excavated face at angles greater than 20° to the horizontal, they should be drawn to the attention of the person responsible for the design of the works. The shear strength of the material infilling relict joints is often considerably lower than that of the weathered mass, and instability can develop very quickly along these surfaces. Where rock is being excavated, such examinations should include a comparison between the exposed joint system and that assumed in the slope stability analysis and design. When a difference exists, the design should be checked and, if necessary, amended. Random joints, or combinations of joints that do not appear in the analysis should be surveyed and, if they form potentially unstable blocks, should be stabilised. Signs of seepage should also be noted during excavation and should be compared with assumptions on the location of the groundwater table made for the purpose of design.

Areas of over-excavation on slopes flatter than 1 on 1.5 may be made good with suitable fill compacted to 95% of British Standard maximum dry density (BS 1377, test 12). The surface on which fill is to be replaced should be benched, and fill should be placed in horizontal layers, with care being taken to ensure that the compaction of the fill at the surface of the slope meets the required standard. Where it is necessary to reinstate over-excavated slopes that are steeper than 1 on 1.5, cement-stabilised soil or concrete should be used with due regard being given to drainage of groundwater and seepage.

9.4.3 Effects of Vibrations

Constructional operations such as blasting, pile driving or the movement of heavy plant can cause ground vibrations and possibly air vibrations as well. Ground vibrations can have a detrimental effect on adjacent buildings, slope stabilisation measures and retaining structures and can cause discomfort to residents. Air vibrations can be in the audible range (20 to 20 000 Hz) or in the concussion range (below 20 Hz). Audible frequencies give rise to complaints, but air concussion can cause building damage, principally by the breaking of glass (DuPont, 1980).

Ground vibrations have been studied by Langefors & Kihlstorm (1978) and others, and a relationship between peak particle velocity and structural damage established (Hoek & Bray, 1981). Amplitude was previously used as a measure of blasting effects but peak particle velocity has been found to correlate better with the effects of ground vibration.

Current practice in Hong Kong is to limit peak particle velocity to 25 mm/s, or to 13 mm/s at particularly sensitive structures such as service reservoirs, radar installations, etc. Any proposal for blasting must be submitted to the Mines Division of the Labour Department for approval.

It is important to monitor vibrations during pile driving and blasting operations, and a good quality vibrograph (velocity seismograph) is required that is capable of measuring and recording in three mutually perpendicular directions. A detailed dilapidations survey should also be carried out on adjoining buildings prior to, and at regular intervals during, construction so that any damage can be assessed.

Ground vibrations and airblast during blasting can be effectively controlled by limiting the weight of charge per delay and by careful design of blasthole layout and firing sequence (DuPont, 1980).

9.4.4 Support

The design of strutted excavations is described by the Geotechnical Control Office (1982b). The method of analysis of the forces acting upon the support system assumes some degree of yielding and, as a consequence, movement of the supported material. It may not therefore be acceptable for designing the supports of excavations where movement could endanger the the stability of adjacent buildings or slopes and cause damage. The supporting system must be designed and constructed to prevent significant movements of the soil mass.

The support of excavations by various methods is discussed in detail in Dismuke (1975), who presents typical design calculations.

9.4.5 Drainage

The effect of rainfall in reducing the stability of slopes in Hong Kong is well documented (Lumb, 1962, 1975). The provision of adequate drainage on any major excavation is therefore of prime importance. Before excavation commences, concrete-lined drainage channels, designed as described in Chapter 8, should be constructed around the crest of the proposed excavation to collect runoff from above the excavation. If construction works are scheduled to extend into the wet season, temporary drains should incorporate silt traps suitably constructed to prevent seepage of water from the traps into the ground. Where silt traps are provided, they must be cleared regularly and the material removed from the trap deposited where it will not be washed into the drainage system during subsequent rainfall.

Ponding on the surface of the excavation should be prevented. In the case of excavations for foundations, basements and service trenches, this will be accomplished best by pumping from small, preferably concrete-lined, sumps, which will limit the amount of infiltration through the base of the excavation. The excavated surface of the general site formation and all temporary drains should fall to concrete-lined surface channels discharging to either a stream course or a stormwater drain. Drainage channels should be designed to the standards given in Chapter 8.

The construction of temporary works may cause a lowering of the groundwater table. As this can result in the settlement of adjacent structures, the possible effects of groundwater lowering, which will be more significant in the fine-grained marine deposits than residual soil and weathered rock, should be considered.

The siting of stockpiles of material, which will eventually be placed in the excavations, must not interfere with pre-existing surface water channels that maintain the stability of adjacent slopes. Although stockpiles may not affect adjacent works when first formed, the effects of erosion, due to rainfall and general construction traffic, can lead to the migration of material, and this should be borne in mind when they are sited initially.

9.4.6 Excavation for Drains, Channels and Pits

If possible, excavations for drainage works should not be opened during the wet season. If this is unavoidable, the excavation should proceed from the lowest to the highest point in the drainage system and should be carried out in sections. Each completed section should be lined before the adjacent section of excavations is opened up. This will prevent erosion and infiltration through the bed of the channel. Spoil from the excavation should be removed to a position where it cannot affect the drainage system.

9.4.7 Excavation for Services

Trenches excavated on or above slopes provide a location where infiltration of water into the hillside can eventually lead to slope instability. A trench cut into the toe of a slope can also undermine its stability and should not be permitted. Trenches loosely backfilled with soil will permit almost as much infiltration from the surface as an open trench and will permit the lateral flow of water along the trench through the backfilled material. Excavations for services above slopes should therefore not be opened up during the wet season unless unavoidable. When such excavations are carried out in the wet season, the trench should be protected against the ingress of runoff from the surface in which the trench is excavated by means of sand bags, concrete kerbs or small compacted earthfill bunds along each side of the trench. Pumps should be provided at all low points on the trench to maintain the bottom in a dry state, and a watchman should supervise the maintenance and functioning of pumps at all times when work is not proceeding. On completion of work, the trench should be backfilled in layers not greater than 150 mm deep, and each layer should be compacted to not less than 95% of British Standard maximum dry density (see Sections 9.5.1 and 9.5.5).

In the case of trenches for drains or ducts where pipes are not surrounded with concrete, the first layer of backfilling material should be placed and packed with care. This layer should be entirely free from large rocks or debris likely to penetrate or otherwise damage the service.

9.5 FILL

9.5.1 General

The disastrous landslides at Sau Mau Ping in 1972 (Government of Hong Kong, 1972) and 1976 illustrate the dangers inherent in slopes formed from loose fills being subjected to prolonged and heavy rainfall. On liquefaction, the material in the filling mass can travel great distances at high speed. Thus, where failure could endanger lives, temporary earthfill structures, even if for access roads and spoil heaps, should be designed and constructed to the same standard as that required for permanent works (see Chapter 5).

Wherever possible, construction programmes should be arranged so that fill is placed during the dry season, when the placement moisture content of the fill can be controlled more easily.

The basic requirements for all fills to be placed in or on slopes in Hong Kong is that they should, in general, be compacted to at least 95% of British Standard maximum dry density (see Chapter 3). In some exceptional cases, such as fills forming platforms that do not and will not support structures, the degree of compaction specified for some of the fill may be reduced to 90% of Standard, providing that the fill forming the peripheral slopes is compacted to 95% (see Figure 9.1).

Surfaces upon which fill is to be placed should be stripped of all trees, logs, brush, undergrowth, boulders, loose filling and debris of any nature. However, care must be taken to preserve and protect from injury all trees not required to be removed. The topsoil should be stockpiled for future landscaping needs. Tap roots and other projections over 40 mm in diameter should be dug out to at least a depth of 450 mm. The resulting holes should be filled with acceptable material and properly compacted.

Where there is very soft level ground, the use of geotechnical fabrics should be considered. Care must be taken, however, not to damage the fabric during placement of the first layer of filling. Alternatively, rockfill can be used to form a more stable surface on which further placement and compaction of fill can proceed.

It is recommended that the surface of the natural ground be scarified to a depth of 200 mm prior to compaction to the required density. Horizontal benches should be cut into sloping ground to a width satisfactory for operation of compaction plant. This will result in a series of steps up the slope as the filling increase. All obstructions should be removed to permit proper operation of compaction plant.

On large-scale filling operations, it is good practice to construct a test fill to determine the depth to which the fill can be placed and compacted to the required standard with the intended equipment. The results should be plotted as shown in Figure 9.2, and the appropriate depth and number of passes of the roller can then be estimated. This allows for easy construction control by a foreman or inspector of works, but control field density tests (see Section 9.5.5) must be carried out at frequent intervals to check that design densities are in fact being achieved. If the type or condition of the fill or the compaction equipment changes, additional tests should be carried out to ensure that the number of passes of the roller required to achieve the specified dry density has not changed from that defined as the result of the initial compaction trial.

The results and location of each test carried out as part of the routine quality control should be recorded on standard sheets (an example of a suitable record sheet is given in Figure 9.3). A record should be kept of the volume of material delivered to the site during each shift. Records should be clear and should describe the action taken in the event of a test failure.

9.5.2 Compaction of Rockfill

Rockfill should be predominantly hard durable pieces of material containing not more than 25% by weight of material classified as grade III and no material classified as grade IV, V or VI. Strong fresh granite is ideal rockfill material, as the particles are unlikely to shatter during placement.

Vibration is usually the most efficient method of compaction for rockfill. However, the method of spreading the material ultimately determines its effectiveness. Vibrations are transmitted through contact points between particles, and so the dampening action is greater where there is segregation and large voids within the fill.

9.5.3 Compaction of Earthfill

Earthfill should not contain boulders, rock or unsuitable materials such as topsoil, roots, tree stumps or rubbish. Lumps of fill should be broken up when the layer is being graded prior to compaction.

Compaction should be carried out as soon as possible after deposition. In no case should the rate of application of the filling exceed the capacity of plant based on its operational speed and compacting ability. Fill should be placed in such a manner that adequate drainage of the work area is maintained during construction. A properly graded and rolled surface sheds water and prevents ponding. Undulations in the general plane of the slope must not be permitted. Fill placed on the surface of an embankment should be compacted before closing the site for the night or at the onset of rain, so that delays caused by rainfall are limited. Fill adversely affected by rain should be allowed to dry out, or should be removed from the surface of the embankment, before filling recommences.

At embankment edges, spreading of the fill under the influence of the placing and compacting equipment makes compaction difficult. Embankments should therefore be built oversize and should then be trimmed back to the desired profile. The oversized section should be compacted to the same standard as the remainder of the embankment to ensure that the surface exposed by subsequent trimming has been adequately compacted. Density tests should be carried out in the exposed surface to check that the density of the surface material is as designed. Operations should be suspended where satisfactory results cannot be obtained. When making edge runs on fill, the compaction plant should be driven in a forward direction, uphill if practicable, with the reverse run being made well clear of the edge area.

Care must be taken during compaction of fill not to damage adjacent structures or buried services by the use of plant not suitable for the field conditions. Ingold (1979) has studied the effects of compaction on retaining walls and notes that excessive compaction can result in earth pressures exceeding those allowed for in design.

9.5.4 Compaction Control

The most effective method of quality control during a construction project is to establish all the required test data prior to the material being delivered to the formation. Particle size distribution, Atterberg limits, optimum moisture content, maximum dry density and any other required characteristics should be known before placement of material begins. The sampling of stockpiles is particularly important, and care should be taken in the identification of individual samples with respect to the location of such samples in the stockpile. Several samples should be taken from a stockpile to avoid acceptance or rejection of material based on the results of a single sample. Any evidence of a change in material should be immediately recorded. Changes in colour, texture, moisture content, particle size distribution, etc. are good indications that quality may not be satisfactory, and it is the testing officer's responsibility to report this information as soon as possible. Change in material quality is not necessarily reflected by a change of colour.

Where crushed products are being used, the chances of significant changes in quality are reduced and testing can usually be restricted accordingly. Furthermore, most crushed material should be produced to some type of specification, thus ensuring uniform quality.

Compaction should take place at or near the optimum moisture content (OMC), and strict job control is essential to ensure that this condition is achieved. Each layer of filling should be watered by proper sprinkling equipment which gives a uniform distribution of water throughout each lift over the whole area. Drying-out of the underlying compacted material should be prevented by periodic watering, covering with sheeting or further filling. If the fill material becomes too wet, dry material may be incorporated into it by mixing. Moisture control can be most critical in weathered volcanic rocks with high silt contents, and the range of target moisture content should therefore be restricted. Methods for measuring moisture content are discussed in Section 9.5.6; however, the optimum moisture content can be roughly estimated by squeezing a sample of the material. If the material crumbles, it is usually dry of optimum. If free moisture remains on the hands, the material is wet of optimum. Material that just holds together without free moisture is very close to optimum.

The behaviour of compaction equipment is also another means of assessing the amount of moisture in the fill. For example, if the fill is too dry, the surface of the layer will tend to break-up, especially under vibration. Too much moisture will result in sponginess under the action of the roller; high pore pressures will be generated, and any amount of additional rolling will have no effect.

9.5.5 Measurement of Insitu Density

(1) Sand replacement method. This method is described in BS 1377 (1975) (tests 15(A) and 15(B)). The two tests differ only in the size of equipment, the second being more suitable for coarse-grained soils. The method is accurate but requires considerable care. The calibration of the sand is sensitive to humidity and should be checked daily. The sand should be oven-dried and stored for about a week for the moisture content to reach equilibrium with the atmospheric humidity. If the sand is to be used again, it should be dried and sieved to remove any fill material before further use. The test should not be carried out when compaction plant is operating nearby.

(2) The densometer (ASTM D-2167 63T). The densometer allows a much simpler and more rapid determination of density. The equipment consists of a rubber balloon attached to a measuring cylinder that can be pressurised. The equipment is partially filled with water. A zero reading is determined by setting up the equipment on a smooth flat surface and pressurising the system until a minimum volume is recorded in the cylinder. A hole of a suitable size, with walls as smooth as possible and with a hemispherical base, is dug, care being taken to collect all the soil removed from the hole. The soil from the hole is weighed and a sample from which the moisture content can be obtained is taken. The equipment is then placed over the hole and pressurised, the minimum volume in the cylinder being recorded. The difference between the two recorded volumes gives the volume of the hole. From this volume, the moisture content and the weight of material removed from the hole, the density can be calculated.

This method is not suitable for testing very low-permeability clays nor readily-compressible soils where the test hole can be distorted by the applied pressure. The apparatus is cumbersome and prone to leakage.

(3) Core cutter method. A sample of the fill is taken using a thin-walled core cutter (BS 1377) test 15(D)) which can be either driven or jacked into the fill. The weight of the sample is measured and, with the known volume of the core cutter, the density is determined. The sampling process can change the density of the fill, and the presence of gravel in the fill can cause disturbance of the sample and give rise to errors in determining density. This method is not recommended for normal compaction control.

(4) Water replacement method. This method is suitable for determining the field dry density of a natural or compacted coarse-grained soil that includes gravels, cobbles, boulders and rock. The method is similar to the sand replacement technique but on a larger scale, using a plastic sheet to retain water in the excavated hole. It is generally desirable to perform a field sieve analysis on the excavated material.

9.5.6 Measurement of Moisture Content

(1) Standard methods. The best method of determining moisture content is by oven drying (BS 1377, test 1(A)) but this has the disadvantage of taking twenty four hours before a result is available. A decision on whether fill requires more compaction to meet the specification is therefore also delayed for up to twenty four hours.

Moisture content can be determined using a microwave oven, but this method requires individual calibration to ensure adequate soil temporature control. Samples may also be dried by spreading the soil over the bottom of a large metal tray that is heated directly by gas burner; however, heating is uneven. The result is obtained more rapidly than if the rapid methods described in BS 1377 are used (see below).

(2) Rapid methods. Two rapid methods for site use are given in BS 1377 (tests 1(B) and 1(C)). They are not as accurate as the standard method and cannot be used for soils containing large proportions of halloysite, gypsum, or calcareous or organic matter. They are, however, generally suitable for Hong Kong soils. Soil moisture content may also be rapidly determined using the Speedy Moisture Content Tester, which measures the pressure of acetylene gas released when carbide reacts with the soil moisture. However, this method is not sufficiently reliable for strict control.

Any rapid method used for construction control should be checked occasionally against the standard oven drying method, and a calibration curve should be drawn up showing the relationship between the moisture content determined by the two methods. The moisture content used to determine the dry density is the equivalent oven determined moisture content obtained by reference to the calibration curve.

Rapid compaction control can also be achieved using Hilf's method (United States Bureau of Reclamation, 1974). The fill is sampled using the same replacement technique: The result is adequate for compaction control but should be checked against densities obtained using the results of oven-dried moisture content tests. Present evidence indicates that this method may overestimate relative compaction. A typical field sheet is given in Figure 9.4.

Other methods, including the nuclear gauge method, require further research before they can be used with confidence in Hong Kong.

9.6 SURFACE PROTECTION

9.6.1 Grass

Grass can be an effective erosion control measure on slopes when combined with a system of surface drains. Grass also acts as a pioneer species in aiding subsequent establishment of more deeply-rooted shrubs and trees. To establish grass on a newly-formed slope, consideration must be given to actual slope conditions, grass species and season of planting, as discussed in Chapter 8.

Several techniques have been used in Hong Kong for planting grass. These include hydroseeding, sprigging, turfing and broadcast seeding.

(1) Hydroseeding. Hydroseeding is the application of grass seed, fertiliser and mulch in aqueous solution by spraying. A green marker dye is often added to the mix, and occasionally soil stabilising chemicals are added. The relative advantages of this planting technique are that a wide variety of seed species are available, and they can be quickly and cheaply applied to large and/or steep slopes.

A typical hydroseeding mix is shown in Table 9.1. The upper limit of application rates should be adopted on difficult sites. Follow-up applications of fertiliser are also recommended in addition to the initial application listed in Table 9.1. The follow-up fertiliser is normally applied at about one half of the initial rate. For difficult sites, two repeat applications should be made. Follow-up fertiliser should be applied no sooner than two months after hydroseeding, but should be applied within the growing season (March to September). As topsoiling is not usually done in conjunction with hydroseeding, compensations for particular soil deficiencies may also be considered in the fertiliser programme.

If there is potential for severe erosion on a newly-formed slope, then the upper limit of the mulch rate listed in Table 9.1 should be adopted and a temporary erosion control fabric used. The fabric should be placed over the slope after hydroseeding is completed. It serves to reduce raindrop impact and erosion while the grass is becoming established (a period of approximately six weeks). Care should be taken to select an erosion control fabric that serves this purpose without inhibiting the growth of the grass.

(2) Sprigging and turfing. Sprigging and turfing are the direct application of grass plants with developed blades and roots. Turfing implies that a continuous mat of grass is laid on the slope, while sprigging is the planting of individual or small groups of plants at 70 mm to 150 mm intervals. The advantage of these methods is the reliable results often achieved, especially when grassing must be done outside the normal planting season. The technique is limited by the slower application methods and the availability of planting material, both in quantity and species selection. Control of unwanted vines or weeds in the planting material can also be troublesome.

Topsoil is often placed on the slope prior to sprigging or turfing.

While this practice aids establishment of the grass, it increases the erosion potential of the slope until the grass is established. Any topsoil application should therefore be limited in depth on the steeper slopes, and consideration given to a more comprehensive fertiliser programme in lieu of topsoil.

Particular attention should be given to proper transportation and care of the grass stock prior to planting. Fertiliser may be applied at the time of planting (approximately 60 g/m² of NPK fertiliser; see note 4 of Table 9.1) and a follow-up application at one half the initial rate later in the growing season.

When sprigging or turfing is done on slopes steeper than 1 on 1.5, pegs should be used to fasten the plants on the slope. Erosion control membranes are seldom used in conjunction with sprigging or turfing but may be beneficial when there is a high erosion potential.

(3) Broadcast seeding. Broadcast seeding is the application of grass seed by hand or mechanically on a prepared soil surface. It is most often used as a repair or maintenance technique on small or isolated areas. Care should be given to evenly spreading the seed on a prepared soil surface or unreliable results can be expected.

9.6.2 Trees and Shrubs

Planting of shrubs and trees on slopes enhances slope stability and adds considerably to the landscape amenity. Seedling trees of one or two years of age have been successfully planted on slopes in Hong Kong. Shrubs have been planted less extensively, but they can provide an effective covering of vegetation where trees are undesirable because of their height, where pioneer vegetation is needed on difficult sites or where greater diversity of vegetation is desired.

Seedling trees or shrubs may be planted in pits excavated into the slope as shown in Figures 9.5 and 9.6. Pits excavated for planting should not be left standing open in the wet season. Fertiliser at the rate of 50 to 100 grams per tree should be mixed with the excavated soil before backfilling. Incorporation of peat moss, leaf compost or other organic matter will aid establishment. Seedling trees may be planted at approximately 1.5 m to 2 m spacings and should not be planted closer than 1 m to surface drainage channels or other utilities.

Certain shrubs can be successful planted as seeds, although these will take several years to become effective surface protectors. Trees and shrubs should be planted early in the growing season to assist establishment of roots and to minimise watering requirements, as discussed in Chapter 8.

9.6.3 Chunam

Chunam is a cement-lime stabilised soil used as a plaster to protect the surfaces of excavations from erosion and infiltration. The recommended mix for chunam plaster, the proportions being measured by weight, is one part Portland cement, three parts hydrated lime and twenty parts clayey-weathered granite or volcanic soil. The soil should be free from grass, roots, humus and other organic matter.

The cement and lime should be mixed dry before adding the soil. The minimum amount of water consistent with the required workability should then be added to the mix. If the water cement ratio is too high, severe cracking can result.

Before placing chunam on an existing slope all vegetation, topsoil and roots should be removed and the slope graded. To hold the chunam in position during placing, 25 mm diameter bamboo dowels 300 mm long should be driven into the surface at 1.5 m centres on a staggered pitch until only 25 mm of the stake projects from the surface. The chunam should then be applied to the surface in two layers, each not less than 20 mm thick. The surface of the base layer should be suitably scored with a trowel or left with a rough surface to provide a key for the second coat. This should be placed after the first coat has taken its initial set, but without an undue delay. Twenty four hours between coats would normally be considered sutiable. As the chunam is intended to provide an impermeable surface, it should be placed in as compact a condition as possible, and the final surface should be trowelled smooth to improve runoff.

Trowelled chunam can cause excessive reflection of sunlight and, as a consequence of the resulting glare, discomfort to anyone in the area. To reduce this effect, colourants may be added to the chunam in the top layer. Suitable colourants are manganese dioxide or ferrous oxide powder, which may be added to the mix for the top coat at a rate of 3% by weight of the cement content. The extent to which sunlight is reflected is dependent upon the smoothness of the finished surface, and final trowelling with a wooden float rather than a steel float may reduce the amount of reflection.

Chunam is normally placed with no regularly-formed construction joints, although care should be taken to ensure that the joints that do occur in the top and bottom layers do not coincide. When regular bays are formed, sealed joints should be formed between the bays. Chunam used to protect temporary excavation surfaces should be placed in one layer not less than 20 mm thick.

9.6.4 Sprayed Concrete

The general principles of the sprayed concrete method are discussed in the Earth Manual (United States Bureau of Reclamation, 1974), and a draft Code of Practice for the spraying of concrete is published by the Association of Gunite Contractors (1978).

Spraying concrete onto the surface of the slope is an alternative slope protection measure. The specifications for the materials used are identical to those adopted for conventional concreting, although the aggregates are specially selected to meet not only the requirements of the finished surface but also to prevent segregation while the concrete is being pumped. In general, the maximum grain size of the aggregate for slope protection should not exceed 10 mm. Mesh or steel fibre reinforcement can be provided where necessary.

In Hong Kong, careful consideration must also be given to the problems of drying and consequent shrinkage-cracking that can occur when sprayed mortar is used for slope surfacing. Having designed a mix to meet the requirements for segregation and strength, it should be used in a test panel prepared and constructed under the conditions that will obtain during slope surfacing. The performance of the panel with respect to durability, impermeability and shrinkage should then be assessed and, if necessary, the mix should be redesigned to meet all the requirements. The applied mortar should be cured for not less than seven days. The surface obtained by concrete spraying is rougher and therefore less reflective than chunam and, in practice, creates fewer problems of glare. Rebound material should be removed from the surface after completion of spraying.

Drainage facilities (such as weepholes) should be included in the sprayed surface, especially where seepages were noted prior to surface protection works.

9.6.5 Masonry

Various methods of slope protection incorporating masonry blocks, which are aesthetically pleasing, have been used in Hong Kong.

Masonry blocks should be bedded on a minimum 75 mm thick layer of free-draining crushed stone or gravel conforming to the criteria for the design of filters (see Chapter 4). Joints between adjacent blocks should be filled with a one part to three parts cement sand mortar to prevent infiltration between the blocks and the establishment of grass and other vegetation in the joints that would impede runoff. Weepholes draining the bedding material should be provided at the toe of the masonry wall.

9.6.6 Planting on Impervious Surfaces

To improve the appearance of large areas of chunam or concrete surfacing, selected planting of trees or shrubs is permissible in specially prepared beds. Such beds should be formed in the surface as shown in Figure 9.6. Sections 8.4.1 and 9.6.2 give other relevant information on this topic.

9.7 SERVICES

It is bad engineering practice to route ducts (pipes containing electricity or telephone cables) and conduits (pipes conveying water, gas or sewage) close to the crest of a slope. All possible steps must be taken to prevent leakage affecting the stability of the slope. As a general rule, all drains, services, and other pipes should not be placed in a slope nearer to the crest of the slope than a distance equal to its vertical height. This is a minimum standard, but each case should be considered on its own merits.

In cases where the proposed development cannot be modified to permit the siting of ducts and conduits outside this crest area, the slope should be designed to the factors of safety given in Chapter 5, taking into account the effects of possible water leakage. As an alternative, services can be housed within a sealed trench, ducting system or sleeve drained to a suitable discharge point at a surface drain or natural stream. The ducting system should be designed with a drainage capacity equivalent to a predetermined leakage rate. It is recommended that discharge from the ducting system be monitored at six monthly intervals.

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10. FIELD INSTRUMENTATION

10.1 INTRODUCTION

Instruments are installed in the field to provide information on specific parameters for design and construction control, and for long-term monitoring of performance. The most common use for instruments in Hong Kong is for the measurement of pore water pressure. Instruments that measure movement and stresses during and after the construction of slopes are also used, but to a more limited extent.

10.1.1 Planning Field Instrumentation

Careful planning and design of an instrumentation scheme is essential. This should include :

(a) definition of objectives, hazard warning levels and contingency plans,

(b) decisions on the type, number, location and frequency of measurements,

(c) organization of personnel, instrument purchase, installation, monitoring, data processing and reporting, and

(d) other factors, such as accessibility and the effect of the instruments on construction.

A flow chart that broadly identifies the different stages in planning an instrumentation scheme, based on work by Franklin (1977), is given in Table 10.1.

The number of instruments to be used in any scheme should allow for a percentage of instrument failures. When selecting instrument types to be used, the planner should attempt to incorporate cross-checks in the system by using different types of instruments rather than duplicating instruments of the same type.

Sensitive instruments tend to be more expensive, may have a restricted range and can sometimes be less reliable than those that are less sensitive. It is often more appropriate to install a greater number of cheap, reliable but less sensitive instruments when the potential behaviour of the slope or structure being monitored is unknown. It is important to judge an instrument system on its overall accuracy rather than on the quoted sensitivity of each individual component.

10.1.2 Calibration, Installation and Reading

Reliability under site conditions is usually more important than absolute precision, and it is essential that instruments can be calibrated not only prior to their installation but also from time to time during the life of the monitoring system. Where instruments are expected to remain in use for several years, the possibility of instrument drift cannot be ignored, particularly when electrical instruments are used. The complete system, including the detecting device, associated electronics and recorder, should be calibrated as one unit. All instruments, particularly those that are sensitive to weather and gravity variations, must be calibrated locally. Probe-type instruments, which are easier to calibrate, are unsuitable for situations in which continuous monitoring is required.

Instruments should always be installed by technicians who are fully conversant with the equipment and who have detailed knowledge of the factors influencing the performance of the equipment. Manufacturer's installation instructions are seldom adequate, and installation, particularly of complex instruments, is best supervised by an experienced instrumentation engineer. Instruments should be installed well in advance of the time they are required to monitor performance so that checks on drift and background noise level can be made and a datum established for subsequent observations. They should be well protected against corrosion, moisture, other aggressive agents and vandals.

The instruments should be read systematically by a suitably competent person who has an understanding of the purpose of each instrument. The frequency of reading will depend upon the situation and the nature of the changes that the instruments are being used to monitor. Readings are often most critical in inclement weather conditions. For example, piezometers in decomposed granites may require reading several times a day during and after heavy rain. Readings should be recorded on standard field sheets that include details of the probable range of readings from the instruments being observed. Any readings that indicate a marked change in conditions should be checked immediately. The functioning of the instruments should also be checked when unexpected readings occur, although in some cases a complete check may not be possible.

All instrument readings should be plotted on a time base so that the significance of the variation can be assessed more easily. Examples of typical plots are given in Figures 10.1, 10.2 and 10.3.

10.2 GROUNDWATER LEVELS AND PORE PRESSURES

10.2.1 Measurement of Groundwater Levels

Groundwater levels taken in investigation holes during drilling (Chapter 2) are not reliable due to the effects of the flushing water on the groundwater regime. For reliable observation, the groundwater level should be allowed to stabilise, and this may take several days after completion of drilling. The level of the water table can then be measured in an open borehole, but the response time is very slow and infiltration of surface water into the hole may cause the hole to act as a recharge well. As an alternative to leaving the casing in position, a smaller-diameter standpipe may be installed in the hole, around which the borehole may be backfilled with material removed from the hole. Observations of the groundwater level in such a standpipe are more reliable than those obtained during drilling, and standpipes should be installed in any hole that is not to be used for any other form of instrumentation. The standpipe installation can be improved by using a perforated pipe over the lowest 1 m to 2 m, and by backfilling the hole with filter material. The top 0.5 m to 1 m of the borehole should be filled with bentonite and cement to prevent infiltration.

A smaller standpipe will give a shorter response time than that of a standard borehole casing. However, if the hole penetrates more than one water bearing zone, flow between zones can occur, and water level measurements obtained from the standpipe will be meaningless. If the length of filter surrounding the standpipe is limited and is sealed to connect with one particular zone of soil or rock, the installation acts as a crude open hydraulic piezometer (Section 10.2.3) with a very slow response time. The installation of a suitably placed piezometer is however preferable, as the information obtained can be more easily interpreted.

10.2.2 Measurement of Pore Pressures

Piezometers are used to measure pore pressure. They consist of a cavity separated from the soil or rock by a porous element (the tip) and a device for measuring the water pressure in the cavity (Hanna, 1973; Vaughan, 1974).

The measurement of pore pressure requires that the piezometer tip be sealed into the ground, in the specific zone in which a knowledge of pore pressure is required, by placing it in a sand pocket in the borehole. Bentonite is used to seal the pocket above, and sometimes also below, and the hole is grouted. The length of the sand pocket should be at least four hole diameters long, preferably 600 mm to 1 m, and the grout used to seal the piezometer in the borehole should have the same or lower permeability than the surrounding soil.

A grout mix suitable for soils is one part cement to one part bentonite to six parts water. Where large water losses have been recorded when drilling in rock, this mix may require modification to avoid grouting large volumes of the rock. Grout volumes used should be checked and compared with the volume of the hole over the length intended to be grouted. Poor sealing of the piezometer in the hole will permit the migration of water from different levels, and may permit surface water to infiltrate into the piezometer. Readings taken from such an installation will, at best, indicate the level of the water table but, more probably, the borehole will act as a sump, and the readings obtained will be of no significance.

Bentonite balls formed from wet powdered bentonite, although difficult to handle and place, make a satisfactory seal in a borehole. Compressed bentonite pellets, which swell to many times their original size, are much easier to place than bentonite balls. However, they may artifically reduce the pore pressure for long periods in low permeability soils and, therefore, they should be used with care.

The choice of piezometer type depends on the predicted water pressures, access for reading, service life and response time required. Hong Kong soils are normally sufficiently permeable that response time need not be considered when selecting piezometers. The advantages and disadvantages of the different types of piezometers are given in Table 10.2.

10.2.3 Open Hydraulic (Casagrande) Piezometers

This consists of a porous tip, generally about 40 mm outside diameter and 300 mm to 600 mm long, with a standpipe normally 20 mm internal diameter (Figure 10.4). The top of the standpipe should finish above ground level and should be capped and protected from damage by a drained, locking cover box (Clayton et al, 1982). Beneath road surfaces and pavements, the piezometer must finish below ground level, and drainage of the cover box is especially important to stop surface water entering the piezometer.

The response time of this type of piezometer is comparatively slow, but the effect does not become serious until the soil or rock permeability is less than about 10^{-7} m/sec. At this permeability, the response time is about 1.5 hours for a piezometer with a 150 mm x 600 mm sand pocket and a 20 mm diameter standpipe. The response time is inversely proportional to the soil permeability. If the pore pressure temporarily drops below atmospheric, the piezometer discontinues reading but, being self-deairing, it resumes satisfactory operation without maintenance. Standpipes of less than 12 mm diameter may not be self-deairing (Vaughan, 1974). The long-term reliability of these piezometers is very good, except that they are rather more susceptible to vandalism than some of the other types.

An electrical dipmeter is the most commonly used method of measuring the water level. When the water level is fairly close to the surface, a simple method is to lower a small diameter polythene tube down the standpipe and blow down the tube. The change of resistance and bubbling that occurs when the end of the tube is submerged can be felt and heard.

The bubbler system for automatic recording is an extension of the second method. A small-diameter air line is taken to the piezometer tip, and a very small gas flow is passed down to it to produce several bubbles per minute. The pressure of the gas measured is equal to the height of water in the standpipe above the end of the bubbler tube. This method can be used for automatic recording of the piezometric pressure from a number of piezometers by using an electronic pressure transducer and a scanivalve system (Pope et al, 1982).

A common method of recording maximum transient water levels in piezometric tubes in Hong Kong is by using a string of buckets (Brand et al, 1983). These piezometer buckets (Halcrow buckets) are tied to a weighted nylon string at selected depth intervals above the normal base water level in the standpipe. The highest transient water level is read by drawing the string of buckets to the surface and recording the upper limit of filled buckets. The accuracy of water levels recorded in this way depends on the spacing of the buckets. It is recommended that they be placed 0.5 m to 1 m apart in critical piezometer installations.

Where artesian pressures occur, they may be measured using a Bourdon gauge or mercury manometer fitted directly to the standpipe or, alternatively, indirectly using a cap fitted with twin tubes thus forming a deairable closed-system.

10.2.4 Closed Hydraulic Piezometers

A twin-tube closed hydraulic piezometer with a low air entry pressure ceramic filter can be installed in a sand pocket in the same manner as a standpipe piezometer. Because head losses are negligible in a well installed system, hydraulic leads can be taken fairly long distances for remote reading, but a permanent gauge house is required if a Bourdon gauge or mercury manometer is to be used for pressure measurement. Portable pressure measuring systems using pressure transducers and portable deairing units, which eliminate the need for a gauge house, are available. In order to avoid cavitation, the pressure-sensing unit and the connecting tube must not be more than about 7 m above the measured piezometric level (Penman, 1978). On slopes, piezometer tips can be installed in holes drilled at an angle of 10° to 15° above the horizontal, but packers will then be required during grouting to ensure complete sealing of the hole.

The response time is quicker than that of an open standpipe, but is dependent on the method of pressure measurement. The system is not self-deairing, and regular manual or automatic deairing is required to maintain satisfactory operation.

10.2.5 Pneumatic Piezometers

This piezometer comprises a porous tip capped with a flexible diaphragm that controls a pneumatic valve. The valve operates when the pressure in the pneumatic system equals the pressure in the piezometer cavity. The piezometer tip can be installed in a sand pocket in the same way as a standpipe piezometer. As they cannot be deaired after installation (Clayton et al, 1982) pneumatic piezometers are of limited use in Hong Kong soils and should not be used where soil is unsaturated or where negative pore pressures exist. The long-term reliability of these instruments is not yet proven in Hong Kong.

10.2.6 Electrical Piezometers

Electrical piezometers are unsuitable for partially saturated soils, and therefore have a limited use in Hong Kong soils. They comprise a porous tip isolated by a flexible diaphragm, the deflection of which is measured by vibrating wire or resistance strain gauge (Hanna, 1973) Response to pore pressure change is rapid. They have the same deairing problems as the pneumatic piezometer, and the calibration, which is liable to drift (change with time), cannot be checked after installation.

Vibrating wire systems are more reliable than resistance systems. The major disadvantage of electrical resistance systems is drift. Instrument life of vibrating wire systems is moderate and of resistance systems is often very short.

10.2.7 Measurement of Pore Suctions

Suction is negative pore water pressure, relative to atmospheric pressure. In the range of zero to minus one atmosphere, direct measurement of pore suction can be made using a closed hydraulic system (Bishop et al, 1961).

Tensiometers are the most suitable devices for measuring pore suctions applicable to slope stability problems in Hong Kong. The measuring range of the vacuum gauge of a typical tensiometer is generally between zero and -80 kPa. This means that, if the ceramic tip is 3 m vertically below the gauge, the effective measuring range for the pore pressure at the tip is between 30 and -50 kPa. When measurement of suction at great depth is required, a caisson may be excavated and tensiometers installed through the sides of the caisson (Sweeney, 1982). The reliability of a tensiometer depends on a good contact between the soil and ceramic tip, and the complete absence of air from the system. During installation, therefore, it is important that this contact is achieved so that a preferential path for downward seepage of water does not form between the tensiometer tube and the soil.

A tensiometer cannot be used to measure suction greater than one atmosphere because of cavitation. For measurement of suctions larger than this, psychrometers may be used (Richards, 1971).

10.2.8 Location of Piezometers

The majority of slope failures in Hong Kong are shallow slips occurring along distinct geological boundaries. Infiltration from the ground surface can result in the development of perched water tables at the boundary between two materials of different permeability and can cause slope failure. It is therefore important to position piezometers at depths where there are changes in mass permeability so that perched water tables can be detected. Piezometers at greater depths are useful for determining the permanent groundwater level.

10.3 SURFACE MOVEMENTS

10.3.1 Significance of Movements

Landslides in Hong Kong soils often occur without warning and, for these, movement measurement is seldom useful. However, measurement of movements and deformations associated with retaining structures, anchor systems and soil and rock slopes can be important. Methods of measuring surface movement are discussed in Sections 10.3.2 to 10.3.5 and subsurface movement in Section 10.4.

10.3.2 Structural Cracking

A telltale placed across a crack should be designed to indicate if further movement of the crack has occurred since installation; they should also show magnitude and direction of the movement. All telltales should be marked with the dates of installation.

The simplest form of telltale, suitable for monitoring continuing movement of walls, comprises one or more straight lines scribed across a crack. Two lines at right angles will shown any movement in the plane of the face on which the lines are scribed.

Glass plates, mounted across a crack using mortar pats or epoxy resin, are common and are easily installed, but they do not indicate clearly the direction of relative movement. The glass often breaks either through shrinkage of the mortar pats on which it is set or vandalism; glass plates are therefore not recommended.

A more robust telltale, which can be used to measure progressive movement, is shown in Figure 10.5. Movement of the overlying metal strip can be measured relative to datum lines scribed on the underlying plate; these datum lines being offset by recorded distances when the telltale is installed. Movement normal to the face can damage the metal strip and prevent continued reading. To overcome this problem, a second telltale may be placed a short distance from the first, with the overlying strip bonded to the opposite side of the crack. By measuring the separation of the two plates with a feeler gauge, movement normal to the face can be determined.

Mechanical deformation gauges (e.g. demec gauges) can be used for accurate measurements. Two gauge points are attached to the structure on either side of a crack, their spacing being fixed using a special locating bar. The gauge comprises an invar steel beam with two conical gauge points, one fixed at one end of the beam and the other pivoting on a knife-edge. The deformation is recorded by a dial gauge, the movement being magnified by a simple lever system. Temperature compensation is achieved using an invar steel reference bar. Very high accuracies, which are probably higher than required for monitoring crack movement, can be achieved with this instrument. A vernier dial gauge used in conjunction with reference studies is a simpler method, the accuracy of which is usually adequate for this form of monitoring.

10.3.3 Rock and Soil Slopes

The methods discussed in Section 10.3.2 are also applicable to the measurement of movement on joints in fresh or slightly-weathered rock. Surface-mounted gauges are, however, unsuitable for moderately-weathered rock and weaker materials.

Steel pegs, grouted with epoxy resin into holes drilled in the rock on either side of a joint, can be used to monitor relative movement. Usually three pegs are used, two on one side of the joint and one on the other. On soil slopes where movement across a crack is to be monitored, the pegs should be set in concrete beacons on either side of the crack. The top of the pegs should be centre-punched, or preferably have conical seatings, for precise positioning of measuring instruments. The distances from one peg to the two pegs on the other side of the crack or joint are measured using a vernier caliper or a mechanical extensometer. The change in relative level of the pegs can be determined using a spirit level on a straight edge between the pegs.

Démec gauge seatings can be used as an alternative to steel pegs.

10.3.4 Surveying

Relative movements can be assessed using precise surveying techniques, and measurements of absolute movements can be obtained if a fixed datum located outside the zone of influence of the moving masses is used. It is important that the instrument error of the surveying method is less than the minimum required accuracy of the movement measurements. The movements measured are often very small, so the reference points used should allow accurate positioning of measuring instruments. Suitable reference points are described by Burland & Moore (1974), Cheney (1974) and Hanna (1973).

10.3.5 Photogrammetry

Measurements of movement can be made using stereographic pairs of photographs taken with either a phototheodolite or some other form of precise camera. A series of photographs is taken from two fixed stations on a base line parallel to the face or slope being studied. The camera axes should be parallel and normal to the base line. Where possible, fixed ground control points at the top and bottom of the slope should be installed, and at least two should appear on both photographs of each stereopair. If it is not possible to set up fixed ground control points, the points used for control must be accurately surveyed for every pair of photographs. The accuracy of the measurements of movement is dependent on the precision with which the ground controls are surveyed. Steep slopes are more suited to photogrammetric survey than shallow ones because the accuracy varies with distance from the focal plane of the camera.

Descriptions of both the technique and the equipment and its use in geological mapping are given by Ross-Brown & Atkinson (1972) and Ross-Brown et al (1973). Photogrammetry has been used to monitor movements of dams (Moore, 1973) and to map joint movements (Moore, 1974; Burland et al, 1977).

10.4 SUBSURFACE MOVEMENTS

10.4.1 Inclinometers

An inclinometer can be used to assess movements both of and behind retaining structures, and sometimes prefailure movements of fills and cut slopes. It comprises a tilt measuring instrument contained within a torpedo that travels in a grooved or square casing to maintain horizontal alignment. A common inclinometer uses a Wheatstone bridge to measure the angle of tilt, while others use photographic methods, vibration wire or resistance strain gauges. Inclinometers based on servo-accelerometers are increasingly being used for high precision work.

The most commonly used inclinometer casing in Hong Kong is aluminium tubing. The backfill between the casing and the surrounding soil, which is generally cement-bentotite grout, should be of the same stiffness as the surrounding soil. Wherever possible, the inclinometer casing should be installed in a pre-grouted drillhole with clean fresh water inside the casing to overcome its buoyancy during installation. The inside of the tube should be cleaned with fresh water once a month, and the pH of the water after flushing measured. If the pH of the flushed water is alkaline, implying that there has been leakage of grout into the tube or that corrosive alkaline agents are present, the tube should be cleaned until the water is neutral.

When inclinometers are installed to detect the movement of slopes, it is convenient to have the axis of the casing at the right angles to the slope face. The fixity of the inclinometer base should be confirmed by ground survey. Inclinometer readings should be taken on both faces of the instrument, and the average reading should be recorded. An example of an inclinometer record sheet is shown in Figure 10.6.

10.4.2 Slip Indicators

The position of a slip-surface can be obtained using simple instruments. One of the most commonly used slip surface indicators comprises two identical, relatively short lengths of rod within a flexible tube installed in a borehole, one length of rod being kept at the bottom of the hole and one at the top. The zone in which shear is occurring is found by raising the rod from the bottom of the casing and lowering the other from the top until the rods cannot pass through the casing. The same instrument can also become a crude inclinometer by using a series of rods of various lengths and assessing the curvature of the tube by finding the length of rod that can just pass down the casing.

Among other methods that are available is the shear strip which comprises a series of parallel resistances. The depth at which the shear strip fractures is determined by measurement of the resistance. A second type comprises a bubbler tube device connected to a thin glass tube within a larger diameter metal pipe. The air pressure required to continually pass bubbles into the ground decreases when shearing breaks the glass tube. The Japanese pipe strain gauge has foil strain gauges mounted in a PVC pipe, and deformation or shear can be monitoring by recording the change of strain in these gauges.

10.4.3 Extensometers

An extensometer comprises tensioned wires or rods anchored at different points in the borehole, and measures the movement of the anchor points, relative to a surface datum, along the line of the hole in which it is installed. Extensometers are most suited to measuring deformation of, and behind, retaining structures and in soils and rock stressed by anchoring or affected by excavation.

Relative movements between the face and the end of each extensometer rod may be measured with a dial gauge, or with a linear displacement transducer for remote reading. The wire of the more simple tensioned wire extensometer is taken over pulleys, and a constant tension is maintained with weights. Alternatively, the tension may be maintained by spring cantilevers in the measuring head and the deflections of the cantilevers measured by dial gauges or transducers.

In confined spaces, the protection of the tensioning system of wire extensometers is difficult. Rod extensometers with removable measuring heads are less subject to damage. Readings can be affected by thermal expansion of the metal and, for precise measurements, temperature corrections should be made.

10.4.4 Settlement Gauges

Settlement gauges (for installation in either fill during construction or boreholes), remote reading hydraulic gauges and profile gauges are all described by Hanna (1973).

10.5 LOADS AND STRESSES

10.5.1 Load Cells for Rockbolts and Anchors

Only load cells suitable for long-term monitoring of rockbolt and anchor loads are considered in this Section.

The anchor load can either be determined at intervals, by measuring the force required to jack the anchor head away from its seating, or it may be monitored continuously with a compression load cell between the anchor head and bearing plate. The types of load cell available and their advantages and disadvantages are given in Table 10.3.

10.5.2 Earth Pressure Cells

Measurement of earth pressure is unlikely to be required in soil slopes in Hong Kong. In large excavations, cells may be specified to measure the contact pressure between the soil and a retaining structure. Effective stress can be computed by installing pressure cells with piezometers. The type and position of a cell should be chosen with great care, because the introduction of the cell into the soil causes a redistribution of the stresses around it, and the errors depend on the geometry of the instrument. Details of the types of cell available and the problems that may be encountered when using them are given by Hanna (1973).

11. MAINTENANCE

11.1 INTRODUCTION

Regular inspections and maintenance are essential for the continued stability of well designed and well constructed slopes. While those responsible for design may not be able to do anything after completion of construction to ensure that recommendations for maintenance are followed, careful design and detailing can reduce both the amount of maintenance required and the physical labour involved.

When handing-over a development, the designer should provide a data base containing information of relevance to the slope. These data will form the basis of the slope maintenance records. The designer should recommend a programme of maintenance in accordance with Table 11.1. Such a programme should include guidance on maintaining and reading installed instruments and should indicate the probable range of readings that will be obtained if the structure is functioning as designed. If, during the lifetime of the development, the readings obtained from instruments indicate conditions more severe than those allowed for in the design, the owner should be advised to obtain specialist advice. Instrument readings should be kept on standard record sheets, an example of which is shown in Figure 11.1. The owner of the slope should appoint a maintenance officer. This officer would be responsible for ensuring that the maintenance inspections and recommended remedial works are undertaken.

It should be noted that maintenance should not be considered to include ground anchor monitoring, which is a specialist procedure, nor monitoring the condition of the slope to check the design assumptions, which should be the responsibility of the designer.

11.2 FREQUENCY OF INSPECTIONS

For slopes that have been in existence for many years, it may be necessary for the maintenance officer to screen all the slopes under his control to decide the frequency of inspections in view of the current consequence of failure. It is not reasonable that the inspection frequency should be related to the number of slopes to be maintained. As detailed in Section 11.3, inspections should be carried out at the technical officer level and at the engineer level. The recommended intervals between inspections at both levels are shown in Table 11.1. If frequent inspections are carried out and the follow-up preventive work is completed, there will be no need to attempt the difficult task of trying to undertake the maintenance inspections and preventive work in the brief period between the wet seasons.

11.3 INSPECTION REPORTS

It is essential that all inspections should be properly recorded and that a system should exist to transfer inspection recommendations into remedial works, preventive works, or detailed investigation. Where a data base exists, the reports should be added to produce an historical record of the slope. The inspecting officer should always check that the last set of recommendations have been fully implemented. Photographs should be used as records where appropriate. An example of three pages of a maintenance inspection record sheet is shown in Figures 11.2, 11.3 and 11.4.

11.3.1 Technical Inspections

The purpose of these inspections is to ensure that the slope is not deteriorating. These inspections may also be used to identify slopes that need to be upgraded into a higher risk-to-life category. It is important that guidance notes are issued to the technician. These would highlight such points as the fact that the crest and all berms must be inspected. Such notes should be the responsibility of the maintenance officer. It is important that worrying points should be brought to the attention of the maintenance officer. Routine repairs should be carried out without delay.

11.3.2 Engineering Inspections

It is not intended that the inspection by an engineer should solely check up on the technical inspections, although this should be a side benefit. The brief of the engineer would be largely to consider whether the risk-to-life category of the slope needs to be modified because of any changes in the slopes surroundings, for example, increased pedestrian traffic on an adjacent pavement. He should consider systems of slopes and retaining walls, causes of failures, and look for dangers that have developed since the design was completed. If an engineering inspection indicated that routine work was needed, then this would indicate that the technical inspections were not being properly carried out.

In cases where the slope is technically complex, has a history of failure, or the risk-to-life is high, the engineer might consult an engineering geologist or a geotechnical engineer. Where improvement works would require an investigation, then this should be made clear to the maintenance officer.

11.4 MAINTENANCE

11.4.1 Instruments

The instruments that are most likely to be installed and used for continued monitoring are piezometers (Chapter 10). The accuracy and serviceability of these instruments relies, among other things, upon preventing the ingress of water and foreign matter into the standpipe. The surface boxes for the instruments should be examined during each inspection. All standpipes should have tight-fitting caps, and all surface boxes should be drained. If necessary, caps should be renewed and drains cleared.

Other instrument installations should be checked to ensure that they are functioning under the conditions specified by the manufacturers.

11.4.2 Slopes and Slope Surfacing

All slopes should be examined for signs of movement indicative of slope failure. However, in other aspects, the inspection and subsequent maintenance required for a colluvium, fill or soil slope will differ from that required for a rock slope. As soil slopes rarely show signs of progressive failure (slips occur very quickly when slopes become saturated), inspection and subsequent maintenance should be principally directed towards preventing the infiltration of water. Forms of surface protection, other than grass, are generally brittle and therefore are susceptible to cracking. Inspection records should give details of crack positions, lengths, widths and relative movement. Telltales (described in Chapter 10) or gauge measurement points, should be installed on new cracks where appropriate. During inspections of grass-covered slopes, the positions, depth and extent of erosion scars should be noted.

Rock slopes can show signs of progressive failure by movement along joints. Where joints appear to be opening, telltales or measurement points should be installed to monitor progressive movement. Closely-jointed rock is likely to deteriorate generally and not show signs of movement along any one joint or series of joints. Colour photographs of such faces taken during each inspection will assist in assessing the extent to which the slope has deteriorated. Erosion around isolated blocks or boulders should be recorded.

During inspections, seepage traces on and adjacent to all formed slopes should be recorded. Flow from seepage sources, weepholes and horizontal drains should be recorded and, where possible, should be examined for signs of migration of solid material, indicating internal erosion. For the effects of cracked services on flow from these sources, see Section 11.4.3.

The development of vegetation on slopes, if allowed for in the design of the surfacing, can be beneficial both aesthetically and structurally. Rigid surface protection adjacent to trees should be examined for signs of deterioration as a result of root action, and repaired or replaced as needed. Rock slopes in the vicinity of trees or shrubs should be examined for blocks that have been destabilised by root-wedging.

Routine maintenance of all slopes should include the removal of undesirable vegetation. Cracked rigid surfaces should be repaired by cutting a chase along the line of the crack and filling it with material of a suitable mix. On inclined surfaces, it is good practice to cut the chase in a manner that prevents the ingress of water through any shrinkage cracks that may develop between the original surface and the repair.

Cracks in surfaces comprising masonry blocks set in and pointed with cement mortar will tend to follow joints between blocks. The affected joints should be cleaned and repaired.

Rigid slope surfaces that have been undermined by groundwater flow should be removed, the source of the flow identified, the flow stopped or taken to the surface by means of horizontal drains, and the surface made good.

Eroded grass slopes should be regraded, if necessary with fill compacted to a density not less than that required in Section 5.5.2. The fill should be compacted in horizontal layers and not in layers parallel to the slope. If necessary, the eroded area should be benched and graded so that fill is not placed against extensive vertical surfaces. This treatment will be required if the vertical eroded surfaces against which fill is to be placed are greater than 600 mm high. Before placing fill, all concentrations of free-draining material found in the eroded area should be removed. The repaired surface should be replanted with grass, protected from further erosion by a protective membrane until the grass is established, and fertilised as appropriate. Details are given in Chapter 9.

Rock slopes are not usually completely surfaced. They may, however, require local surfacing to prevent water from entering open joints. Care must be taken to ensure that, while the surfacing or capping prevents ingress of water, it does not prevent seepage. Such work should include the provision of weepholes where necessary.

Where there are traces of seepage from a slope in areas where weepholes are not provided, the source of the seepage should be investigated and adequate drainage provided.

11.4.3 Surface Drainage

Records should be kept of those features of the drainage system that require modification to reduce routine maintenance. Inspection records should include :

(a) position and extent of broken and cracked channels and catchpits,

(b) position and extent of silted-up sections of channels and catchpits,

(c) position and extent of deteriorating channels and catchpits, and

(d) details of construction works, possibly outside the boundaries of the development, from which mud and debris could migrate to block the drains of the system being inspected.

In addition to formal maintenance inspections, it is advantageous to pay particular attention to surface drainage systems in the wet season. If it appears possible that a drainage system can be blocked by soil washed from works on an adjacent site, preventive action should be taken. As discussed in Section 8.3.1, it is preferable to have trash grills and sand traps or sumped catchpits, before slope drainage feeds into other drainage networks. Material from these trash grills, sandtraps, and sumped catchpits, together with that from conventional catchpits and channels, should be removed regularly, and disposed of where it cannot be washed back into the drainage system during subsequent storms.

Cracks in channels should be repaired with cement mortar or with a suitable plastic sealing compound. If a channel cracks from settlement, the settled section should be removed and reconstructed in such a way that it is not susceptible to damage from further settlement. A mortar repair to a channel that is settling is only a temporary measure. Where channels are found to be settling, flexible sealants can be used to repair cracks, but major repair works should not be carried out during the wet season. Most flexible sealants, whether bitumen or epoxy based, require that the material on which they are to be placed is dry. The efficiency of the sealant will be severely limited unless the manufacturer's instructions are followed.

Where sections of channels have to be rebuilt, this work should only be done in the dry season when the existing channels may be safely removed. All reconstructed sections of channels should be built in accordance with the recommendations contained in Chapter 8. The reconstruction of channels to an increased capacity may require reconstruction of all channels downstream of the repair.

Pipes should not generally be used for surface drainage on slopes. Where pipes have been used and are found to leak or have to be replaced, they should, if possible, be replaced with channels constructed as described in Chapter 8.

11.4.4 Subsurface Drainage

The efficiency of horizontal subsurface drains will, in general, decrease with time. When interpreting the records of flow from horizontal drains, it should be remembered that any initially high discharges should decline as the drains lower the groundwater regime and a steady state is achieved. It is therefore important to record flows from individual drains during each inspection, and to correlate these with rainfall records for that area and with the readings of the piezometers which should be installed as an integral part of any subsurface drainage system. Where increased flows are recorded, the discharge should be examined for any signs that the water originates from leaking services. If these exist, the appropriate utility company should be notified and requested to trace and repair the leak. The discharge from drains should be examined for signs of migrating solid material, and chemical analyses should be employed, if necessary, to assist in positively dientifying the source.

If the piezometer readings indicate a rise in groundwater level and, at the same time, the discharge from the horizontal drains decreases, it should be concluded that the efficiency of the drainage system is decreasing. It may be possible to partially reinstate the system by flushing the drains with a suitably designed compressed air and water jet. In general, horizontal drains should be pressurised with water in an attempt to clear the filter material surrounding the drains as the resulting infiltration of water into the slope may cause slope failure. If flushing of the drains fails to raise the effectiveness of the system to an acceptable level, additional drains should be installed. Routine maintenance of an horizontal drainage system should include if possible the removal of obstructions at the outlets, removal of silt from the inside of the pipe, and cleaning or replacement of internal filters.

Drainage galleries should be examined for signs of structural distress. The locations and rates of inflow should also be recorded, and this should be compared with the total discharge. When increases in flow occur, which are not a direct result of rainfall, the discharge from the gallery should be checked for signs that indicate leakage from sewers and water mains. A comparison of previous records of the locations and rates of inflow may indicate the probable location of the leakage giving rise to the increased flow. Radial drain holes installed within the gallery should be inspected in the same way as horizontal drains. Flow measuring devices within the gallery should be checked to ensure that they are functioning properly and the measuring edge of V-notches should be examined for, and subsequently cleaned of, adhering debris or slime. Stilling pools behind V-notches should be cleared of settled sand and silt.

Should the gallery show signs of distress, expert advice should be sought. No remedial measures should be carried out until this advice has been obtained.

11.4.5 Services

Stormwater drains, sewers and water mains are the services most likely to affect slope stability. However, other conduits such as telephone ducts, electric cable ducts and disused pipes can also transmit water into slopes and reduce their stability. During routine inspections, all services should be examined for signs of leakage or water flow. Where this is detected, the service duct should be treated as described in Chapter 9. The inspection records should include a drawing showing the position and nature of all services in the vicinity of the slope. The appropriate organization should be requested to test water mains and sewers where leakage is suspected and where it could lead to instability of the slopes being examined.

11.5 ACCESS

Access should be provided to all berms, channels and drainage galleries to permit inspection and maintenance. For all new slopes, proper access should be included in the design. Lockable gates should be provided to prevent unauthorised entry and vandalism.

12. SOURCES OF INFORMATION

12.1 INTRODUCTION

This chapter summarises the main sources of information relevant to the broad field covered by this Manual. It deals with sources of geological and geotechnical information overseas, and with sources of geological, geotechnical, topographical, hydrological and meteorological information available in Hong Kong itself. It does not aim to be comprehensive as far as overseas sources are concerned, mention being made of only the main information centres that are most readily accessible from Hong Kong. A more comprehensive coverage is given to local sources of information.

The full names and addresses of all the organisations mentioned in this chapter are assembled alphabetically in Section 12.5.

12.2 OVERSEAS SOURCES OF GEOTECHNICAL INFORMATION

A number of information services in the fields of earth sciences and geotechnical engineering are available from overseas agencies. Some of these are comprehenisve in scope, whilst others are restricted to the supply of photocopies of known publications. Most of the major organisations involved in these activities were represented at the Workshop on Geotechnical Information Systems held in Bangkok in 1976, and the Proceedings of this Workshop (Brand & Brenner, 1976) should be consulted for full details of the services available.

12.2.1 Asian Information Center for Geotechnical Engineering (AGE)

AGE was established in 1973 to provide a worldwide low-cost geotechnical information service, with particular emphasis on Asia. It operates from the Asian Institute of Technology in Bangkok. The annual subscription entitles members to receive a number of regular publications. The Current Awareness Service (quarterly) contains a listing of the contents of all recently published journals, books, conference proceedings, etc. Asian Geotechnical Engineering Abstracts (biannual) contains abstracts of items published in Asia. The three volumes of Holdings Lists contain complete listings of the holdings of geological and geotechnical publications in the AGE collection. The quarterly AGE News keeps members informed of geotechnical engineering activities around the world.

Members of AGE also receive the AGE Digest, which contains in printed form the 30 000 entries stored in the computerised AGE data base. Any published item can be readily retrieved from the data base (or more slowly from the Digest) using the comprehensive system of keywords. The computer tapes containing the data base can also be purchased from AGE.

AGE provides bibliographic and reference services to its members. It will also supply by airmail copies of any published item held in the very extensive AGE collection to both members and non-members.

12.2.2 Geotechnical Abstracts and Geodex

Geotechnical Abstracts and the Geodex Retrieval System together form an integrated geotechnical information retrieval system that has the endorsement of the International Society for Soil Mechanics and Foundation Engineering. It operates on the basis of a simple 'peep hole' card system to retrieve the code numbers of abstracts of books, papers, etc contained in a series of printed booklets.

The abstracts for the system are produced by Geotechnical Abstracts, which operates under the aegis of the German National Society for Soil Mechanics and Foundation Engineering. Each month, 128 selected abstracts are published of items that have appeared worldwide. The abstracts are available as a paper edition for use with Geodex, or as a card edition for those who wish to maintain a conventional card file.

The Geodex Retrieval System has been in existence for many years. Prior to the commencement of publication of Geotechnical Abstracts in 1969, Geodex produced its own abstracts on individual file cards, and the system was very comprehensive in its coverage of the world's geotechnical literature. Since 1969, Geodex has continued to use its manual abstract retrieval system in conjunction with Geotechnical Abstracts. The System now uses 347 punched keyword cards, one card for each keyword used in indexing the Geotechnical Abstracts. Keyword cards selected as being appropriate to a particular geotechnical problem are simply stacked on top of each other to locate the abstracts of publications that satisfy the selected key works.

Both Geotechnical Abstracts and the Geodex Retrieval System can be obtained by annual subscription, either separately or together. Twelve issues of Geotechncial Abstracts are published annually. The Geodex card system is updated every four months, subscribers receiving a replacement deck of punched cards updated on the basis of the previous four months' Geotechnical Abstracts.

Since 1982, Geotechnical Abstracts has also produced a monthly publication entitled New Geotechnical Titles, which is a catalogued listing of recently published items. This serves as a 'current awareness' service, and it is automatically sent to anyone who subscribes to Geotechnical Abstracts.

12.2.3 Other Overseas Services

Other than those described above, there appear to be no other geotechnical information retrieval systems in the English language that are available internationally on a subscription basis. There are, however, some excellent libraries throughout the world that will provide copies of published papers on request.

The British Library has a very comprehensive collection of material, and a prompt airmail postal service is provided. They are linked with virtually all major libraries in Britain and are therefore able to locate almost any publication that is available anywhere in the country.

Other notable sources of geotechnical information in Britain are the British Geological Survey, the Geological Society of London and the

Institution of Civil Engineers. There are also many notable University libraries.

Good collections of geotechnical literature are held by the Norwegian Geotechnical Institute and the Swedish Geotechnical Institute. Apart from having probably the finest geotechnical engineering library in the world, the Norwegian Geotechnical Institute also houses the unique Terzaghi Collection which contains all of Terzaghi's original manuscripts and project reports, together with a large number of historical documents from other pioneers in geotechnical engineering.

Good geotechnical libraries are also maintained by the Laboratoire Central des Ponts et Chaussées, France and by the Division of Applied Geomechanics, CSIRO, Australia.

12.3 LOCAL SOURCES OF GEOTECHNICAL INFORMATION

12.3.1 Published Information Specific to Hong Kong

More than 600 items are known to have been published specifically on aspects of the geology and geotechnical engineering of Hong Kong. These are listed in the Bibliography on the Geology and Geotechnical Engineering of Hong Kong (Brand, 1984) produced recently by the Geotechnical Control Office and available upon request. A selection of the more important publications is contained in the Hong Kong Bibliography on page 159 of this Manual.

In addition to published material that is readily accessible to members of the general public, there is a large amount of 'unpublished' material available in Hong Kong that is valuable in the context of geotechnical engineering investigation and design.

The City Hall Public Library contains a reference section that contains a number of published documents on the geology and geotechnical engineering of Hong Kong, together with some unpublished reports. It also houses an Hong Kong Collection of considerable interest. Unfortunately, no direct access is permitted to the shelved items, and items required for examination must first be located in the card catalogue system. A coin-operated photocopying machine is available for public use.

The University of Hong Kong, the Chinese University of Hong Kong and the Hong Kong Polytechnic each has a large library. The first two, in particular, have collections of general geological and geotechnical information. All three, however, can only be used by members of the general public by special permission, although this is usually not difficult for bona fide visitors to obtain. The University of Hong Kong maintains an outstanding Hong Kong Collection, which contains a good deal of unpublished information, as well as a large number of master's and doctor's degree theses on geological and geotechnical topics. Coin-operated photocopiers are available in the library for general use.

The Public Records Office of Hong Kong is the central repository for the permanent archives of the Hong Kong Government. The majority of its holdings date from 1945, but it does have some earlier material. It maintains catalogued collections of maps and photographs dating from 1860, together with almost complete collections of the Hong Kong Government

Gazette, Blue Books, Sessional Papers, Annual Departmental Reports, Ordinances and Regulations, and Hong Kong Hansard. The Sessional Papers are of particular interest because, from 1889, they include the Annual Reports of the Director of Public Works which give information on failures and remedial works. Also of great value is the comprehensive newspaper collection held by the Public Records Office.

12.3.2 Geotechnical Information Unit of the GCO

The Geotechnical Control Office has an established Geotechnical Information Unit which maintains a large collection of published and unpublished documents that are specific to Hong Kong. This collection of information may be referred to by bona fide users. Photocopying facilities are available.

A full copy of every 'short' publication listed in the Bibliography on the Geology and Geotechnical Engineering of Hong Kong (Brand, 1984) is kept in the Geotechnical Information Unit of the Geotechnical Control Office, together with copies of the title and contents pages of the 'long' publications. These copies are contained in bound volumes by year of publication and then in alphabetical order of authors' surnames. Full copies of some of the long publications are also available in the Geotechnical Information Unit, but these are shelved separately. Copies of future publications will be added to the collection as they become available.

The Geotechnical Information Unit also maintains a large collection of 'unpublished' documents that are directly concerned with the geology and geotechnical engineering of Hong Kong. This includes records of landslides, site investigation reports that show drillhole logs and laboratory test results, and factual reports and drawings prepared by Government Departments and Consulting Engineers for a wide range of large and small building and civil engineering projects.

The documents contained in the Geotechnical Information Unit can be accessed on a geographical basis by means of a simple map referencing system which has been devised for this purpose. A computerised retrieval system is now being devised.

12.3.3 Government Departments

Several Government Departments possess information that is of value for geotechnical engineering in Hong Kong, but this is often not readily accessible. Arrangements can usually be made, however, for specific information to be made available to bona fide users.

Each Government Department retains its own files on projects that were carried out under its control, and copies of design reports and record drawings of completed projects are usually made available for examination to those who require the information for design or construction purposes.

The Buildings Ordinance Office retains detailed records of private developments for about seven years after their completion, after which time the files are transferred to the Public Records Office. Permission to view a particular set of records may be obtained from the Secretary of the BOO, who will require to know the address of the property and the lot number.

The Highways Office of the Engineering Development Department has records of the design and construction of the majority of the Territory's roads. It also has copies of drainage plans and location plans of disused Second World War defence tunnels, together with details of remedial works carried out on these. The Civil Engineering Office of the same Department maintains record drawings of marine works and certain trunk drains and sewers.

The Water Supplies Department maintains records of all the facilities under its control, which include water mains, catchwaters and reservoirs. It has a comprehensive system of stream gauging in the main catchment areas, and this information is published in annual reports on rainfall and runoff.

The Lands Department maintains records of lot boundaries throughout the Territory, and these may be viewed by appointment. The Department is also responsible for the production of topographical maps and aerial photographs, which may be purchased by members of the public from the Department's Sales and Distribution Counter. Tables 12.1 and 12.2 list the aerial photographs and maps that are currently available.

The Royal Observatory collects detailed weather information on a continuous basis, and rainfall records are published monthly and annually for public use.

The Government Secretariat Library contains certain information that could be useful from an historical point of view. This includes Sessional Papers, Administrative Reports, Statistical Abstracts and Legislative Council Minutes. The Photographic Library and Reference Library of the Information Services Department hold sets of old photographs, microfilms of newspaper cuttings and other useful material.

Copies of any Government publications in print may be purchased from the Government Publications Centre.

12.3.4 Other Local Sources

Some of the locally published newspapers have good photographic collections to which access is permitted upon request. There are also several interesting private collections of old photographs.

It may sometimes be found useful for nearby residents to be consulted about the details of a specific past incident. Good qualitative descriptions of failures, etc. can often be obtained in this way, but the details of such things as timing are often unreliable.

12.4. INFORMATION ON SERVICES AND UTILITIES

Information on services and utilities is often of importance for the design of new slopes, retaining walls or site formations, or for the execution of remedial or preventive works. The addresses of the major private companies and Government Departments concerned with services and utilities in Hong Kong are included in Section 12.5.

12.5 ADDRESSES OF ORGANISATIONS REFERRED TO IN THIS CHAPTER

This section contains an alphabetical listing of all the organisations referred to in this chapter.

Asian Institute of Technology
Website: http://www.ait.ac.th

British Geological Survey
BGS Library
Website: http://www.bgs.ac.uk

British Library
Website: http://www.bl.uk

Buildings Department
(Formerly Buildings Ordinance Office)
HKSARG*
Website: http://www.bd.gov.hk

China Light & Power Company Ltd.
Website: http://www.clpgroup.com

The Chinese University of Hong Kong
University Library System
Website: http://www.lib.cuhk.edu.hk/

City Hall Public Library
Leisure and Cultural Services Department
HKSARG*
Website:
http://www.cityhall.gov.hk/en/library.php

Civil Engineering Office
Civil Engineering and Development Department
HKSARG*
Website: http://www.cedd.gov.hk

CSIRO Library Services
Website: http://www.csiro.au

Geodex International Inc.
P.O. Box 279
Sonoma, California 95476, USA

The Geological Society
(Formerly Geological Society of London)
Website: http://www.geolsoc.org.uk

Geotechnical Abstracts
38 Römerstr., 5000 Köln 50
Germany

Geotechnical Engineering Office
(Formerly Geotechnical Control Office)
Civil Engineering and Development Department
HKSARG*
Website: http://www.cedd.gov.hk

Geotechnical Information Unit
Civil Engineering and Development Department
HKSARG*
Website: http://www.cedd.gov.hk

Highways Department
(Formerly Highways Office)
HKSARG*
Website: http://www.hyd.gov.hk

Hong Kong and China Gas Company Ltd.
Website: http://www.towngas.com

Hong Kong Electric Company Ltd.
Website: http://www.heh.com

Hong Kong Polytechnic University
Pao Yue-kong Library
(Formerly Hong Kong Polytechnic Library)
Website:
http://library.polyu.edu.hk

Institution of Civil Engineers Library
Website: http://www.ice.org.uk

Laboratoire Central des Ponts
et Chaussées
Website: http://www.lcpc.fr/

Lands Department
HKSARG*
Website: http://www.landsd.gov.hk

Norwegian Geotechnical Institute
WebSite: http://www.ngi.no/en/

Online Government Bookstore
(Formerly Government Publications
Centre)
Information Services Department
HKSARG*
Website: http://www.bookstore.gov.hk

PCCW Limited
(Formerly Hong Kong Telephone Company
Ltd.)
Website: http://www.pccw.com

Public Records Office
Government Records Service
HKSARG*
Website: http://www.grs.gov.hk

Swedish Geotechnical Institute
Website: http://www.swedgeo.se

The University of Hong Kong
University Library System
Website: http://lib.hku.hk

Water Supplies Department
HKSARG*
Website: http://www.wsd.gov.hk

* Denotes "The Government of the Hong Kong Special Administrative Region"

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TABLES

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LIST OF TABLES

Table 2.1 - Guidance on Site Investigation

Risk Category		Formed Slope Classification					Angle of Natural Hillside in the Vicinity of the Site		
Category	a. Loss of Life b. Economic Loss	Features	Soil		Rock	Retaining Wall	0° to 20°	20° to 40°	Greater than 40°
			Fill	Cut			Description of Site Investigation		
Negligible	a. None expected (no occupied premises).	Height	≯ 7.5 m	≯ 5 m	≯ 7.5 m	≯ 3 m	Assessment of surrounding geology and topography for indication of stability. Visual examination of soil and rock forming the site or to be used for the embankment. Specialist Advice - Requirement (A)	As for 0° to 20°. More detailed geology and topography survey. For the steeper slopes information on soil and rock joint strength parameters. Survey of hydrological features affecting the site. Specialist Advice - Requirement (B)	As for 20° to 40°. Area outside confines of site to be examined for instability of soil, rock and boulders above the site. Specialist Advice - Requirement (B)
	b. Minimal structural damage. Loss of access on minor roads.	Angle	≯ 50°	≯ 30°	-	-			
Low	a. Few (only small occupies premises threatened).	Height	≯ 15 m	≯ 10 m	≯ 7.5 m	≯ 6 m	Geology and topography survey of site and surrounding area. Soil and rock joint strength parameters for foundations and cut slopes. For embankments steeper than 1 on 3, recompacted strength parameters of fill. For cuts, information on groundwater level. Specialist Advice - Requirement (B)	As for 0° to 20°. Survey of hydrological features affecting the site. Specialist Advice - Requirement (B)	As for 20° to 40°. Extend outside limits of site to permit analyses of slopes above and below the site. Specialist Advice - Requirement (C)
	b. Appreciable structural damage. Loss of access on sole access roads.	Angle	≯ 60°	≯ 30°	-	-			
High	a. More than a few.	Height	> 15 m	> 10 m	> 15 m	> 6 m	Detailed geology and topography survey of site and surrounding area. Soil and rock joint strength parameters for foundations and cut slopes. Recompacted strength parameters for fill. For cuts, information on groundwater level. Specialist Advice - Requirement (B)	As for 0° to 20°. Survey of hydrological features affecting the site. Extend investigation locally outside limits of the to permit analyses of slopes above and below the site. - Specialist Advice - Requirement (C)	As for 20° to 40°. Extend investigation more widely outside limits of site to permit analyses of stability of slopes above and below the site. Specialist Advice - Requirement (C)
	b. Excessive structural damage to residential and industrial structures. Loss of access on regional trunk routes.	Angle	> 60°	>30°	-	-			

Note : (1) This Table is intended to provide guidance only. Each situation must be assessed on its merits to decide whether or not the recommended investigation procedures are necessary or if peculiar conditions require even more detailed examination.

(2) Whilst the above gives an indication of the requirements for a site investigation under certain general conditions, Table 2.2 gives more precise information on how the above requirements can be met.

(3) For slopes on which there are unstable boulders, the services of an experienced geotechnical engineer or engineering geologist will always be necessary.

(4) Risk category should be assessed with reference to both present use and development potential of the area.

(5) Formed slope classification to be based upon either slope height or angle whichever gives the highest risk category.

(6) Requirements for specialist advice :
(A) Services of an experienced geotechnical engineer or engineering geologist not necessary.
(B) Services of an experienced geotechnical engineer or engineering geologist to depend on location relative to developed or developable land.
(C) Services of experienced geotechnical engineer or engineering geologist essential.

Table 2.2 - Content of Site Investigation

Risk Category	Angle of Natural Hillside in the Vicinity of the Site		
	0° to 20°	20° to 40°	Greater than 40°
Negligible	B1 D E1 G1	B1 C1 D E1 G1 G3	A B1 C1 D E1 F1 G1 C2 E2 G3
Low	A B1 C1 D E1 F1 G1 C2 E2 G2 G3	A B1 C1 D E1 F1 G1 B2 C2 E2 F2 G2 G3	A B1 C1 D E1 F1 G1 B2 C2 E2 F2 G2 E3 G3
High	A B1 C1 D E1 F1 G1 C2 E2 G2 E3 G3	A B1 C1 D E1 F1 G1 B2 C2 E2 F2 G2 E3 G3	A B1 C1 D E1 F1 G1 B2 C2 E2 F2 G2 E3 G3

A. Examination of terrestrial photographs, aerial photos and geological maps.

B. Survey of 1. topographical, geological and surface drainage features.
2. hydrological features.

C. Geological mapping of 1. surface features.
2. structures.

D. Investigation holes, such as trial pits, boreholes or drillholes, as appropriate.

E. Sampling 1. quality class 4)
2. quality class 3) see Table 2.5
3. quality class 1 or 2)

F. Field measurements of 1. groundwater level.
2. permeability.

G. Laboratory tests 1. classification tests.
2. density tests for fill materials.
3. strength tests for soils and rock joints.

Note : (1) This Table is intended to provide guidance only.

(2) Vane testing may be appropriate in marine silts or other fine grained soils. Installation of instruments for long term monitoring of displacements and pore pressures, should be considered during the site investigation stage (Chapter 10).

(3) Chemical tests will be required if aggressive soil/water is suspected in the vicinity of steel or concrete.

Table 2.3 - Material Decomposition Grades for Weathered Granite and Volcanic Rocks in Hong Kong[1]

Grade	Description	Typical Distinctive Characteristics[2]
VI	Residual soil	A soil formed by weathering in place but with original texture of rock completely destroyed.
V	Completely decomposed rock	Rock wholly decomposed but rock texture preserved No rebound from N Schmidt hammer[3] Slakes readily in water[4] Geological pick easily indents surface when pushed
IV	Highly decomposed rock	Rock weakened so that large pieces can be broken by hand Positive N Schmidt rebound value up to 25 Does not slake readily in water Geological pick cannot be pushed into surface Hand penetrometer strength index greater than 250 kPa[5] Individual grains may be plucked from surface
III	Moderately decomposed rock	Completely discoloured Considerably weathered but possessing strength such that pieces 55 mm diameter cannot be broken by hand N Schmidt rebound value 25 to 45 Rock material not friable
II	Slightly decomposed rock	Discoloured along discontinuities Strength approaches that of fresh rock N Schmidt rebound value greater than 45 More than one blow of geological hammer to break specimen
I	Fresh rock	No visible signs of weathering, not discoloured

Note : (1) This Table is based on Moye (1955) and Hencher & Martin (1982).

(2) These characteristics may be affected by moisture content and microfracturing.

(3) N Schmidt rebound hammer values are for the hammer held horizontally. After seating' blows, take the average of the highest five of ten blows at the same location. Only record as zero if there is no rebound. This index test is not applicable for the description of drillcore.

(4) Samples which are already saturated are less likely to slake.

(5) The presence of residual quartz in decomposed coarse grained rocks can result in wide variation of hand penetrometer values. Take an average of ten values avoiding disturbed or friable areas and divide by two to arrive at the strength index.

Table 2.4 - Common Types of Core-barrels, Casings and Drilling Rods That Are Used in Hong Kong

Core-barrels						Casings			Rods		Comments
Name	Type	Retractor	Liner	Bit Outside Diameter (mm)	Core Diameter (mm)	Size	O.D. (mm)	I.D. (mm)	Size	O.D. (mm)	
N size					54	NW	89	76	NW	67	
H size					76	HW	114	102	HW	89	
P size	Double tube	Without	None		92	PW	140	123	HW	89	
						SW	168	156	HW	89	
						106	118	106	HW	89	Isolated use
TNW				75.6	61.5	NW	89	76	NW	67	
T2-76	Thin wall double tube	Without	None	76	62	NW	89	76	NW	67	Isolated use
T2-101				101	84	PW	140	128	NW	67	Used with mazier
T6-101				101	79	NW	89	76	NW	67	Isolated use
Mazier	Triple tube	With	76 mm OD plastic	101	72	PW	140	128	NW	67	HW casing can be used, although this is not recommended
HMLC											
Water	Triple tube	With or without	Split steel liner	88	64	HW	114	102	NW	67	
Air/foam				91	64	HW	114	102	NW	67	
						106	118	106	NW	67	Isolated use
4C-MLC (SMLC)											
Water	Triple tube	With or without	Split steel liner	139	101	SW	168	156	HW	89	
Air/foam				145	101	SW	168	156	HW	89	

Note : (1) Flush jointed casings and rods (W series) are now used instead of the flush coupled systems (X series).
(2) This list is not exhaustive and should not imply the exclusion of other recognised core-barrels and casing/rod systems.

Table 2.5 - Sample Quality Classes

Quality Class	Purpose	Soil Properties Obtainable	Typical Sampling Procedure
1	Laboratory data on undisturbed soils	Total strength parameters Effective strength parameters Compressibility Density and porosity Water content Fabric Remoulded properties	Piston thin-walled sampler with water balance. Air-foam flush triple-tube core barrel. Block samples.
2	Laboratory data on undisturbed insensitive soils		Pressed or driven thin or thick-walled sampler with water balance. Water flush triple-tube core-barrel.
3	Fabric examination and laboratory data	Water content Fabric Remoulded properties	Pressed or driven thin or thick-walled samplers. Water balance in highly permeable soils. SPT liner samples.
4	Laboratory data on remoulded soils, sequence of strata	Remoulded properties	Bulk and jar samples
5	Approximate sequence of strata only	None	Washings

Table 2.6 - Notes on Logging

A. Trial Pit Logs
(1) All faces of the trial pit should be logged. The orientation of all sides should be recorded and preferably shown in a plan sketch on the log. (2) Water levels recorded on logs should be dated and the time noted.
B. Borehole Logs
(1) Water levels should be read and recorded (a) before boring starts in the morning, (b) after the lunch break, and (c) at the end of the working day. (2) The sections of the hole from which the undisturbed samples are recovered should be shaded in on the log to indicate percentage recovery. The number of blows required to drive the sampler should also be recorded. (3) The standard penetration test result is given as an N value. (4) Results of all field tests should be given on the log.
C. Drillhole Logs
(1) Quantity and colour of water return are estimated visually. (2) Penetration rate of drilling, if required, can be given by any convenient method using numbers or diagrams, providing the system is explained on the logs. (3) Core recovery can be shown graphically, numberically or by both methods. (4) Various fracture indices which are measured on the core can be given. Those shown on the logs are RQD and average fracture spacing. Other indices (not shown) are: fractures per metre over a given length, usually shown as a diagram; percentage of solid core recovered, usually indicated as a number and maximum, minimum and mean core lengths, given as a diagram or a number. (5) Details of instruments can either be shown in diagram form or as a note.

Table 4.1 - Filter Design Criteria To Be Used in Hong Kong

Rule Number	Filter Design Rule	Requirement
1	$D15F_c < 5 \times D85S_f$	Stability (i.e. the pores in the filter must be small enough to prevent infiltration of the material being drained)
2	$D15F_c < 40 \times D15S_f$	
3	$D50F_c < 25 \times D50S_f$	
4	Should not be gap graded	
5	$D15F_f > 5 \times D15S_c$	Permeability (i.e. the filter must be much more permeable than the material being drained)
6	Not more than 5% to pass 75 μm sieve and this fraction to be cohesionless	
7	Uniformity Coefficient $4 < \frac{D60F}{D10F} < 20$	Segregation (i.e. the filter must not become segregated or contaminated prior to, during, and after installation)
8	Maximum size of particle should not be greater than 75 mm	

Note : (1) For uniform soils (i.e. $\frac{D60S}{D10S} < 4$) the criterion given in Rule 2 should be $20 \times D15S_f$.

(2) In this Table, D15F is used to designate the 15% size of the filter material (i.e. the size of the sieve that allows 15% by weight of the filter material to pass through it). Similarly, D85S designates the size of sieve that allows 85% by weight of the base soil to pass through it. The subscript c denotes the coarse side of the envelope and subscript f denotes the fine side.

Table 4.2 - Typical Calculations for the Design of Filters

Filter Design Rule	Soil To Be Protected (mm)		Filter (mm)	
$D15F_c < 5 \times D85S_f$	$D85S_f = 1.0$		$D15F_c < 5$	
$D15F_c < 40 \times D15S_f$	$D15S_f = 0.18$		$D15F_c < 7.2$	
$D50F_f < 5 \times D15S_c$	$D50S_f = 0.48$		$D50F_c < 12$	
$D15F_f > 5 \times D15S_c$	$D15S_c = 0.35$		$D15F_f > 1.75$	
Uniformity Coefficients	D10	D60	Uniformity Coefficient	
$4 < \frac{D60F_f}{D10F_f} < 20$	1.4	6.0	4.3	O.K.
$4 < \frac{D60F_c}{D10F_c} < 20$	4.0	16.5	4.1	O.K.

Note : (1) In this example calculation, the soil to be protected is the USCE sand filter (A) shown in Figure 4.9. The calculated grading for the drainage material is also shown in Figure 4.9.

(2) The filter design rules used in the calculation are those shown in Table 4.1.

Table 5.1 - Recommended Factors of Safety for New Slopes for a Ten-year Return Period Rainfall

<table>
<tr><td colspan="2" rowspan="2">RISK TO LIFE
ECONOMIC RISK</td><td colspan="3">Recommended Factor of Safety against Loss of Life for a Ten-year Return Period Rainfall</td></tr>
<tr><td>Negligible</td><td>Low</td><td>High</td></tr>
<tr><td rowspan="3">Recommended Factor of Safety against Economic Loss for a Ten-year Return Period Rainfall</td><td>Negligible</td><td>>1.0</td><td>1.2</td><td>1.4</td></tr>
<tr><td>Low</td><td>1.2</td><td>1.2</td><td>1.4</td></tr>
<tr><td>High</td><td>1.4</td><td>1.4</td><td>1.4</td></tr>
<tr><td colspan="5">Note : (1) In addition to a factor of safety of 1.4 for a ten-year return period rainfall, a slope in the high risk-to-life category should have a factor of safety of 1.1 for the predicted worst groundwater conditions.
(2) The factors of safety given in this Table are recommended values. Higher or lower factors of safety might be warranted in particular situations in respect of economic loss.</td></tr>
</table>

Table 5.2 - Typical Examples of Slope Failures in Each Risk-to-Life Category

Example	Risk to Life		
	Negligible	Low	High
(1) Failure affecting country parks and lightly used open-air recreation areas.	√		
(2) Failures affecting roads with low traffic density.	√		
(3) Failures affecting storage compounds (non-dangerous goods).	√		
(4) Failures affecting densely used open spaces and recreational facilities (e.g. sitting-out areas, playgrounds, car parks).		√	
(5) Failures affecting roads with high vehicular or pedestrian traffic density.		√	
(6) Failures affecting public waiting areas (e.g. railway platforms, bus stops, petrol stations).		√	
(7) Failures affecting occupied buildings (e.g. residential, educational, commercial, industrial).			√
(8) Failures affecting buildings storing dangerous goods.			√

Table 5.3 - Typical Examples of Slope Failures in Each Economic Risk Category

Example	Economic Risk		
	Negligible	Low	High
(1) Failures affecting country parks.	√		
(2) Failures affecting rural (B), feeder, district distributor and local distributor roads which are not sole accesses.	√		
(3) Failures affecting open-air car parks.	√		
(4) Failures affecting rural (A) or primary distributor roads which are not sole accesses.		√	
(5) Failures affecting essential services which could cause loss of that service for a temporary period (e.g. power, water and gas mains).		√	
(6) Failures affecting rural or urban trunk roads or roads of strategic importance.			√
(7) Failures affecting essential services, which could cause loss of that service for an extended period.			√
(8) Failures affecting buildings, which could cause excessive structural damage.			√
Note : These examples are for guidance only. The designer must decide for himself the degree of economic risk and must balance the potential economic risk in event of a failure against the increased construction costs required to achieve a higher factor of safety.			

Table 5.4 - Recommended Factors of Safety for the Analysis of Existing Slopes and for Remedial and Preventive Works to Slopes for a Ten-year Return Period Rainfall

Risk to Life	Recommended Factor of Safety Against Loss of Life for a Ten-year Return Period Rainfall		
	Negligible	Low	High
	> 1.0	1.1	1.2

Note : (1) These factors of safety are minimum values to be used only where rigorous geological and geotechnical studies have been carried out, where the slope has been standing for a considerable time, and where the loading conditions, the groundwater regime and the basic form of the modified slope remain substantially the same as those of the existing slope.

(2) Should the back-analysis approach be adopted for the design of remedial or preventive works, it may be assumed that the existing slope had a minimum factor of safety of 1.0 for the worst known loading and groundwater conditions.

(3) For a failed or distressed slope, the causes of the failure or distress must be specifically identified and taken into account in the design of the remedial works.

Table 5.5 - Methods of Stability Analysis for Soil Slopes

Method	Failure Surface	Assumptions	Advantages	Limitations	Reference	Recommendation
Infinite Slope	Straight line	Any vertical slice is representative of the whole slope	Simple hand calculation method	Failure surface assumptions always an approximation. Method may only be used for slip surfaces where the length to depth ratio is large and end effects can be neglected.	Lambe & Whitman (1969)	Suitable for long slopes, especially those with a thin layer of weathered soil over rock.
Sliding Block	Two or more straight lines	The sliding mass can be divided into two or more blocks, the equilibrium of each block is considered independently using interblock forces	Suitable for hand calculation when 2 or 3 blocks are used.	Does not consider the deformation of blocks. Result sensitive to the angle to the horizontal chosen for the interblock forces and the inclination of the surface between the blocks.	Lambe & Whitman (1969)	Useful where there is a weak stratum within or below the slope and when the slope rests upon a very strong stratum.
Bishop	Circular	Considers force and moment equilibrium for each slice. Rigorous method assumes values for the vertical forces on the sides of each slice until all equations are satisfied. Simplified method assumes the resultant of the vertical forces is zero on each slice.	Simplified method compares well with finite element deformation methods (average F within 8%). Computer programs readily available.	Circular failure surface not always suitable for Hong Kong slopes but large radius circles can sometimes be used.	Bishop (1955)	Useful where circular failure surfaces can be assumed.
Bishop & Morgensterns Charts	Circular	Uses Bishop's simplified method with an average r_u value.	Simple to use. More accurate than Hoek's charts.	Limited to homogeneous soils and slopes flatter than 27°	Bishop & Morgenstern (1960)	Limited usefulness.
Hoek's Charts	Circular	Sliding mass considered as a whole. Lower bound solution, assuming normal stresses are concentrated at one point.	Slope angles from 10° to 90° given. Very simple to use.	Limited to homogeneous soils and five specified groundwater conditions.	Hoek & Bray (1981)	Very useful for preliminary calculations or for small low risk slopes.
Janbu	Non-circular	Generalised procedure considers force and moment equilibrium on each slice. Assumptions on line of action of interslice forces must be made. Vertical interslice forces not included in Routine procedure and calculated F then corrected to allow for vertical forces.	Realistic shear surfaces can be used. Routine analysis can be easily handled by a programable calculator or by hand.	Published f_0 factors are for homogeneous materials and routine procedure can give large errors in slopes composed of more than one material. Factor of safety is usually underestimated in these cases. Generalised method does not have the same limitations.	Janbu (1972) Routine method given in Hoek & Bray (1981)	Very useful for the majority of soil slopes in Hong Kong. Limitations of routine method must be considered.
Morgenstern & Price	Non-circular	Considers forces and moments on each slice, similar to Janbu Generalised procedure.	Considered more accurate than Janbu. Computer programs readily available.	No simplified method. Computer solution necessary, often very time consuming.	Morgenstern & Price (1965)	Usually unnecessarily detailed for Hong Kong soils where strength and pore pressure are not known with accuracy. Most useful for back analysis of landslides.
Sarma	Non-circular	A modification of Morgenstern & Price which reduces the iterations required by using earthquake forces	Considerable reduction in computing time without loss of accuracy.	Computer programes not yet readily available but can be used with a calculator.	Sarma (1979)	Can be used as an alternative to Morgenstern & Price.

Table 5.6 - Methods of Stability Analysis for Rock Slopes

Method	Failure Surface	Assumptions	Advantages	Limitations	Reference	Recommendation
Plane Failure	Single plane with tension crack	Both sliding surface and tension crack strike parallel to the slope surface. Release surfaces are present so there is no resistance on lateral boundaries.	Water pressures in tension crack and on sliding plane can be included simple analysis method.	Moments not considered in analysis. Can give over estimate of factor of safety on steep slopes where toppling could occur.	Hoek & Bray (1981)	Useful where plane failure can be assumed such as on sheet joints.
Wedge Failure	Two joint planes form 3 dimensional wedge.	Line of intersection of joints dips less steeply than rock face and daylights within the face. Both joint planes remain in contact during sliding.	Tension crack and water pressures can be included in analysis. Charts, which consider friction only, are available.	Moments not considered	Hoek & Bray (1981)	Useful. Charts can be used for a preliminary assessment.
Toppling Failure	Stepped cross joints	Analysis assumes that some blocks will slide and some topple. Water pressures not included.		Limited to a few simple cases with suitable geometry.	Hoek & Bray (1981)	Not yet a rock slope design tool but may occasionally be useful.

Table 5.7 - Rock Slope Stabilisation Measures

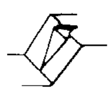
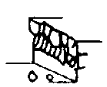
Failure Type		STABILISATION MEASURES																				
		Excavation			Structural Support									Drainage				Rockfall Control				
Name	Sketch	Flatten slope	Bench	Local excavation	Gunite facing	Permeable (masonry) facing	Local structural "dentition"	Buttress	Anchored wall	Strap	Dowel	Bolt	Anchor	Drainage ditch	Screeded (paved) surface	Short drainholes	Long drainholes /adits	Move structure/highway	Rock trap ditch	Rock trap fence/ wall	Netting	Scaling of loose blocks
Plane Failure		√	√					√	√			√	√	√	√	√		√	√	√		
Wedge Failure		√						√	√		√	√	√	√	√		√	√				
Toppling Failure		√										√	√	√	√		√		√			
Rock or Debris Fall & General Degradation		√	√	√	√	√	√	√	√	√	√	√		√	√	√		√	√	√	√	√

Table 7.1 - Recommended Factors of Safety for Gravity Retaining Walls for a Ten-year Return Period Rainfall

Mode of Failure	Recommended Factor of Safety for a Ten-year Return Period Rainfall	
	New Walls	Remedial and Preventive Works to Existing Walls
Sliding	1.5	1.25
Rotation	2.0	1.5
	(For a masonry wall, the resultant force should lie within the middle-third)	
Bearing Capacity	3.0	Existing value to be maintained if below 3.0. For a wall with a toe slope, overall stability of the of the slope must be adequate
Slope Failure	Refer to Section 5.2	

Note : (1) The factors of safety for remedial and preventive works are minimum values to be used only where rigorous structural, geological and geotechnical studies have been carried out, where the wall has been standing for a considerable time, and where the loading conditions, the groundwater regime and the basic form of the modified wall remain substantially the same as those of the existing wall.

(2) Should the back-analysis approach be adopted for the design of remedial or preventive works, it may be assumed that the existing wall had a minimum factor of safety of 1.0 for the worst known loading and groundwater conditions.

(3) For a failed or distressed wall, the causes of the failure or distress must be specifically identified and taken into account in the design of the remedial works.

Table 8.1 - Effects of Vegetation on Slope Stability

BENEFICIAL FACTORS	Type
Interception of rainfall by foliage, including evaporative losses	H
Depletion of soil moisture and increase of soil suction by root uptake and transpiration	H
Mechanical reinforcement by roots	M
Restraint by buttressing and soil arching between tree trunks	M
Surcharging the slope by large (heavy) trees *	M
Arresting the roll of loose boulders by trees	M
ADVERSE FACTORS	
Surcharging the slope by large (heavy) trees *	M
Maintaining infiltration capacity	H
Root wedging of near-surface rocks and boulders and uprooting in typhoon	M

Legend :

H hydrological
M mechanical
* This mechanism may be either beneficial or adverse to stability, depending on particular site factors (see Gray, 1978).

Table 8.2 - Slope Angle Limitations on Establishment of Vegetation

Slope Angle	Establishment of	
	Grass	Shrub/Trees
0° - 30° slope	Low in difficulty; routine planting techniques may be used	Low difficulty; routine planting techniques may be used
30° - 45° slope	Increasingly difficult for sprigging or turfing; routine application for hydroseeding	Increasingly difficult to plant
Greater than 45° slope	Special consideration required	Planting must generally be on benches

Table 8.3 - Characteristics of Grass Species Commonly Used in Hong Kong

Grass	Botanical Name	Height (m)	Characteristics
Bahia	Paspalum notatum	0.3-0.5	Root system is extensive and strong, effective for erosion control, compatible with shrub and tree planting, well adapted to wide range of sites, good shade, drought and wear tolerance; early growth rate may be slow.
Bermuda	Cynodon dactylon	0.3	Root system is extensive, effective for erosion control, well adapted to a wide range of sites; browns in winter.
Buffel	Cenchrus ciliaris	0.7-1.5	Root system is extensive and strong, fire and drought tolerant, will grow on infertile sites.
Carpet	Axonopus compressus	0.1-0.2	Root system shallow but effective for erosion control, usually planted by sprigging/turfing; prefers moist conditions, browns in the winter.
Centipede	Eremochloa ophuroides	0.2	Will grow on infertile sites, easily established, effective for erosion control.
Rhodes	Chloris gayana	0.5-1.0	Tufted*, aggressive, fire tolerant; not recommended for general use.
Perennial Rye	Lolium perenne	0.1-0.9	Quick germination and good early growth, winter sowing recommended; does not survive very hot weather.
Weeping Love	Eragrostis curvula	0.5-0.9	Tufted*, aggressive; chokes out other grasses, browns in winter and may create a fire hazard; not recommended for general use.

Legend :

* Tufted denotes a cluster of shoots arising from a common root system, and may not be as effective for erosion control as nontufted grasses.

Note : All species listed are perennial (long-lasting).

Table 8.4 - Characteristics of Shrub Species Commonly Used in Hong Kong

Shrub Species		Growth Rate			Extreme Conditions Tolerated					
Scientific Name	Common Name	Fast	Medium	Slow	Rocky sites	Wet soils	Exposed sites	Shady sites	Urban Situation	Rural Situation
Allamanda Cathartica	Allamanda		√						√	
Duranta Repens	Golden Dewdrops	√							√	
Gordonia Axillaris	Gordonia		√		√		√			√
Hibiscus Species	Hibiscus		√						√	
Melastoma Species				√	√	√	√	√		√
Nerium Indicum	Oleander	√							√	
Rhaphiolepis Indica	Hong Kong Hawthorn			√	√			√		√
Rhododendron Species	Rhododendron		√		√		√		√	√
Rhodomyrtus Tomentosa	Rose Myrtle			√	√		√			√
Thevetia Peruviana	Yellow Oleander		√						√	
Tithonia Diversifolia	Mexican Sunflower	√			√		√			√

Note : Terms used in Tables 8.4 and 8.5 are defined as followed :

(a) Wet soils : groundwater level near the surface for most of the year.

(b) Exposed sites : windy site with southerly to westerly aspect (dip direction).

(c) Shady sites : shaded by buildings or a steep northerly aspect.

(d) Salt spray : marine sites usually within 100 m of the sea.

(e) Typhoon tolerant : resists breakage of limbs in a typhoon or, if broken, is able to regenerate.

(f) Fire tolerant : able to regenerate after being burned.

Table 8.5 - Characteristics of Tree Species Commonly Used in Hong Kong

Tree Species		Height		Growth Rate			Extreme Conditions Tolerated						
Scientific Name	Common Name	Over 7 m	Under 7 m	Fast	Medium	Slow	Rocky sites	Wet soil	Exposed sites	Shady sites	Salt spray	Typhoon tolerant	Fire tolerant
Acacia Confusa	Acacia		√	√			√		√			√	√
Albizza Lebbek	Lebbek Tree	√		√			√	√	√		√	√	
Alnus Formosana	Japanese Alder	√		√									
Bauhinia Species	Bauhinia		√		√								
Casuarina Equisetifolia	Horsetail Tree	√		√			√	√	√		√	√	
Casuarina Stricta	Long-Leaved Ironwood	√			√		√		√		√	√	√
Celtis Sinensis	Chinese Hackberry	√				√				√		√	
Eucalyptus Torrelliana		√		√			√		√		√	√	
Leucaena Leucocephala	White Popinac		√	√			√		√		√	√	√
Pinus* Elliottii	Slash Pine	√			√		√	√	√				
Tristania Conferta	Brisbane Box	√		√			√	√	√			√	√

Legend :

* May be adversely affected by the disease pine wilt nematode in some locations

Note : (1) See Table 8.4 for definition of terms used.
(2) All species listed have proven to be acceptable and successful on both cut slopes and fill slopes and have sufficient root systems to resist uprooting in typhoons.

Table 9.1 - Typical Hydroseeding Mix

Application Rate[(2)] (g/m^2)	Mix Component[(1)]
25 - 30	Grass seed[(3)]
100	Fertiliser[(4)] (NPK)
170 - 250	Fibrous mulch

Note :
(1) Dye and soil stabiliser may also be included.
(2) Application rate should be computed on slope distances rather than horizontal or vertical projections of area.
(3) A three species mix containing chiefly Bermuda and Bahia is often used (refer to Table 8.3).
(4) This rate is for a chemical composition of 15 : 15 : 15 (Nitrogen : Phosphorus : Potassium) and should be adjusted when other compositions are used. Follow up fertiliser applications are also recommended.

Table 10.1 - Schematic Flow Chart for Planning a Monitoring Operation

1 GROUND BEHAVIOUR WARNING LEVELS AND CONTINGENCY ACTION	2 GENERAL MONITORING PLAN	3 DETAILED MONITORING PLAN
PROJECT DEFINITION Geometry; geology; groundwater; stress; construction programme ↓ GROUND BEHAVIOUR Mechanism; critical locations; magnitudes; rates ↓ CONTINGENCY PLANNING Decisions on hazard warning levels; action plans if warning levels exceeded	TERMS OF REFERENCE Monitoring objectives; budget ↓ WHAT TO MEASURE Displacement; water; pressure; load ↓ WHERE TO MEASURE Identify key locations and depths; establish priorities ↓ WHEN TO MEASURE Project duration; frequency of readings; frequency of reports	PERSONNEL No. of persons; allocation of responsibilities; liaison and reporting channels ↓ INSTRUMENTS Selection; calibration; detailed layout ↓ INSTALLATION Define installation locations, times and procedures ↓ MONITORING Define detailed monitoring programme ↓ DATA PROCESSING Draft & print data sheets and graphs; set up computation procedures ↓ REPORTING Define reporting requirements; timing; contents; responsibilities

Table 10.2 - Piezometer Types

	Type	Range	Response	Dearing	Remote Reading	Long Term Reliability	Other: Advantages	Other: Disadvantages	Recommendation
Positive pressure	Open Hydraulic (Casagrande)	Atmospheric to top of standpipe level	Slow	Self deairing	Not normally, but possible with Halcrow buckets or bubbler system	Very good	Cheap, simple to read & maintain. Insitu permeability measurement possible	Vandal damage often irreparable	First choice for measurement within positive pressure range unless very rapid response or remote reading required
Positive pressure	Closed Hydraulic (Low air entry pressure)	Any positive pressure	Moderate	Can be deaired	Yes	Depends on pressure measuring system 1) Mercury manometer - very good 2) Bourdon gauge - poor in humid atmosphere 3) Pressure transducer - moderate but easily replaced	Fairly cheap. Insitu permeability measurement possible	Gauge house usually required. Regular deairing necessary. Uncovered tubing liable to rodent attack	Useful when remote reading required and for artesian pressures
Positive pressure	Closed Hydraulic (High air entry pressure)	-1 atmosphere to any positive pressure	Moderate	Can be deaired	Yes	As above	Fairly cheap. Insitu permeability measurements in low permeability soil are possible	As above. Very regular deairing required when measuring suctions	Useful for measuring small suctions
Positive pressure	Pneumatic	Any positive pressure	Rapid	Cannot be deaired. Only partially self deairing	Yes Some head loss over long distance	Moderate to poor but very little long term experience available.	Fairly cheap. No gauge house required	No method of checking if pore water or pore air pressure is measured	Only suitable when tip almost always below groundwater level and no large suctions occur
Positive pressure	Electric vibrating - wire type	Any positive pressure	Rapid	As above	Yes but special cable required	Signal quality degenerates with time. Instrument life about ten years but reliability of instrument that cannot be checked is always suspect		As above. Expensive. Zero reading liable to drift and cannot be checked	Not generally recommended
Positive pressure	Electric resistance type	Any positive pressure	Rapid	As above	Yes but with care because of transmission losses	Poor		As above	Not recommended
Suction	Tensiometer	-1 to positive pressure	Moderate to rapid	Can be deaired	Yes	Good	Cheap, simple to read and maintain	Vandal damage often irreparable. Regular deairing required	First choice for measuring pore suction
Suction	Psychrometer	Below -1 atmosphere	Variable	Not relevant	Short distances only	Instrument life one to two years. Little long term experience available		Not accurate between 0 and -1 atmosphere	Research stage at the moment

Table 10.3 - Load Cells

Type	Long Term Reliability	Remote Reading	Other		Recommendation
			Advantages	Disadvantages	
Hydraulic Load Cell	Good	Not with dial gauge, possible with linear displacement transducer	Simple to operate	System fails if load cell or hydraulic tubes leak. Gauge needs to be connected permanently	Most suitable for long term monitoring if a transducer is used
Hydraulic Jack	Good except pressure gauges subject to corrosion	No, access required to measure	Simple hydraulic system	Not continuous reading. Jack must be pressurised until anchor head begins to move. Difficult to just lift head, large movements often occur	Not very suitable for long term regular monitoring. Removable jack suitable for occasional testing
Photoelastic	Good	No	Cheap, simple to install easily replaced	Reading requires practice. Limited to low loads	Suitable for rock bolts and small anchors
Electrical (vibrating-wire type)	Moderate (up to 10 years)	Yes	Remote reading not affected by environment	Fairly expensive. Calibration may drift but recalibration possible	Most suitable where remote reading required
Electrical (resistance type)	Poor	No (a few metres is possible)	Simple, easy to read	As above. Stain gauges very susceptible to corrosion	Not suitable for long term use

Table 11.1 - Interval between Maintenance Inspections

Inspecting Officer	Recommended Interval	
	Slopes in the High Risk-to-life Category	Slopes in the Negligible or Low Risk-to-life Category
Technical Officer	6 months	1 year
Engineer	2 years	5 years

Note : (1) Slopes in the high, low and negligible risk-to-life categories are defined in Chapter 5.
(2) In certain cases where the risk-to-life is high, it may be necessary to seek advice from a geotechnical engineer.

Table 12.1 - Summary of Aerial Photographs Available from Lands Department

Year of Photographs	Remarks
1924, 1945, 1949, 1950, 1954 1956, 1959, 1961, 1962, 1963	Photographs taken by RAF. Partial coverage of the territory only
1963, 1964, 1967	Photographs taken by Hunting Survey Ltd. Full coverage of the territory available
1968, 1969, 1970	Photographs taken by Crown Lands and Survey Office. Partial coverage of the territory only
1972 onwards	Photographs taken annually by Lands Department. Full coverage of the territory available

Table 12.2 - Map Catalogue

	Code	Coverage	Number of Sheets	Price per Copy (1984) (HK$)
Large Scale (Basic) Plans				
1:600 (50 ft. to 1 in.)		Hong Kong	140	10.00
1:1 200 (100 ft. to 1 in.)		NT & Islands	1000 (approx.)	10.00
1:1 000 (replacing 1:600 and 1:1 200 series)	HPIC	Kowloon Hong Kong NT & Islands	126 192 1132	10.00 10.00 10.00
Medium Scale (Derived) Plans				
1:2 500	HP2.5C	Urban	73	10.00
1:5 000	HP5C	Urban NT	20 11	10.00 10.00
1:7 500 Street Maps	SM7D	Urban & NT Township	22	10.00
1:15 000	SM15D	Urban	2 (1 HK & 1 Kln.)	10.00
Topographic Maps				
1:10 000	L884	Full (Obsolete)	62	2.00-3.00
1:20 000	HM20C	Full	16	10.00
1:50 000	HM50CL	Full	2	16.00
1:100 000	HM100CL	Full	1	6.00
Special Maps				
1. Official Guide Map	TM100CL	Full	1	8.50
2. Countryside Series				
(a) Hong Kong Island	CM25C		1	10.00
(b) New Territories-West	CM50C		1	10.00
(c) Lantau & Islands	CM35C		1	10.00
(d) Sai Kung & Clear Water Bay	CM25C		1	10.00
3. Geological Report & Map/Map only				60.00/16.00
4. Streets & Places Guide Books				
i) Vol. I		HK & Islands		20.00
ii) Vol. II		Kowloon & NT		20.00
5. Maps for HK Annual Reports			6	2.00
Miscellaneous				
Aerial photographs - copies available by special order				16.00
Approved Town Planning Development and Layout Plans				Varies

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FIGURES

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LIST OF FIGURES

Zones of Weathering (based on Ruxton & Berry, 1957)	
Zone A	Structureless sand, silt and clay. May have boulder concentration at the surface
Zone B	Residual material with corestones. Rock percentage is less than 50%, and corestones are rounded and not interlocked
Zone C	Corestones with residual material. Rock percentage is 50 to 90% and corestones are rectangular and interlocked
Zone D	More than 90% rock. Minor residual material along major structural discontinuities which may be considerably iron stained

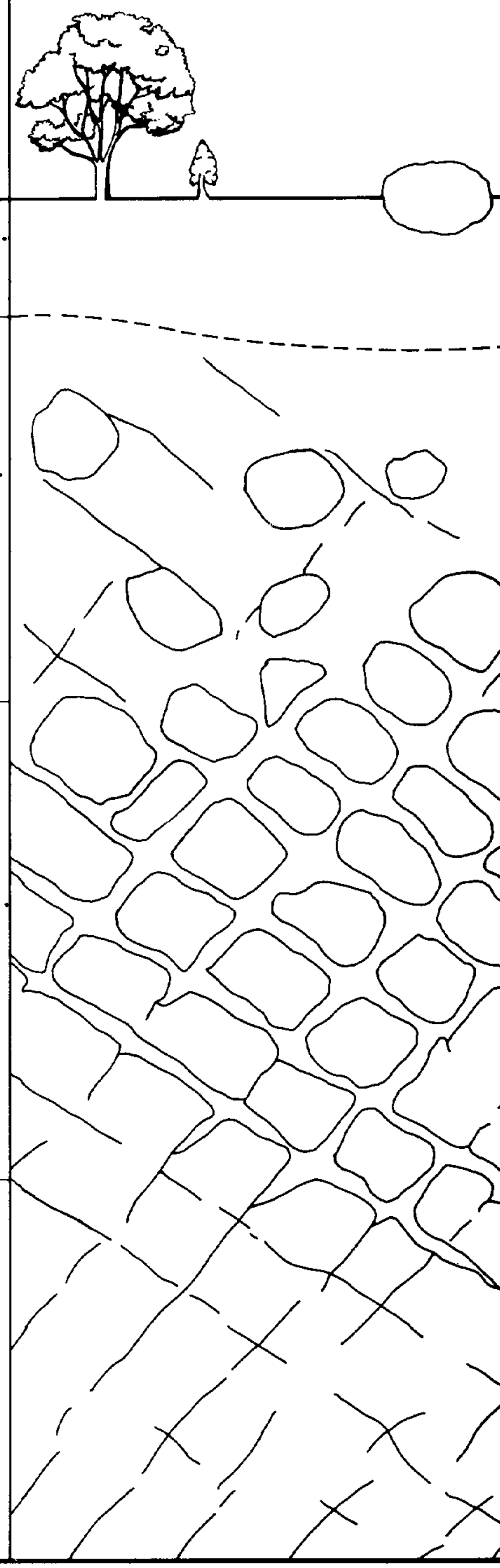


Figure 1.1 - Idealized Weathering Profile in Granitic Rocks

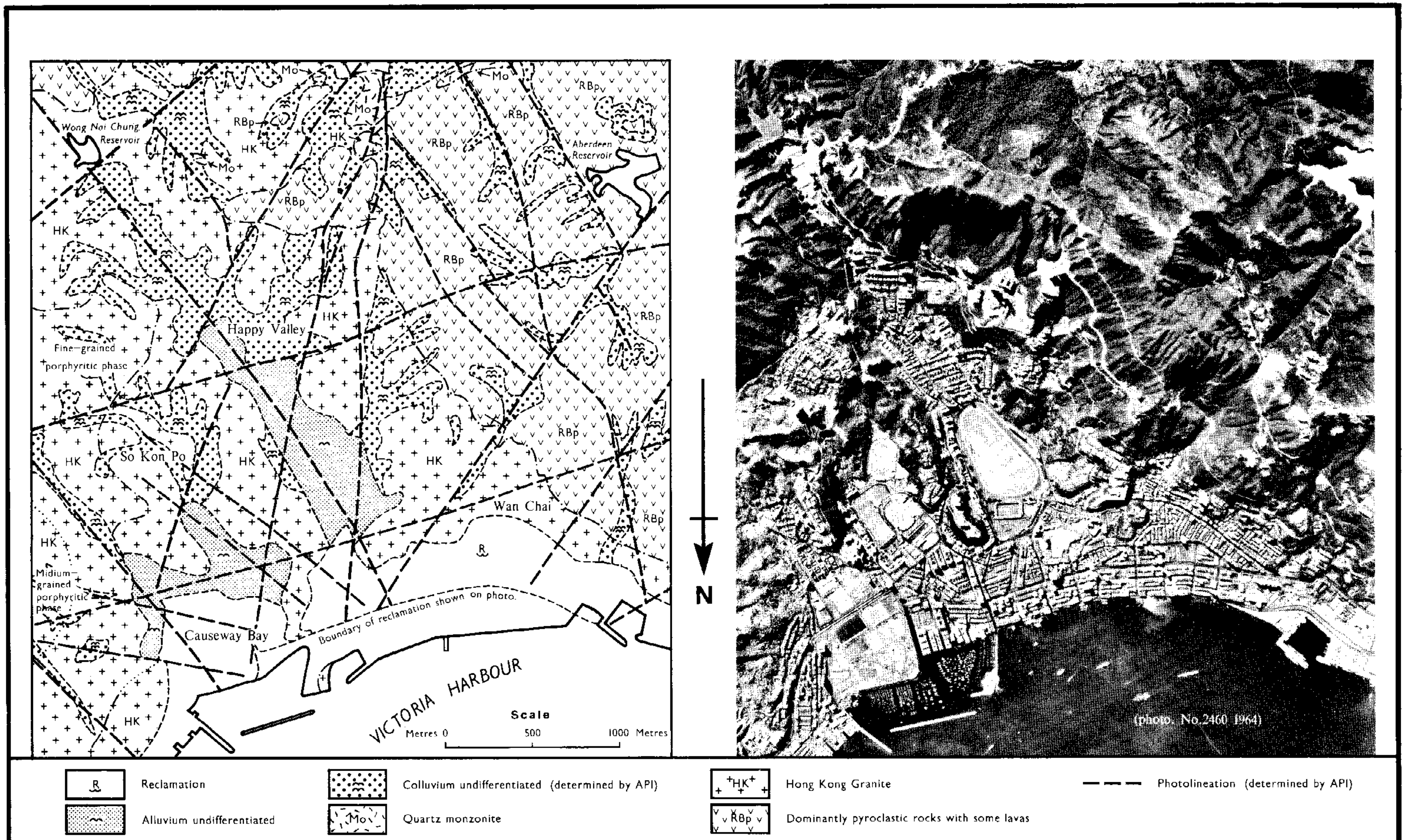


Figure 2.1 - Geological Map and Aerial Photograph that Identify Some Major Structural and Lithological Features

DRILLHOLE No : ALC/12
Sheet 1 of 3

Type of drilling : Rotary
Rig : D-1
Bit : T.C. & Diamond

Coordinates : E 34071
N 11453
Angle from horizontal : 90°
Bearing : N -- E --

Feature : HKHA - Geotechnical Advice
Location : Ap Lei Chau Site 'B'
Ground level : +86.50 m.P.D.
Water table level : See piezometer sheets

Drilling progress	Casing depth, size	Water level	Lugeon value	Notes e.g. Colour water return caving instrumentation	Depth & diameter	Reduced level m.P.D.	Core recovery %	Rate of penetration	R.Q.D.	Fracture index	Legend	Description	Grade
					metres	86.50	0 50				0.00 m		
2/3					0	86.26	95				0.24 m	Grey brown friable fine sandy SILT with rootlets. Topsoil.	
					U100 (0–1.90)		93, 45, 45				1.90 m	Yellow brown friable sandy SILT with occasional rootlets with coarse quartz SAND. Residual Soil.	VI
				90% water return from 1.68 m to 10.05 m	1 – 4	84.60	72, 77, 60				4.50 m	Pink to yellow-brown with creamy white patches soft SILT with occasional black speckles and some limonite stained relict discontinuities. Completely decomposed volcanic TUFF.	V
2/3	HXC at 4.67 m	2.15 m at 19.00 hrs.			HMLC (1.90–10)	82.00	71, 89				6.30 m	Creamy white with pink and yellow patches soft SILT with pink and limonite stained relict discontinuities. Completely decomposed volcanic TUFF.	V
3/3		5.07 m at 8.00 hrs.			6 – 7	80.20	77, 87, 61				7.42 m	Pink with yellow brown soft to firm SILT. Completely decomposed volcanic TUFF.	
					8	79.08	83, 90				8.35 m	Light grey soft to firm SILT with red brown stained relict discontinuities. Completely decomposed TUFF.	V
					9	78.15	86				9.21 m	Yellow with red brown patches firm SILT. Completely decomposed TUFF.	V
3/3	HXC at 9.58 m	0.86 m at 19.00 hrs.			10	77.29	92					Light grey with occasional creamy yellow patches and dark brown stained discontinuities firm sandy SILT. Highly decomposed porphyritic TUFF.	IV

Legend :
W.R. Water return
Large disturbed sample
Undisturbed sample
Standard penetration test
Piezometer tip
Mazier sample
Permeability test
m Moisture content

Remarks :
See sheet 3 for standpipe piezometer installation.

Contractor : --
Date started : 2/3/79
Date finished : 10/3/79

Scale : 1:50
Logged by : JLP
Checked by : JLP
Date : 13/8/79

Figure 2.2 - Sheet One of Log of Drillcore Shown in Plates 2.2 to 2.14

DRILLHOLE No : ALC/12
Sheet 2 of 3

Type of drilling : Rotary	Coordinates : E 34071	Feature	: HKHA - Geotechnical Advice
	N 11453	Location	: Ap Lei Chau Site 'B'
Rig : D-1	Angle from horizontal : 90°	Ground level	: +86.50 m.P.D.
Bit : T.C. & Diamond	Bearing : N -- E --	Water table level	: See piezometer sheets

Column headings: Drilling progress | Casing depth, size | Water level | Lugeon value | Notes e.g. Colour water return caving instrumentation | Depth & diameter (metres) | Reduced level m.P.D. | Core recovery % (0–50) | Rate of penetration | R.Q.D. | Fracture index | Legend | Description | Grade

Drilling progress: 5/3 (10.05 to 12.5 m); 6/3 (12.5 to 16.5 m); 7/3 (16.5 to 20 m)

Casing depth, size: NXC at 11.55 m (5/3); NXC at 11.55 m (6/3)

Water level: 1.26 m at 8.00 hrs.; 7.65 m at 19.00 hrs.; 11.48m at 8.00 hrs.; 14.80m at 19.00 hrs.; 15.96m at 8.00 hrs.

Notes: No water return from 10.05 m to 20.00 m

Depth & diameter: HMLC (10.05–11 m); TNW (11–14.5 m); SPT (14.5–14.8 m); TNW (14.8–20 m)

Core recovery %	R.Q.D.	Fracture index
46	0	
88	12	>10
91	73	4
75	0	>10
99	90	3
97	72	7
93	93	0
0 (SPT)	-	-
94	79	3
99	99	0
100	64	5
83	22	6
99	77	4
95	39	9

Legend (depth)	Reduced level m.P.D.	Description	Grade
10.05 m – 11.04 m	76.50 / 76.45 – 75.46	Light grey hard moderately decomposed medium grained porphyritic TUFF. Discontinuities closely spaced.	III
11.04 m – 11.92 m	75.46 – 74.58	Light to dark grey hard slightly to moderately decomposed fine grained slightly porphyritic TUFF. Discontinuities moderately widely spaced.	III/II, II, II, III/II, II
11.92 m – 13.35 m	74.58 – 73.15	Dark grey hard moderately to slightly decomposed porphyritic TUFF. Discontinuities closely to moderately spaced.	III/II, II
13.35 m – 14.40 m	73.15 – 72.10	Light grey to grey green hard slightly decomposed fine TUFF. Discontinuities closely spaced.	II
14.40 m – 20.00 m	72.10 –	Dark grey hard slightly decomposed porphyritic TUFF with occasional inclusions. Discontinuities moderately widely spaced. Moderately to slightly decomposed.	II, V, III/II

Legend :
- W.R. Water return
- Large distrubed sample
- Undistrubed sample
- Standard penetration test
- Piezometer tip
- Mazier sample
- Permeability test
- m Moisture content

Remarks : See sheet 3 for standpipe piezometer installation.

Contractor : --
Date started : 2/3/79
Date finished : 10/3/79

Scale : 1:50
Logged by : JLP
Checked by : JLP
Date : 13/8/79

Figure 2.3 - Sheet Two of Log of Drillcore Shown in Plates 2.2 to 2.14

DRILLHOLE No :	ALC/12
Sheet 3 of 3	

Type of drilling : Rotary
Rig : D-1
Bit : T.C. & Diamond

Coordinates : E 34071
N 11453
Angle from horizontal : 90°
Bearing : N -- E --

Feature : HKHA - Geotechnical Advice
Location : Ap Lei Chau Site 'B'
Ground level : +86.50 m.P.D.
Water table level : See piezometer sheets

Drilling progress	Casing depth, size	Water level	Lugeon value	Notes e.g. Colour water return caving instrumentation	Depth & diameter (metres)	Reduced level m.P.D.	Core recovery %	Rate of penetration	R.Q.D.	Fracture index	Legend	Description	Grade
				Standpipe tip at 24.00 m	20	66.50	79		53	7	21.19 m	Dark grey slightly decomposed porphyritic TUFF with occasional inclusions. Discontinuities moderately spaced.	II
		14.85m at 19.00 hrs.			21	65.31	98		54		22.00 m	Dark grey hard slightly decomposed fine grained TUFF.	II
7/3	NXC at 11.55 m			No water return from 20.00 m to 25.08 m	22 TNW	64.50	97		97	3		Light to dark grey hard slightly decomposed porphyritic TUFF. Discontinuities closely spaced.	II
8/3					23		98		74	5	23.39 m		
					24	63.11	93		42	8		Light grey green hard slightly decomposed fine grained TUFF. Discontinuities very closely spaced.	II
8/3		15.07m at 19.00 hrs.			25	61.42	95		0	>10	25.08 m		
					26 – 30							Drillhole completed.	

Legend :
- W.R. Water return
- Large disturbed sample
- Undisturbed sample
- Standard penetration test
- Piezometer tip
- Mazier sample
- Permeability test
- m Moisture content

Remarks : Standpipe piezometer installation
1. Tip at 24.00 m; perforated at 21.00 m - 24.00 m.
2. Sand/gravel filters at 18.00 m - 25.08 m.
3. Bentonite ball seals at 15.00 m - 18.00 m.
4. Hole grouted from 15.00 m to surface.

Contractor : --
Date started : 2/3/79
Date finished : 10/3/79

Scale : 1:50
Logged by : JLP
Checked by : JLP
Date : 13/8/79

Figure 2.4 - Sheet Three of Log of Drillcore Shown in Plates 2.2 to 2.14

DRILLHOLE No : PPF7
Sheet 1 of 3

Type of drilling : Rotary, waterflush
Rig : DR5
Bit : T.C. and Diamond

Coordinates : E 35448
N 21332
Angle from horizontal : 90°
Bearing : N -- E --

Feature : Slope 11-NW-B/C39
Location : Police Playing Fields
Ground level : +70.00 m.P.D.
Water table level : See piezometer plots

Drilling progress	Casing depth, size	Water level	Lugeon value	Notes e.g. Colour water return caving instrumentation	Depth & diameter	Reduced level m.P.D.	Core recovery %	Rate of penetration	R.Q.D.	Fracture index	Legend	Description	Grade
					metres		0 50	min/m					
28/2	0.50 104 mm			80% water returns	0 NX		40	20				Loose brown silty coarse SAND. Completely decomposed GRANITE.	V
						69.00	42	18			1.00		
				1	1 101 mm		78	12				Loose to dense orange brown silty coarse SAND. Highly decomposed GRANITE.	IV
	2.50 127 mm	0.35 (18.00) 1.45 (08.00)			2		92	16					
1/3	3.00 127 mm				NX		32	14					
	3.59 127 mm	2.00 (12.00) 3.50 (13.00)			3 101 mm	66.41	88	20			3.59		
					4		66	51				Moderately weak yellowish brown coarse grained medium jointed moderately decomposed GRANITE.	III
					5	64.45	78	70			5.55		
	5.50 127 mm	2.01 (18.00)			NX		80	24				Loose to medium dense yellow brown silty coarse SAND. Highly decomposed GRANITE.	IV
2/3	6.00 127 mm	5.50 (08.00)			6 101 mm		23	14					
	7.50 127 mm			2	7		41	17					
	8.00 104 mm				8		56	28					
	9.00 104 mm	1.50 (12.00) 2.51 (13.00)		3	NX 9 101 mm		66	24					
					10		89	15					

Legend :
W.R. Water return
Large disturbed sample
Undisturbed sample
Standard penetration test
Piezometer tip
Mazier sample
Permeability test
m Moisture content

Remarks :

Contractor : --
Date started : 28/2/79
Date finished : 8/3/79

Scale : 1:50
Logged by : HC
Checked by : JLP
Date : 12/3/79

Figure 2.5 - Sheet One of Log of Drillcore Shown in Plates 2.15 to 2.26

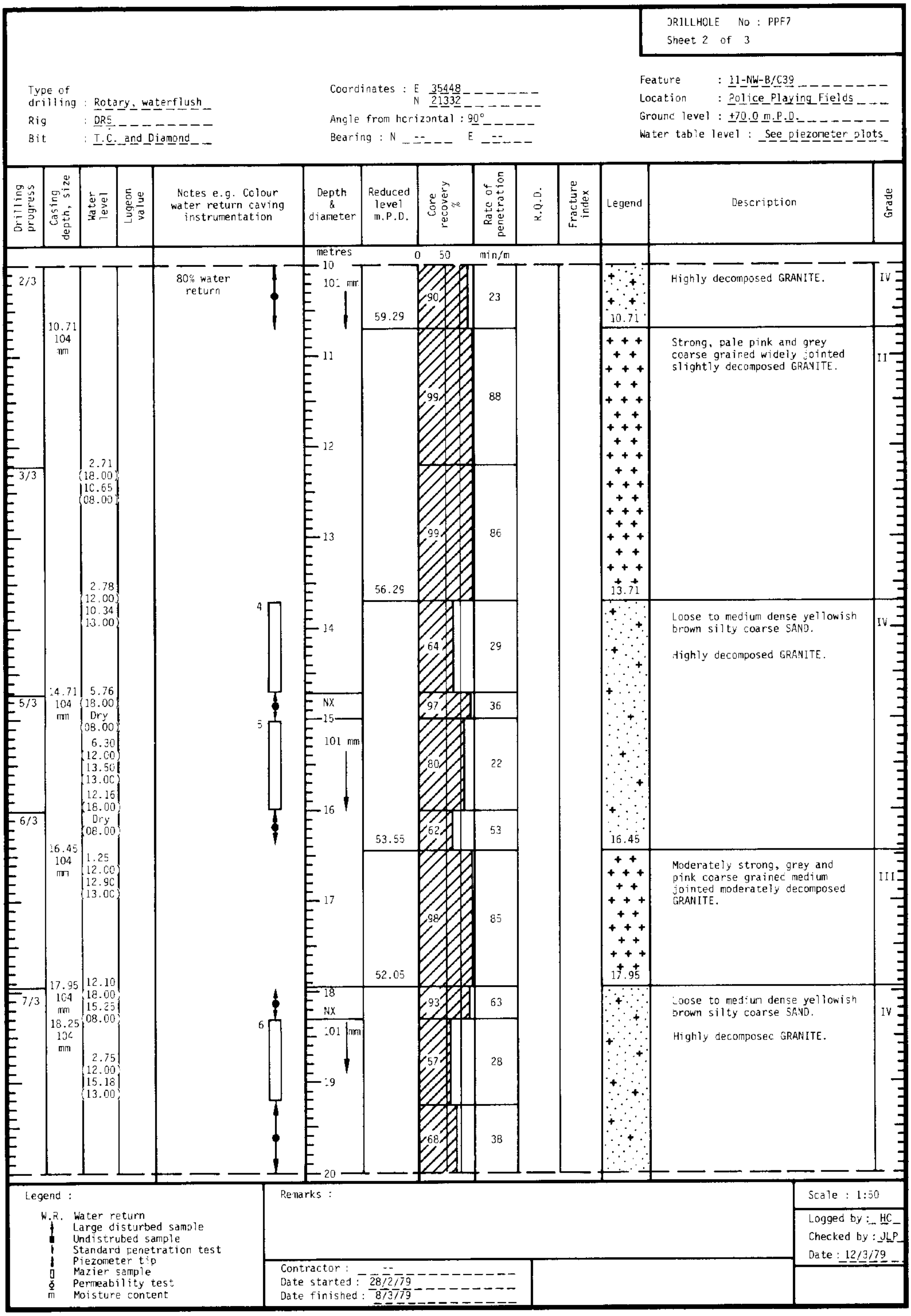


Figure 2.6 - Sheet Two of Log of Drillcore Shown in Plates 2.15 to 2.26

DRILLHOLE No : PPF 7
Sheet 3 of 3

Type of drilling : Rotary, waterflush
Rig : DR5
Bit : T.C. and Diamond

Coordinates : E 35448
N 21332
Angle from horizontal : 90°
Bearing : N -- E --

Feature : 11-NW-B/C39
Location : Police Playing Fields
Ground level : +70.0 m.P.D.
Water table level : See piezometer plots

Drilling progress	Casing depth, size	Water level	Lugeon value	Notes e.g. Colour water return caving instrumentation	Depth & diameter (metres)	Reduced level m.P.D.	Core recovery %	Rate of penetration (min/m)	R.Q.D.	Fracture index	Legend	Description	Grade
					20	49.83	68	38			20.17	See sheet above	IV
7/3	20.17 104 mm			60% water returns; Piezometer tip at 20.17 m	101 mm	48.77	100	105	95		21.23	Strong, pale pink and grey medium jointed slightly decomposed GRANITE.	II
8/3		13.25 {18.00}, 17.37 {08.00}			21	48.00				3	22.00	Moderately strong grey and pink coarse grained medium jointed moderately decomposed GRANITE.	III
		9.60 {12.00}, 14.93 {13.00}			22		97	95	81			Strong, pale pink and grey medium jointed slightly decomposed GRANITE.	II
					23, 24		96	87	96	1			
		13.40 {18.00}			25	44.05	93	93	88		25.95		
					26–30							Hole completed at 25.95 m	

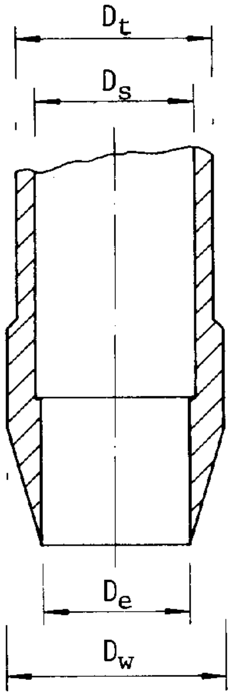
Legend :
W.R. Water return
Large distrubed sample
Undisturbed sample
Standard penetration test
Piezometer tip
Mazier sample
Permeability test
m Moisture content

Remarks : Piezometer installation
1. Piezometer tip at 20.17 m.
2. Sand filter from 20.32 to 19.32 m.
3. Bentonite ball seal from 22.32 to 20.32 m and 19.32 to 17.32 m.
4. Grouted from 25.95 to 22.32 m and 17.32 to surface.

Contractor : --
Date started : 28/2/79
Date finished : 8/3/79

Scale : 1:50
Logged by : HC
Checked by : JLP
Date : 12/3/79

Figure 2.7 - Sheet Three of Log of Drillcore Shown in Plates 2.15 to 2.26



Area ratio $= \dfrac{\text{(Net projected area of sampler)}}{\text{(Projected area of sample core)}}$

$$= \frac{D_w^2 - D_e^2}{D_e^2}$$

Inside clearance $= \dfrac{\text{(ID of sample tube)} - \text{(Dia. at cutting edge)}}{\text{(Dia. at cutting edge)}}$

$$= \frac{D_s - D_e}{D_e}$$

Outside clearance $= \dfrac{\text{(OD of shoe)} - \text{(OD of sample tube)}}{\text{(OD of sample tube)}}$

$$= \frac{D_w - D_t}{D_t}$$

Figure 2.8 - Definition of Sampler Proportions

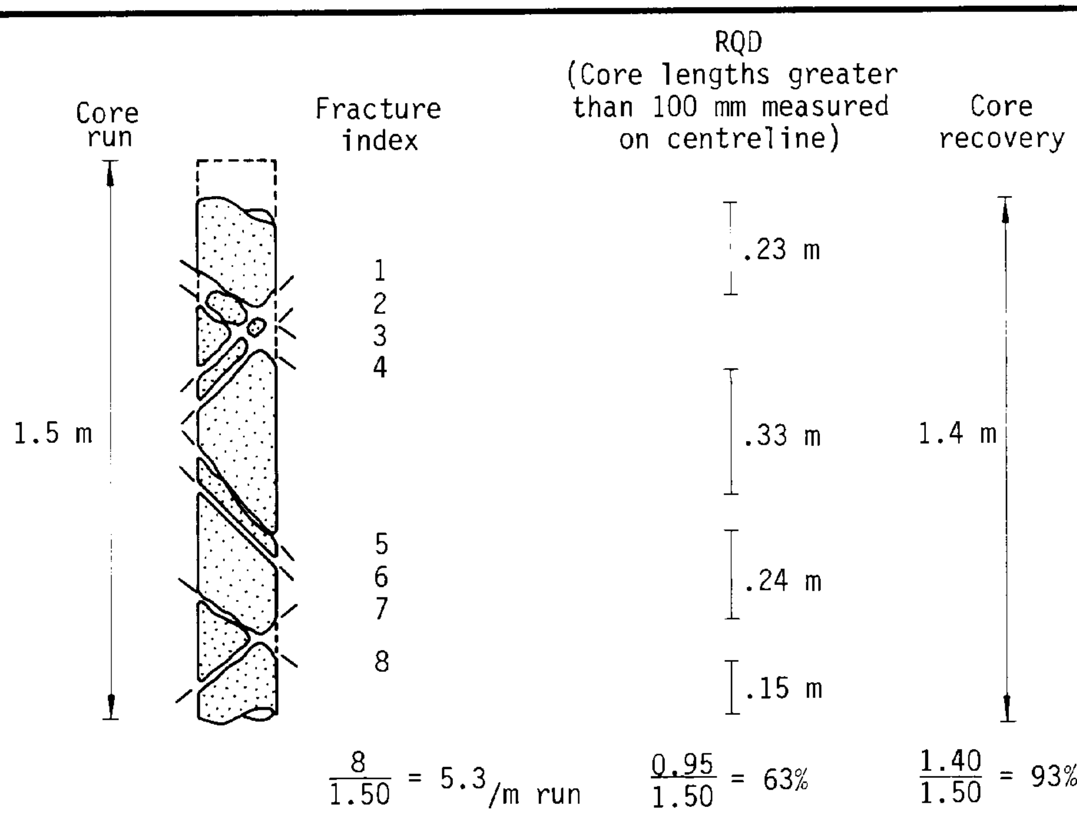


Figure 2.9 - Core Recovery and Fracture Indices

GENERAL INFORMATION

Seq. No. 7489 | Site ANYWHERE | Date (Day Month Year) 10 03 76 | Operator AC | Method of location 3 | Co-ordinates or chainage (metres): Northing or Chainage | Eastings | Elevation

Method of location: 1. By co-ordinates 2. Chainage 3. On attatched map/drawing/photograph

Locality type 2 | Size of locality 1 | No. of supplementary sheets of discontinuity data 12 | Sketch 1 | Photograph 1 | Slope dip 80 | Dip direction 100

Locality type: 1. Natural exposure 2. Construction excavation 3. Tunnel

Size of locality: 1. $>10 m^2$ 2. $5-10m^2$ 3. $1-5 m^2$ 4. $<1m^2$ 5. Line survey

Sketch / Photograph: 0. No. 1. Yes.

Remarks: 32 m high. Signs of instability. Some wedge failures have occured.

ROCK MATERIAL INFORMATION

Colour 119 | Grain size 2 | Compressive strength 4 | Rock type GRANITE

Colour: 1. Light 2. Dark; 1. pinkish 2. reddish 3. yellowish 4. brownish 5. olive 6. greenish 7. bluish 8. greyish; 1. pink 2. red 3. yellow 4. brown 5. olive 6. green 7. blue 8. white 9. grey 10. black

Grain size: 1. Very coarse (>60 mm) 2. Coarse (2 - 60mm) 3. Medium (60μ - 2mm) 4. Fine (2 - 00μ) 5. Very fine (<2μ)

Compressive strength: 1. Very weak - can be broken in the hand 2. Weak - crumbles unber firm blows with a pick 3. Mod. strong - indents with a pick 4. Strong - breaks with single hammer blow 5. Very strong - requires several hammer blows to break

Qualifying terms to describe rock: Slightly weathered, moderately weathered on joints.

ROCK MASS INFORMATION

Fabric 1 | Block size 2/3 | State of weathering 2 | No. of major discontinuity set 3

Fabric: 1. Blocky 2. Tabular 3. Columner

Block size: 1. Very large ($>8m^3$) 2. Large ($0.2 - 8m^3$) 3. Medium ($0.008 - 0.2 m^3$) 4. Small ($0.0002 - 0.008m^3$) 5. Very small ($0.0002 m^3$)

State of weathering: 1. Fresh 2. Slightly 3. Moderately 4. Highly 5. Completely 6. Residual soil

LINE SURVEYS TO DETERMINE DISCONTINUITY SPACINGS

	Plunge of line	Trend of line	Length of line (metres)	No. of fractures	Spacing	Remarks
Line 1	00	10	60	47	2/3	Base of slope
Line 2	00	10	48	40	2/3	Berm level
Line 3	80	100	32	50	2/3	Centre line of slope

Discontinuity spacing: 1. Ext. wide (<2m) 2. Very wide (600mm - 2m) 3. Wide (200 - 600mm) 4. Mod. wide (60 - 200 mm) 5. Mod. narrow (20 - 60mm) 6. Narrow (6 - 20mm) 7. Very narrow (<6mm)

Figure 2.10 - Example of Description Sheet for Rock Mass Survey

GENERAL INFORMATION

Seq No. 7 9 8   Site ANYWHERE   Date (Day Month Year) 10 03 76   Operator AC   Discontinuity data sheet No. 2 of 12

NATURE AND ORIENTATION OF DISCONTINUITIES

Chainage or No.	Type	Dip	Dip direction	Persistence	Aperture	Infilling	Consistency	Roughness	Differentially weathered zone width mm	Waviness wavelength	Waviness amplitude	Water	Remarks
1113	2	56	178	3.5	4	4	2		150	2.0	.05	1	
1114	2	86	313	1.5	5	2	-		50	1.0	.04	1	
1115	2	86	235	.9	7	2	-		20	-	-	1	
1116	2	66	176	.7	6	6	5		40	-	-	1	
1117	2	84	246	4.3	4	4	2		120	2.0	.05	1	Shot hole shows 50mm movement
1118	2	55	149	2.4	5	2	-		30	1.5	.10	1	
1119	1	44	64	25.0	3	4	4		200	3.0	.08	2	
1120	2	90	144	1.9	7	6	5		20	0.5	.05	1	
1121	2	82	234	3.3	4	2	-		100	1.5	.03	1	
1122	2	81	232	.4	7	2	-		20	-	-	1	
1123	2	62	144	1.3	5	2	-		50	1.0	.05	1	
1124	1	80	263	18.0	3	4	4		200	4.5	.20	3	

Type
0. Fault zone
1. Fault
2. Joint
3. Cleavage
4. Schistosity
5. Shear
6. Fissure
7. Tension crack
8. Foliation
9. Bedding

Dip, Dip direction
(Expressed in degrees)

Persistence
(Expressed in metres)

Aperture
1. Wide (>200 mm)
2. Mod. wide (60 - 200 mm)
3. Mod. narrow (20 - 60 mm)
4. Narrow (6 - 20 mm)
5. Very narrow (2 - 6 mm)
6. Ext. narrow (<2 mm)
7. Tight

Nature of infilling
1. Clean
2. Surface staining
3. Non - cohesive
4. Cohesive
5. Cemented
6. Calcite
7. Chlorite, talc
8. Others - specify

Consistency of infilling

Soil strength
1. Soft
2. Firm
3. Stiff
4. Hard

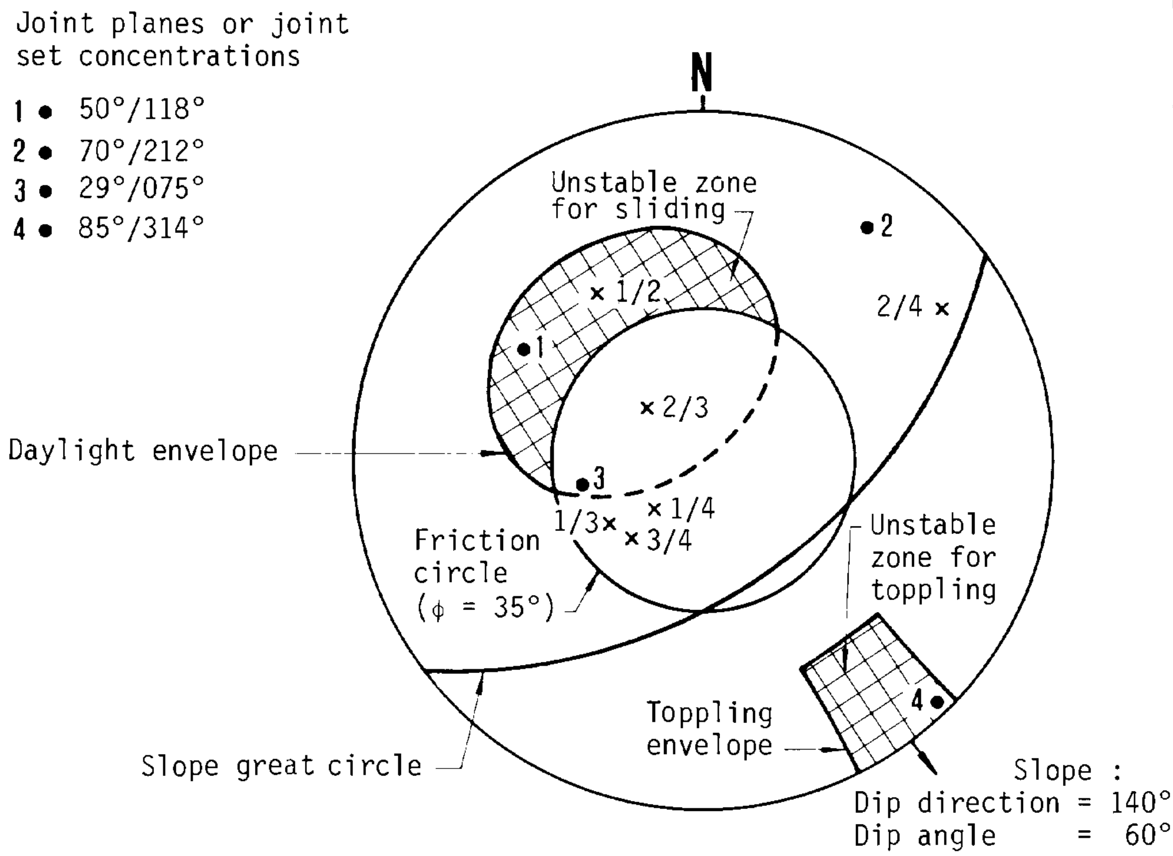
Rock strength
5. Weak
6. Mod. strong
7. Strong
8. Very strong

Roughness
1. Polished
2. Slickensided
3. Smooth
4. Rough
5. Defined ridges
6. Small slope
7. Very rough

Waviness
Express wavelength & amplitude in metres

Water
1. Dry
2. Seepage
3. Slight flow <0.1 l/sec
4. Mod. flow 0.1 - 1 l/sec
5. High flow >0.1 l/sec

Figure 2.11 - Example of Data Sheet for Discontinuity Survey



Interpretation :

UNSTABLE
- Plane 1 daylights and dips at more than 35°
- Plane 4 may result in toppling
- Planes 1 and 2 combine to form a daylighting wedge that dips at more than 35°

STABLE
- Plane 2 does not daylight
- Intersections 1/4, 1/3, 2/4, 3/4 do not daylight

QUESTIONABLE
- Plane 3 and intersection 2/3 daylight but dip at less than 35°. Would be unstable if friction angle less or water pressure reduces strength.
- If pole 3 represents concentration of set, check that individual points do not daylight or form wedges.

Legend :

● 1 Pole of plane (plane 1)
× 1/2 Pole of line of intersection (planes 1 and 2)

Note : (1) Water is not taken into account
(2) Cohesion and characteristics of individual joints are not taken into account
(3) Wedges may not actually be developed in the field
(4) Results should be checked by a site visit

Figure 2.12 - Example of a Stereoplot

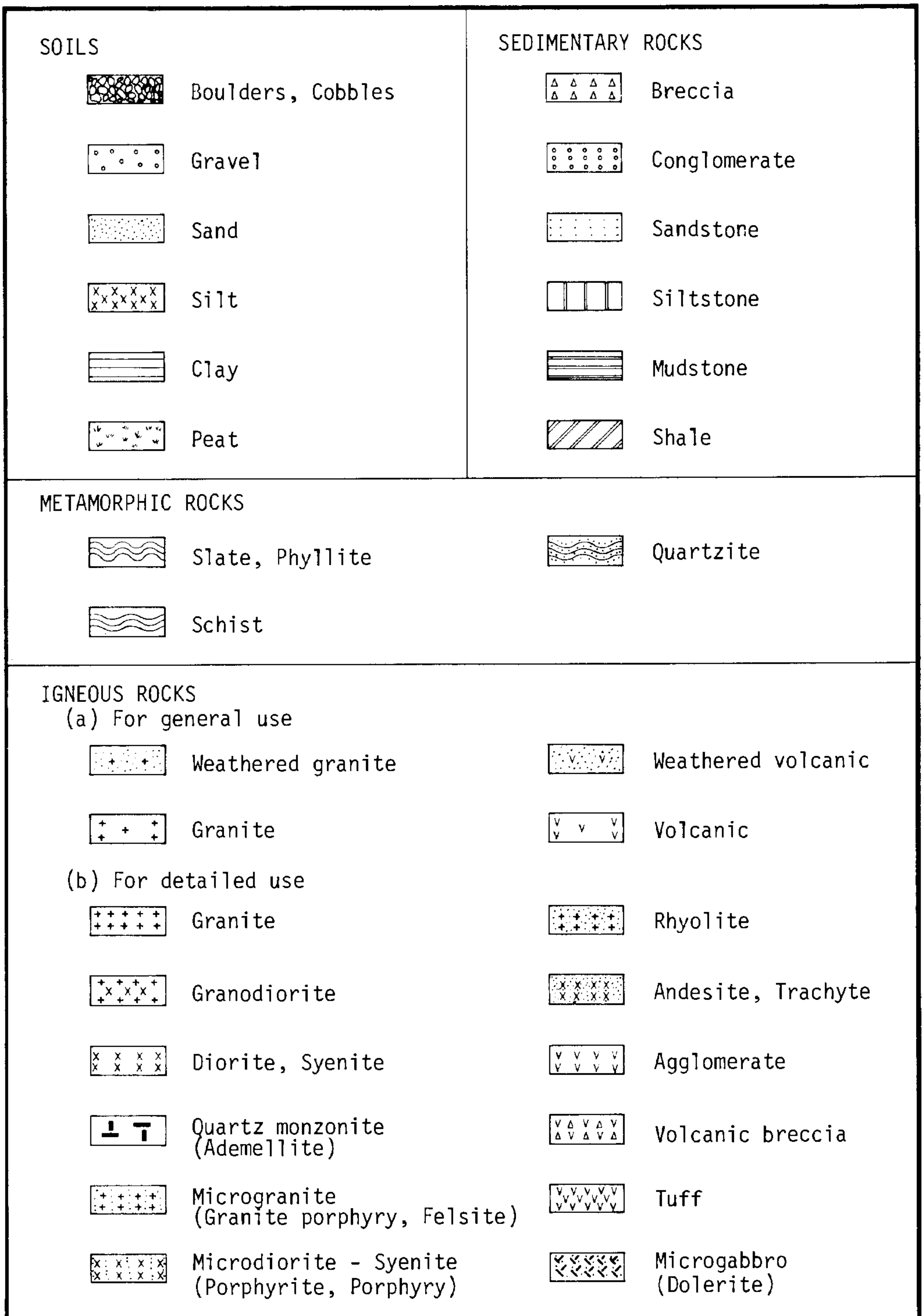


Figure 2.13 - Legend for Use on Logs

TRIAL PIT NO. : TPN 6
FACE C Sheet 3 of 4

Type of excavator	: Hand dug	Contractor	:	Study area	:
Type of pump (if used)	: Nil	Date dug	: 17/3/82	Location	:
Timbering	: Nil	Date backfilled	: 2/4/82	Ground level	: 87.09 m.P.D.
				Coordinate	: E 42381 N 19234.75

Water Conditions	Depth (sample & test)	Reduced level	Depth (metres)	Profile of face C (Width = m)	Description	Grade
			0		Loose yellowish brown and greyish brown silty fine to medium SAND with some angular gravel of concrete and pieces of glass, plastic and other general rubbish; roots throughout. (FILL)	
		86.69	0.4		Loose to medium dense brown textureless sandy SILT with yellow-white sub-rounded cobbles and gravel of moderately and highly decomposed granite with iron-stained patinas; roots throughout (SLOPEWASH). Top 100 mm is greyish brown humic sandy silt, old topsoil.	
	U100 (0.8)		0.5			
Damp to 1.2 m Dry		85.89	1.2		Very dense yellowish brown-white SAND (HIGHLY DECOMPOSED GRANITE) with corestones of very weak highly to moderately decomposed GRANITE and with relict joints; dry.	IV with some III/IV
			1.5			
Dry to 2.0 m		85.09	2.0		Trial pit complete at 2.0 m depth as instructed.	
			2.5			
			3.0			
			3.5			
			4.0			
			4.5			
			5.0			

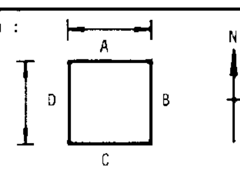
Legend :

- ● Small disturbed sample
- Large disturbed sample
- Undisturbed sample, vertical
- ▬ Undisturbed sample, horizontal
- Block sample
- ▲ Water sample
- Insitu density test
- m Moisture content

Remarks :

Large granite corestone is exposed adjacent to Face D. Squatter platform adjacent to Face A.

Plan :



Scale : 1:25

Logged by : TG
Checked by :
Date : 18/3/82

Fig. No. :

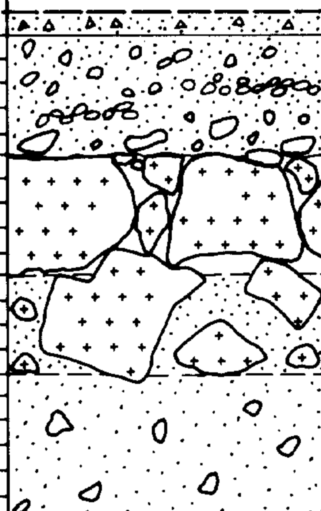
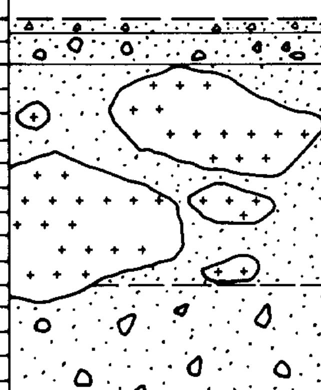
Figure 2.14 - Trial Pit Log - Example 1

TRIAL PIT NO. : TPN 9
FACE B and D Sheet 2 of 2

Type of excavator : Hand dug
Type of pump (if used) : Nil
Timbering : Nil

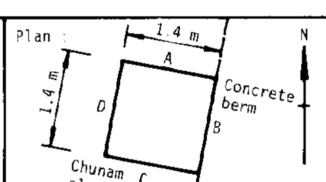
Contractor :
Date dug : 16/3/82
Date backfilled : 2/4/82

Study area :
Location :
Ground level : 83.71 m.P.D.
Coordinate : E 42376.14 N 19198.55

Water conditions	Depth (sample & tests)	Reduced level	Depth (metres)	Profile of face B Width = 1.4 m	Description	Grade
			0		Concrete (surface of berm)	
		82.57	0.5 0.6		Compacted light brown fine to coarse SAND with some angular gravel of slightly decomposed granite; layered structure with horizontal layers of angular gravel up to 70 mm thick; some angular cobbles of granite from 0.4-0.6 m depth. (FILL)	
		82.07	1.0 1.1		Large angular BOULDERS of pink with iron staining slightly decomposed granite; boulders are in point contact with voids between. (FILL)	
Dry to 1.5 m		81.67	1.5		Large angular BOULDERS and some cobbles of slightly decomposed granite in a matrix of medium dense light brown fine to coarse SAND. (FILL)	
Damp Damp to 2.1 m	U100 ■ ɣ (2.0)	81.07	2.1 2.0		Dense light brown fine to coarse SAND with some angular gravel of moderately and highly decomposed granite; no layering. (FILL; derived from decomposed granite) Fill is well compacted; excavation by pick resulted in groove marks.	
					Trial pit complete at 2.1 m depth as instructed.	
				Profile of face D Width = 1.4 m		
					Chunam, 10 mm thick, over sand/lime mortar, 25 mm thick, over light brown SAND with some gravel. (FILL)	
					Larger angular BOULDERS and some cobbles of slightly decomposed granite in a matrix of medium dense light brown fine to coarse SAND. (FILL)	
Damp to 1.6 m					Dense light brown fine to coarse SAND with some angular gravel of moderately and highly decomposed granite. (FILL)	
			5.0			

Legend :
- ● Small disturbed sample
- Large disturbed sample
- Undisturbed sample, vertical
- ▬ Unidsturbed sample, horizontal
- ▣ Block sample
- ▲ Water sample
- ɣ Insitu density test
- m Moisture content

Remarks :
Voids in boulder layer appear to be present beneath berm only.

Plan :


Scale : 1:25

Logged by : TG
Checked by :
Date : 18/3/82

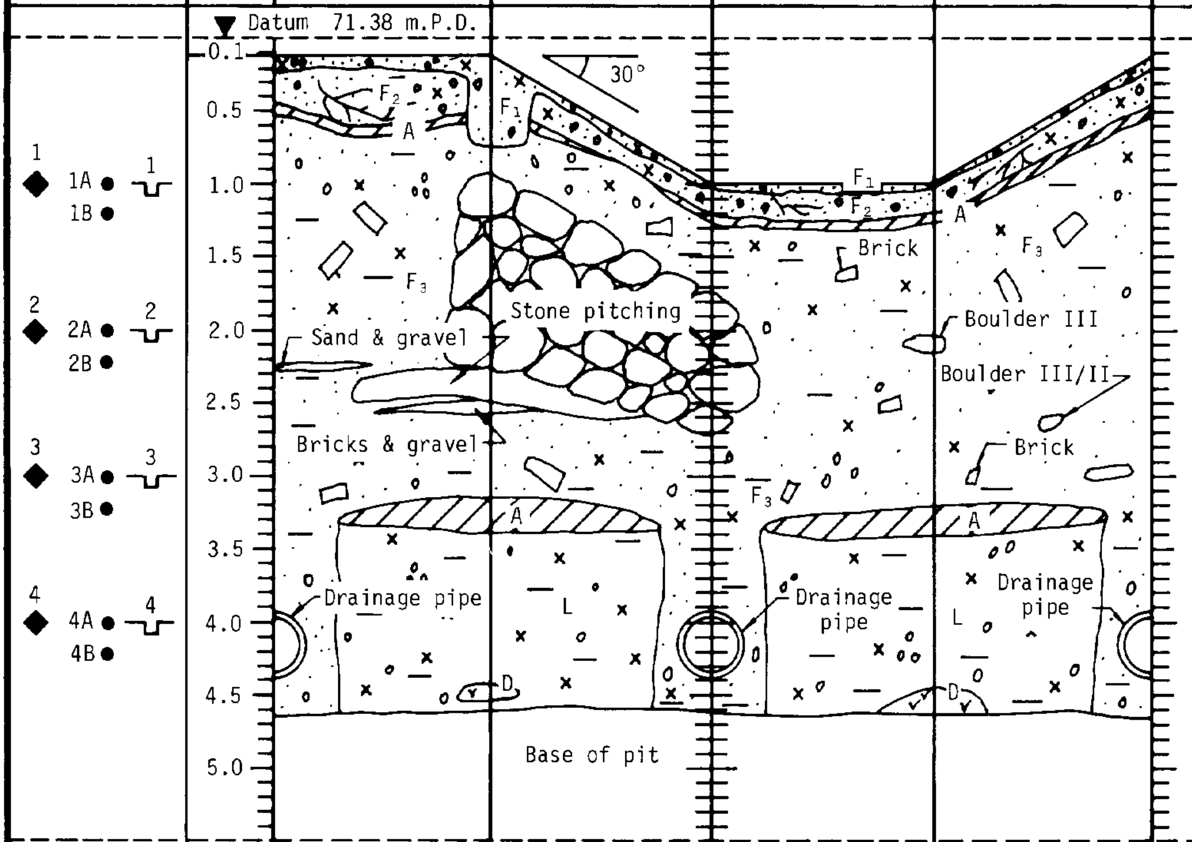
Fig. No. :

Figure 2.15 - Trial Pit Log - Example 2

Slope No.: 11SW-B/CR685 Location : Garden Road	TRIAL PIT No. TP - 6 Sheet 1 of 1

Logged by : ABC	Excavation method: Hand (Timber shoring full height)	Date Excavated : 1.1.1983
Date : 5.1.83	Co-ordinates: E. 34444.44 N. 15555.55	Date Backfilled: 20.1.1983

Samples & Tests	Depth m.	FACE A : width 1.60 m	FACE B : width 1.40 m	FACE C : width 1.60 m	FACE D : width 1.40 m



Legend	Description	Grade
F1	Loose, light brown gravelly silty SAND (FILL)	
F2	Loose, light brown silty SAND with some gravel and many roots (FILL)	
F3	Loose to medium dense, reddish brown, clayey silty SAND with some gravel and pieces of glass and brick (FILL)	
A	Soft dark brown, organic, clayey SILT with trace of gravel (OLD TOP SOIL)	
L	Soft, reddish brown gravelly, silty CLAY (COLLUVIUM MATRIX). Slakes easily. Hand Penetrometer strength = 40 kPa. No rebound to N Schmidt hammer	
D	Moderately decomposed (grade III) sub-angular and angular cobbles of volcanic rock (COLLUVIAL FRAGMENT) N Schmidt hammer rebound value 32	III

Plan (not to scale)

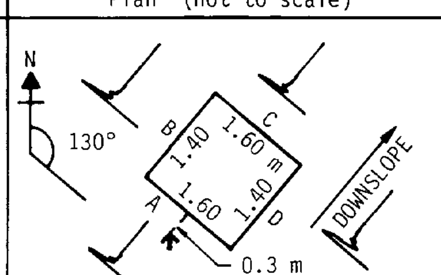


Plate bearing test	Large dist. sample
Insitu density test	Undist. sample, hori.
Moisture content test	Undist. sample, vert.
Water sample	Block sample
Seepage	Photograph
Small dist. sample	Bench mark

Contractor : CONTRACTOR NAME Works Order No. : 2/1/65	ENGINEER NAME

Figure 2.16 - Trial Pit Log - Example 3

Study area :	DRILLHOLE NO. : LTN 11
Location :	Sheet No. : 1 of 2
Contractor :	Logged by : TG
Job No. :	Date of works : 22/2/82 to 25/2/82
Co-ordinates : E 42497.74 N 19259.55	
Ground level : 91.6 m.P.D.	

Daily progress	Core barrel	Depth of Casing (size) m (mm)	Field tests, samples and instrumentation	Core Rec. % (0 50)	Core RQD %	Core FI	Legend	Depth m	Reduced level m.P.D.	Description of strata	Grade
22 - 2			Inspection pit excavated to 1.5 m depth.					1.0 2.0 3.0 4.0 5.0		Concrete slab, 100 mm thick. Recovered as angular COBBLES and large GRAVEL of medium to coarse grained slightly decomposed granite; matrix lost. (FILL)	
								5.4	86.2		
		6.5 (140)						6.0 7.0		Nil recovery. (FILL (?); probably previous cultivation terrace)	
			SPT-1 N=21 (7.5-8.0)					7.5	84.1	Brown and black clayey sandy SILT with traces of relict granite texture (COMPLETELY DECOMPOSED GRANITE).	
22 - 2		8.0	• -1								VI
23 - 2		(120)	Piezometer LTN 11a					8.2	83.4	Strong pink with iron staining coarse to medium grained widely jointed slightly decomposed GRANITE.	
	T2-101	8.2 (120)		76	0	∞		9.0		Moderately weak slightly to moderately decomposed zone at 8.2-8.7 m, recovered as angular cobble-size core pieces.	III
				100	93	nil		10.0		Sub-vertical incipient black stained joints from 8.7-9.5 m; from 10.8-11.0 m are very close spaced planar iron-stained sub-horizontal joints.	II

Remarks :

From site formation drawing No. A/30794, original ground level before construction of fill platform and playground was approx. 86 m.P.D.

Morning/evening water levels

Date	22-2	22-2	23-2	23-2	24-2	24-2		
BH Depth	-	8.0	8.0	13.1	13.1	18.1		
Casing	-	8.0	8.0	12.1	12.1	12.1		
Water	-	3.0	2.9	3.5	7.8	7.6		

Legend :

WR	Water return
•	Small disturbed sample
	Large disturbed sample
U76, 100	Undisturbed drive samples of 76 mm or 100 mm dia. (blow count, depth)
M	Mazier sample
SPT	Standard penetration test N value; (blow count/pen., depth)
	Permeability test

Type of boring/drilling :

Rotary water flush

Diameter of boring/drilling :

0.00 - 6.50 m	140 mm
6.50 - 8.20 m	120 mm
8.20 - 11.50 m	101 mm
11.50 - 12.10 m	89 mm
12.10 - 18.10 m	75 mm

Casing tubes :

0.00 - 6.50	Px
6.50 - 8.20	106
8.20 - 12.10	Nx

Figure 2.17 - Example of Sheet One of Drillhole Log

Study Area :	DRILLHOLE NO. : LTN 11
Location :	Sheet No. : 2 of 2
Contractor :	Logged by : TG
Job No. :	Date of Works : 22/2/82 to 25/2/82
Co-ordinates : E 42497.74 N 19259.55	
Ground level : 91.6 m.P.D.	

Daily progress	Core barrel	Depth of casing (size)	Sample: Field tests, samples and instrumentation	Core Rec. %	Core RQD. %	Core FI	Legend	Depth	Reduced level	Description of strata	Grade
		m (mm)		0 50				m	m.P.D.		
	T2-101			100	100	nil	+ + +	11.0	80.6	Slightly decomposed GRANITE (see Sheet 1 for details).	II
			SPT-2 (17/300) (11.5-11.8) ● -2	38	nil	3.75				Nil recovery from 11.0-11.5 m and 11.8-12.1 m. Jar sample No. 2 contains pink SAND with some silt; relict granite texture. (COMPLETELY DECOMPOSED GRANITE)	V
				nil	-	-		12.1	79.5		
	TNW	12.1 (89)		86	35	6.0		12.5	79.1	Strong pink coarse to medium grained mainly widely jointed slightly decomposed becoming fresh below 13.2 m, GRANITE. Moderately/highly decomposed zone at 12.5-12.8 m recovered as angular cobbles. Sub-horizontal joint with grade III fringes at 13.1 m; planar smooth tight joints, 40°, at 12.95, 13.2 and 14.8 m; close jointed zone, undulating, smooth to rough, black-stained at 15.0-15.5 m. Nil joints 15.5-17.1 m.	II
23 - 2								12.8	78.8		III/IV
24 - 2				89	73	1.11		14.0			II
				100	100	nil		15.0		Sub-vertical incipient joints with chlorite vein infill, 1-2 mm thick, at 17.1-17.6 m; undulating, iron-stained, tight, sub-horizontal joints at 17.1 m and 17.2 m; planar, smooth, tight joint with kaolinite coating at 18.0 m.	I
				100	78	5.0		16.0			
				100	97	1.29		17.0			
			Piezometer LTN 11b	100	100	nil		18.1	73.5		
24 - 2		12.1 (89)									
										Drillhole complete at 18.1 m depth.	

Remarks :

Piezometer LTN 11b installed at 17.5 m depth below ground surface with sand filter from 18.1 m to 10.5 m, bentonite seal from 10.5 m to 8.5 m; piezometer LTN 11a installed at 8.0 m depth below ground surface with sand filter from 8.5 m to 6.5 m, bentonite seal from 6.5 m to 4.5 m and cement-bentonite grout from 4.5 m to ground surface.

Morning/evening water levels

Date	
BH depth	See Sheet 1
Casing	
Water	

Legend :

WR	Water return
●	Small disturbed sample
	Large disturbed sample
U76,100	Undisturbed drive samples of 76 mm or 100 mm dia. (blow count, depth)
M	Mazier sample
SPT	Standard penetration test N value; (blow count/pen., depth)
	Permeability test

Type of boring/drilling :

Rotary water flush

Diameter of boring/drilling :

0.00 - 6.50 m	140 mm
6.50 - 8.20 m	120 mm
8.20 - 11.50 m	101 mm
11.50 - 12.10 m	89 mm
12.10 - 18.10 m	75 mm

Casing tubes :

0.00 - 6.50 m	Px
6.50 - 8.20 m	106
8.20 - 12.10 m	Nx

Figure 2.18 - Example of Sheet Two of Drillhole Log

FIELD DATA FROM WATER ABSORPTION TEST
DRILLHOLE No. T3 TEST No. 4

Date of test 24/11/75	Tested by MKF
Packer type (delete as necessary)	Tested section from 19.81 m to 22.86 m
~~single~~/double	Depth of hole at time of test 33.84 m
pneumatic/~~hydraulic/mechanical~~	Details of casing at time of test -
Packer pressure	Gauge height above ground level 1.32 m
Depth to centre of test section	(measured down line of drillhole 21.34 m

Depth to groundwater level (measured down line of drillhole)

FIRST PERIOD Gauge pressure 124 kPa

Time (minutes)	0	5	10	15		Average Flow (l/min)
Flowmeter ~~Dipstick~~ reading (l)	218.6	229.3	239.9	250.7		
Water take (l)	10.7	10.6	10.8			2.14

SECOND PERIOD Gauge pressure 248 kPa

Time (minutes)	0	5	10	15		Average Flow (l/min)
Flowmeter ~~Dipstick~~ reading (l)	281.8	296.4	311.2	326.3		
Water take (l)	14.6	14.8	15.1			2.96

THIRD PERIOD Gauge pressure 372 kPa

Time (minutes)	0	5	10	15		Average Flow (l/min)
Flowmeter ~~Dipstick~~ reading (l)	255.9	276.6	297.5	318.5		
Water take (l)	20.7	20.9	21.0			4.17

FOURTH PERIOD Gauge pressure 248 kPa

Time (minutes)	0	5	10	15		Average Flow (l/min)
Flowmeter ~~Dipstick~~ reading (l)	54.5	69.9	85.4	101.1		
Water take (l)	15.4	15.5	15.7			3.10

FIFTH PERIOD Gauge pressure 124 kPa

Time (minutes)	0	5	10	15		Average Flow (l/min)
Flowmeter ~~Dipstick~~ reading (l)	377.3	388.6	400.0	411.5		
Water take (l)	11.3	11.4	11.5			2.28

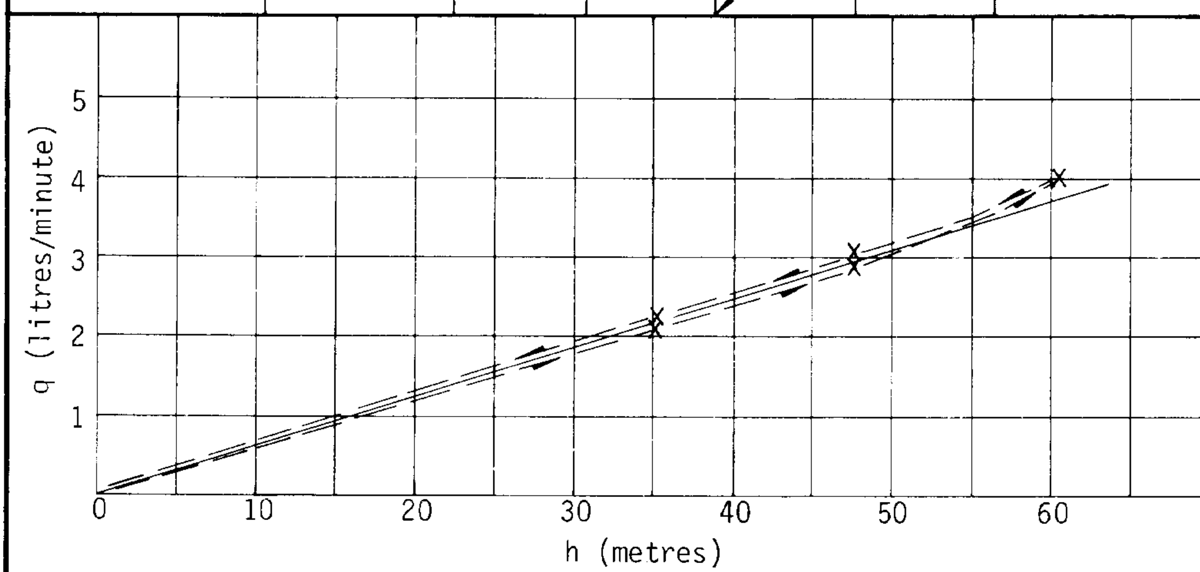
Figure 2.19 - Example of Field Sheet for Water Absorption Test

WATER ABSORPTION TEST

DRILLHOLE No. T3 TEST No. 4

Date of test 24/11/75	Test section from 19.81 m to 22.86 m
Packer type (delete as necessary)	Depth of hole at time of test 33.84 m
~~single~~/double	Diameter of hole in test area 102 mm
pneumatic/~~hydraulic/mechanical~~	Drillhole inclination from horizontal 90°
Packer pressure	Casing details -
	Rock type GRANITE GRADE II

LEGEND OF TEST SECTION 3.05 m	FLOW	GUAGE PRESSURE		FRICTION HEADLOSS		TOTAL HEAD
	q litres/min	units : kPa	head of water m	in basic pipe work m	in extra rods or pipes m	h (2+3+6-7-8) m
(1)	(4)	(5)	(6)	(7)	(8)	(9)
VERTICAL DEPTH TO GROUND-WATER FROM G.L. 21.34 m(2)	2.14 2.96 4.17	124 248 372	12.6 25.2 37.8	Negligible	Negligible	35.26 47.86 60.46
HEIGHT OF PRESSURE GAUGE ABOVE G.L. 1.32 m(3)	3.10 2.28	248 124	25.2 12.6			47.86 35.26



FROM GRAPH : $^{q}/_{h}$ = 3.4/54

$L = \frac{100}{lh} q$ = 2.06 lugeon units

where l = length of test section in metres

Tested by	Calculated by
	MKF

Note : If groundwater level unknown or below test section use depth to centre of test section.

Figure 2.20 - Example of Calculation Sheet for Water Absorption Test

FALLING-HEAD PERMEABILITY TEST

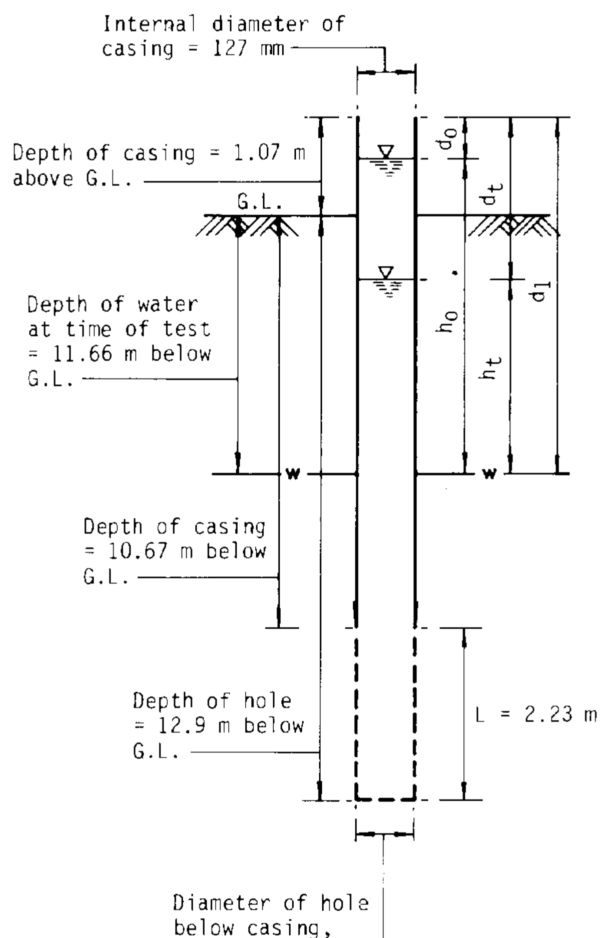
FIELD DATA :

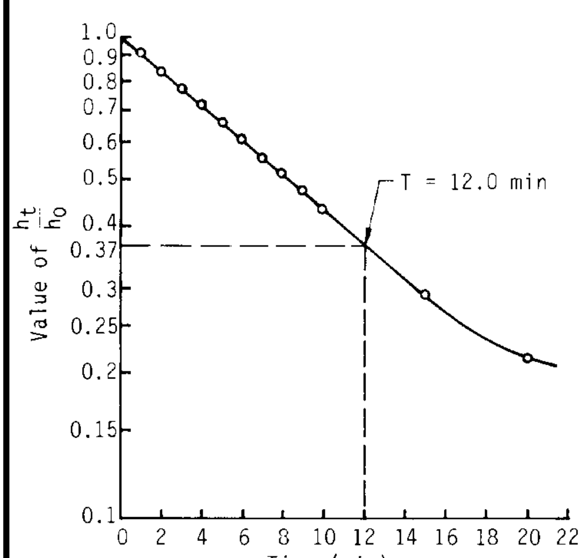
Time on clock	Time elapsed min	sec	Depth of water below top of casing = d_t	$h_t = (d_1 - d_t)$	$\frac{h_t}{h_o}$
	.	0	9.601 m	3.129 m	1.000
	1	0	9.854	2.876	0.919
	2	0	10.109	2.621	0.838
	3	0	10.300	2.430	0.777
	4	0	10.484	2.246	0.718
	5	0	10.668	2.062	0.659
	6	0	10.826	1.904	0.608
	7	0	10.985	1.745	0.558
	8	0	11.100	1.630	0.521
	9	0	11.227	1.503	0.480
	10	0	11.366	1.364	0.435
	15	0	11.824	0.906	0.290
	20	0	12.065	0.665	0.212

Borehole .... Date ....

Drillhole ...D7... Observer ...ABC...

Use only CLEAN water for the test.
Has water been added during boring? ~~yes~~/No





CALCULATIONS :

$$K = \frac{A}{FT},$$

where : $A = \frac{0.140^2\pi}{4} = 0.01539\ m^2$

$F = 2.5$ (based on case (d) in Figure 7 in B.S. 5930 : 1981)

$T = 12$ min x 60 = 720 sec

therefore : $K = \frac{A}{FT} = \frac{0.01539}{2.5 \times 720} = 8.5 \times 10^{-6}$ m/s

Figure 2.21 - Example of Results and Calculation Sheet for Falling-head Test

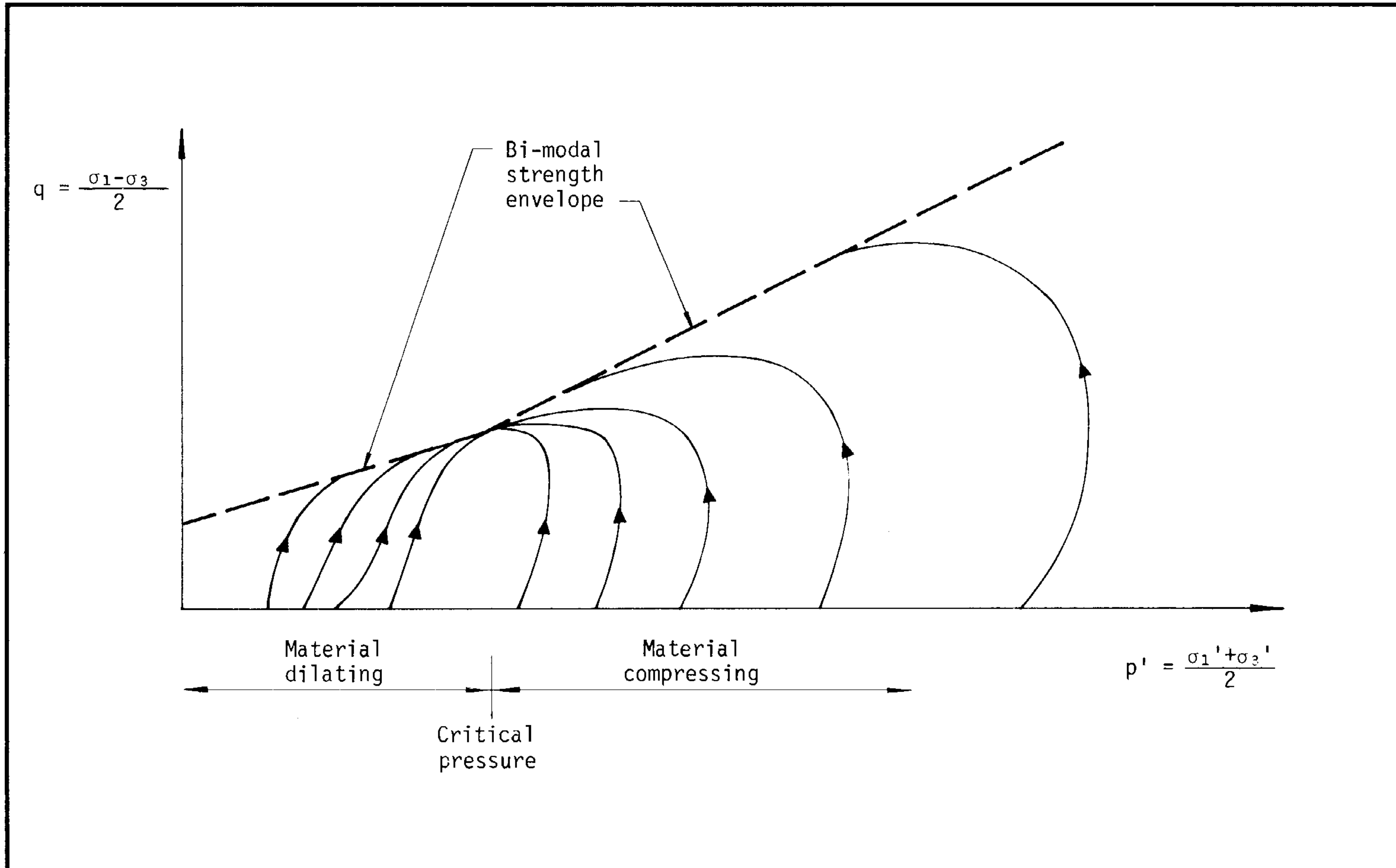


Figure 3.1 - Typical Stress Paths and Strength Envelope for Undrained (CU) Triaxial Tests

TRIAXIAL COMPRESSION TEST - SUMMARY OF SOIL PROPERTIES

Consolidated undrained with p.p.m. Single stage

Project Ref. B.H.No. A

Sample No., Type & Nominal Dia. 2/M/76 Depth 5.7-6.3 m

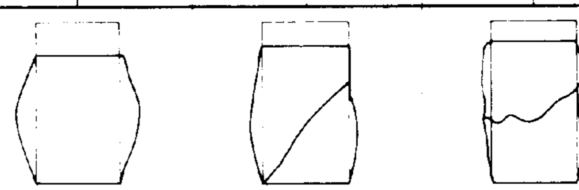
SPECIMEN NO.		1		2		3	
Effective cell pressure, σ_3 kPa		35		70		115	
		Before	After	Before	After	Before	After
Diameter of specimen	mm	76.0		76.0		76.0	
Length of specimen	mm	152.0		152.0		152.0	
VOLUME OF SPECIMEN	cm^3	689.54	664.71	689.54	667.04	689.54	670.94
Wet mass of specimen	g	1302.1	1308.1	1313.7	1313.7	1318.7	1318.1
Dry mass of specimen	g	1024.1		1031.9		1037.1	
Mass of moisture	g	278.0	284.0	281.8	282.0	281.6	281.0
MOISTURE CONTENT	%	27.1	27.7	27.3	27.3	27.2	27.1
WET DENSITY	Mg/m^3	1.89		1.91		1.91	
DRY DENSITY	Mg/m^3	1.49	1.54	1.50	1.55	1.50	1.55
Specific gravity		2.69		2.68		2.66	
VOID RATIO		0.811	0.746	0.791	0.732	0.769	0.721
DEGREE OF SATURATION	%	90.0		92.5		94.0	
SATURATED MOISTURE CONTENT	%	30.2		29.5		28.9	
Sketch of failed specimen							
Soil Description		Red, brown & black very clayey SILT/ SAND		Red, borwn & black very clayey very silty SAND		Red, yellowish brown & black gravelly very clayey silty SAND	
COHESION c'	kPa	3.1					
ANGLE OF INTERNAL FRICTION ϕ'	deg.	36.6					
LIQUID LIMIT (%)							
PLASTIC LIMIT (%)							
% By Weight: GRAVEL 2-60	mm	3		3		16	
SAND 0.06-2	mm	41		41		40	
SILT 0.002-0.06	mm	36		32		23	
CLAY <0.002	mm	20		24		21	

Figure 3.2 - Example of Triaxial Test Soil Properties Data

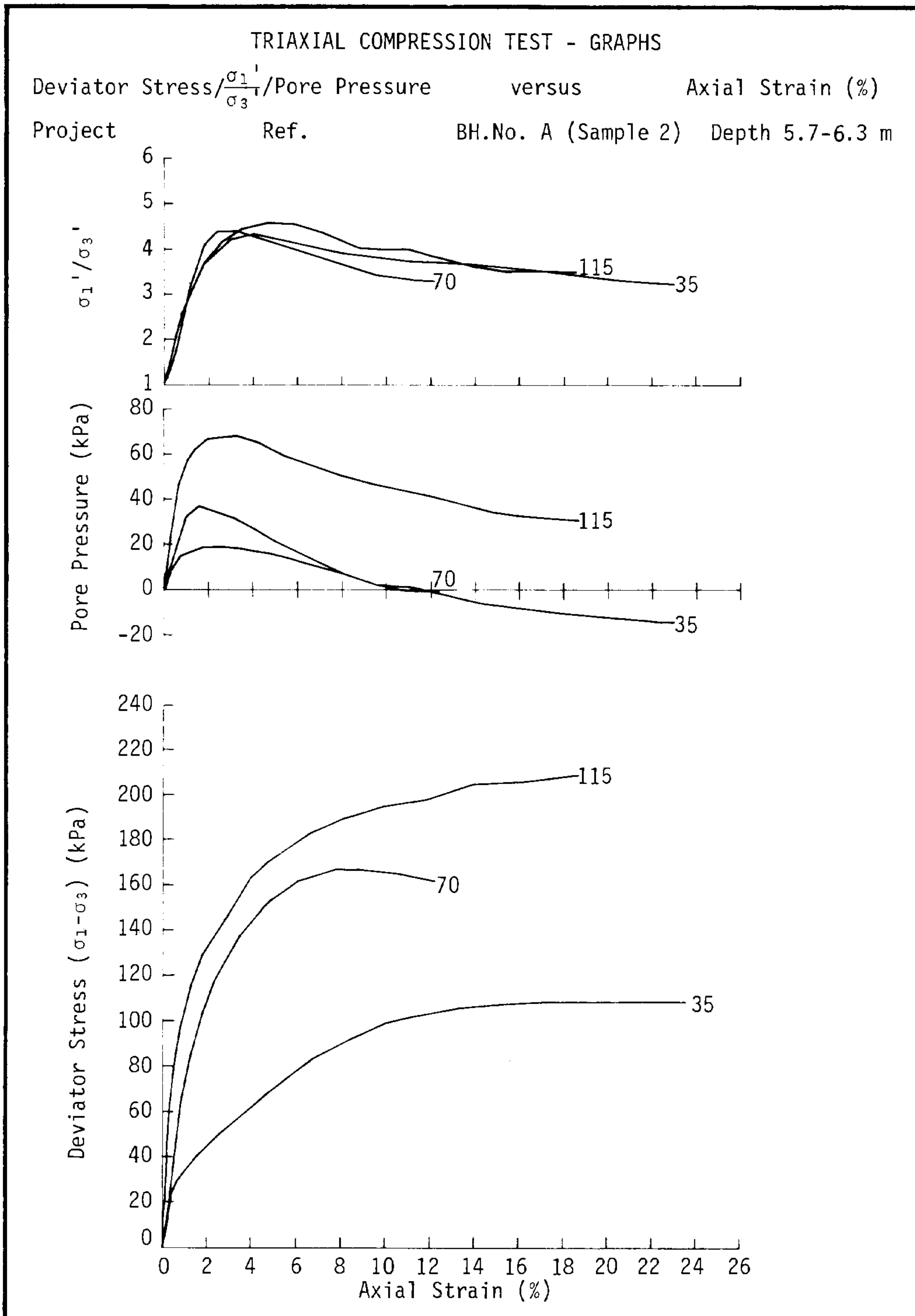


Figure 3.3 - Example of Triaxial Test Graphical Data

TRIAXIAL COMPRESSION TEST - PLOT OF STRESS PATHS

Consolidated undrained with P.P.M. Single stage

Project Ref. B.H.No. A (Sample 2) Depth 5.7-6.3 m

B.H. No	Spec. No	Depth (m)	$(\sigma_1-\sigma_3)_e$ kPa	σ'_{3e} kPa	σ'_{1e} kPa	$\frac{(\sigma_1-\sigma_3)_e}{2}$	$\frac{(\sigma_1'+\sigma_3')_e}{2}$	Strain at failure (%)	γ_d Mg/m³	Nat. m/c %	Sat. m/c %	$B=\frac{\Delta u}{\Delta\sigma_3}$
A	1	5.7	110	49	159	55	104	23.03	1.49	27.1	30.2	1.00
	2	6.0	168	64	232	84	148	8.33	1.50	27.3	29.5	0.99
	3	6.3	210	83	293	105	188	17.35	1.50	27.2	28.9	0.98

Note : ^{e} = end point

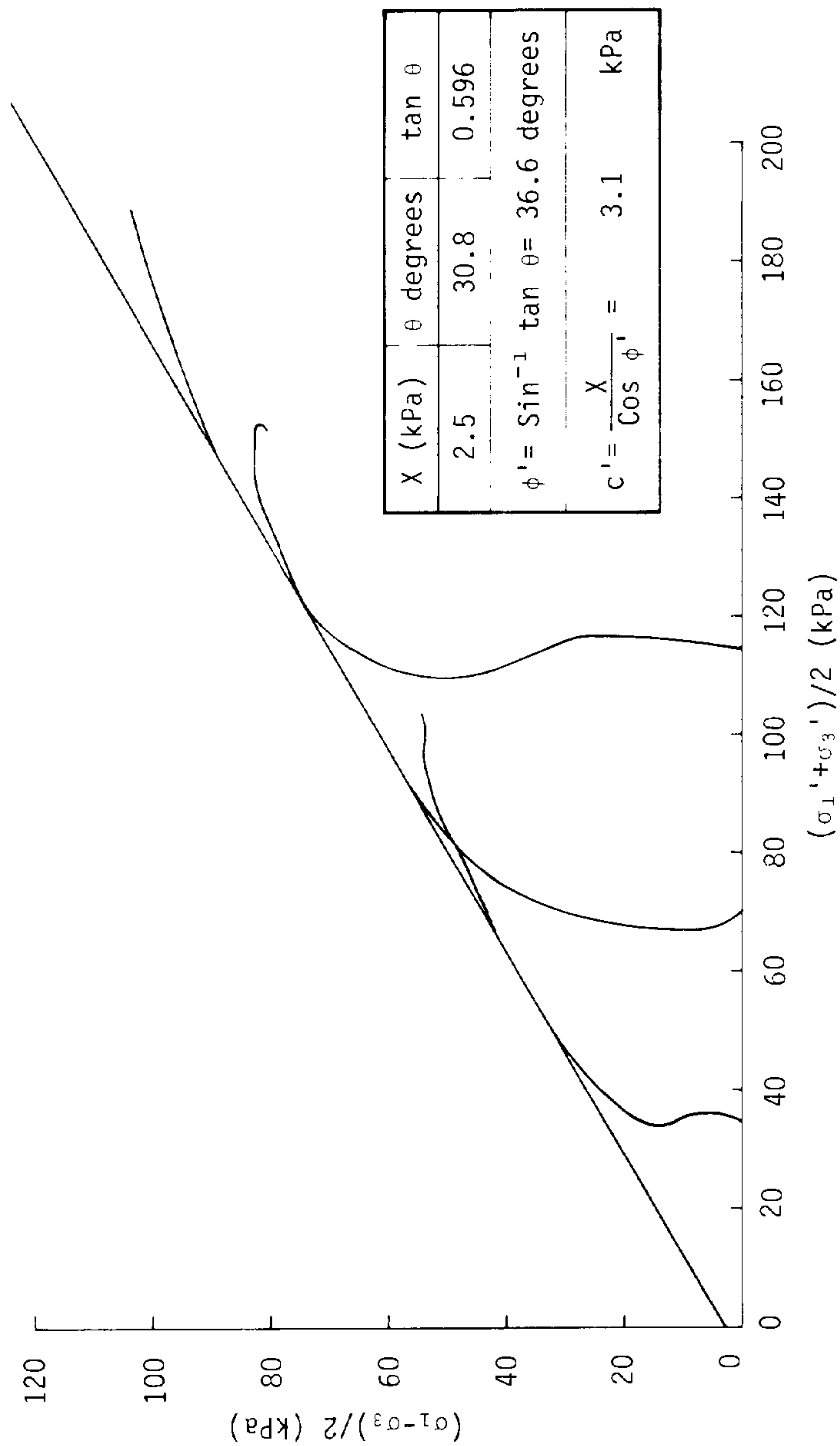


Figure 3.4 - Example of Triaxial Test p'-q/Stress Path Plot

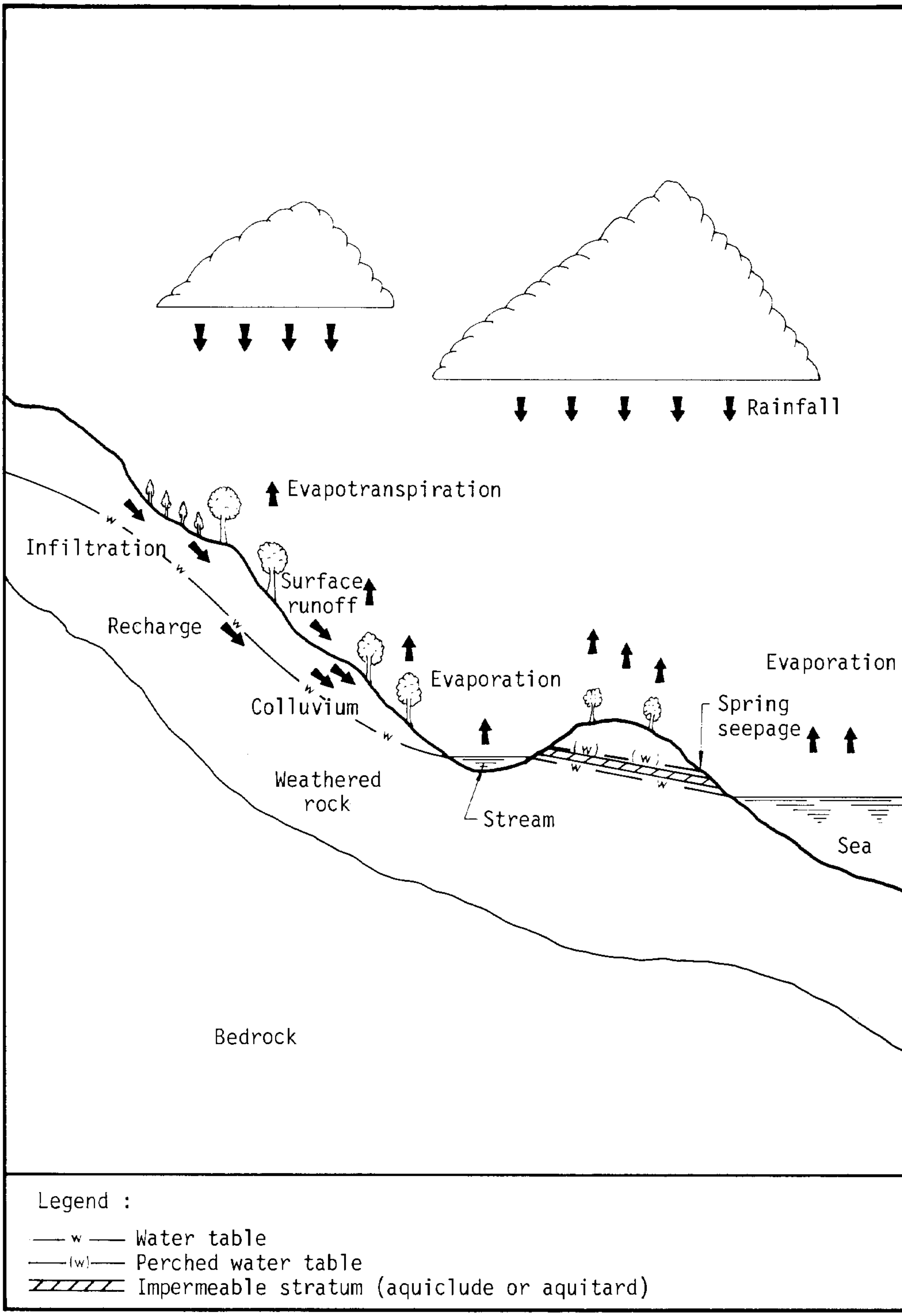


Figure 4.1 - Simplified Representation of the Hydrological Cycle

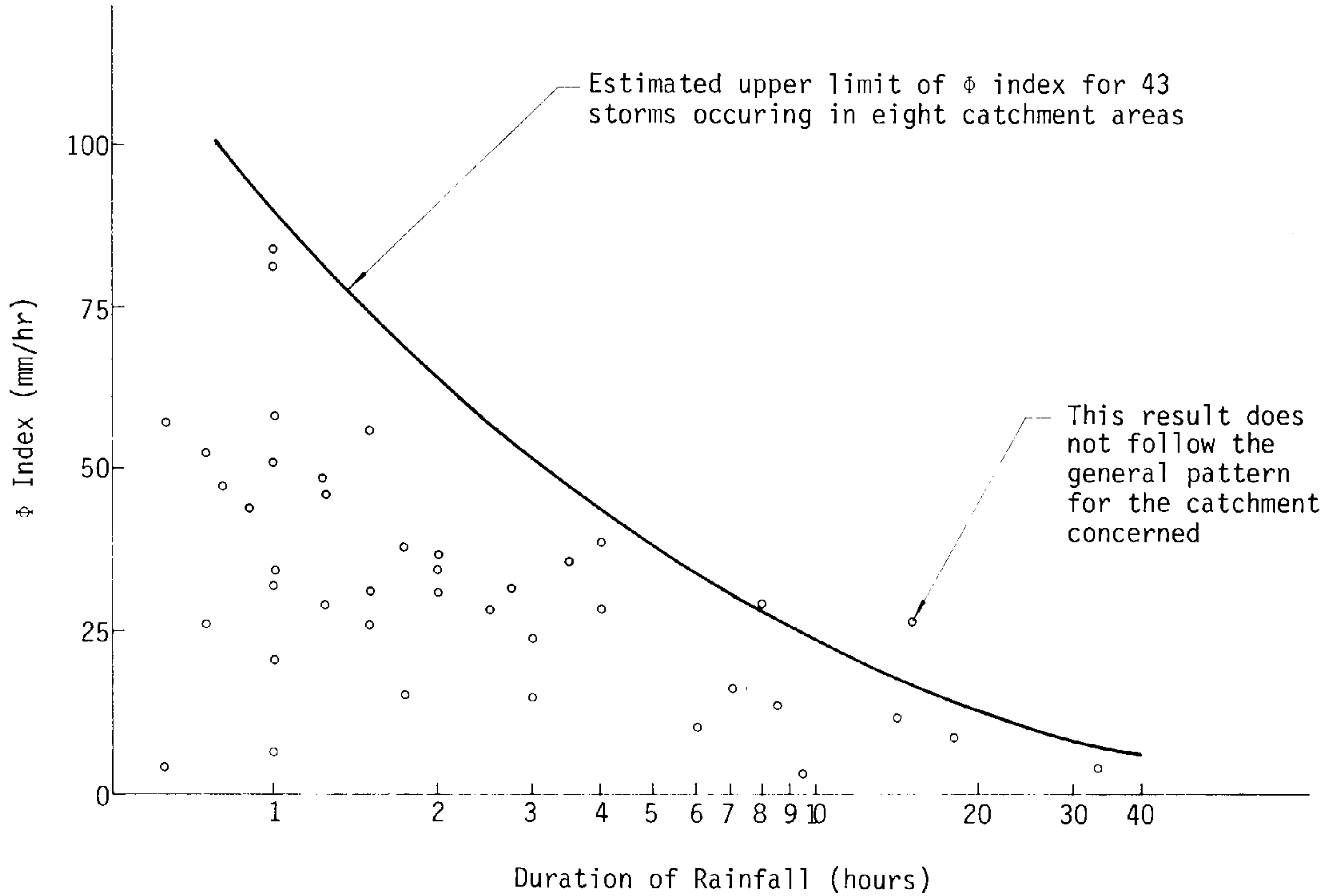
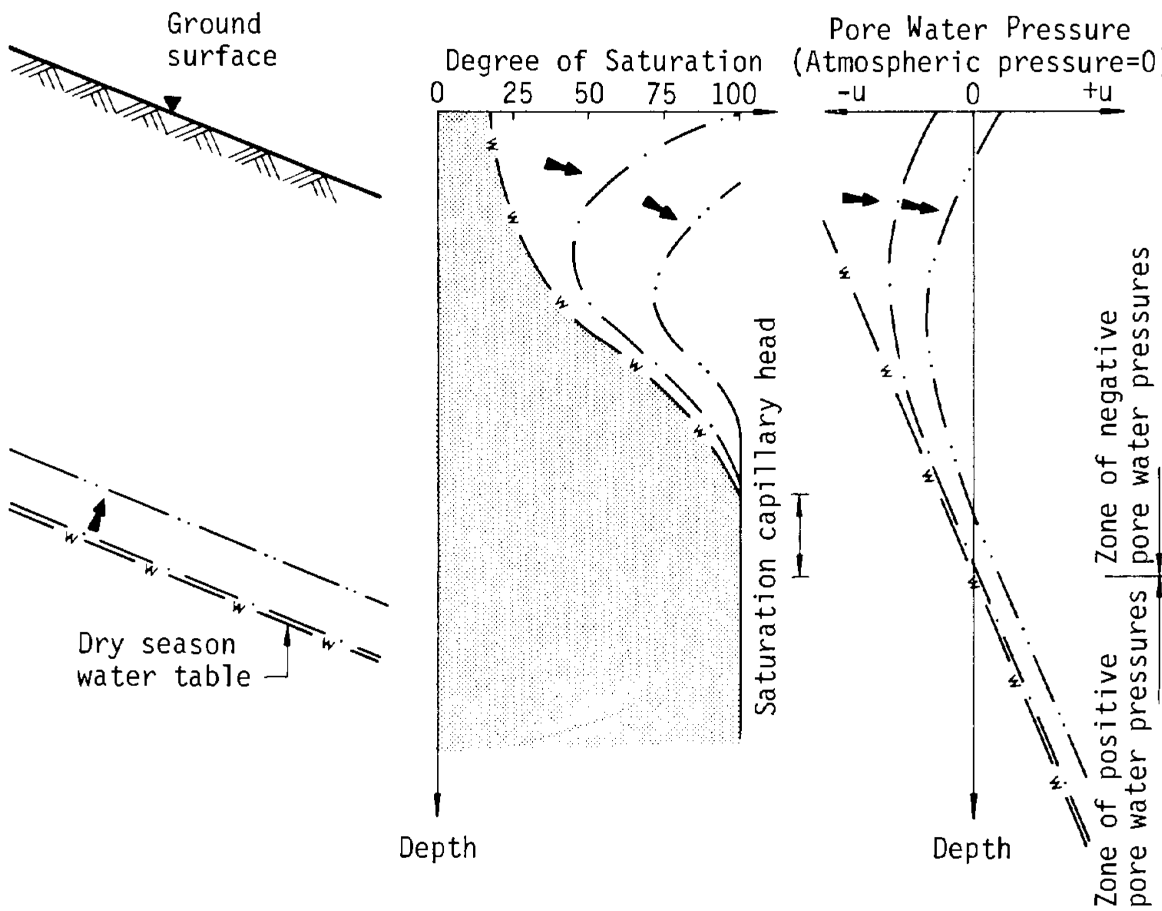


Figure 4.2 - Φ Index for Hong Kong Catchments



Legend :

—— w —— Conditions at time t_0 (Before the rainfall)
—— · —— Conditions at time t_1 (Immediately after the start of rainfall)
—— ·· —— Conditions at time t_2 (After a long period of rainfall)
➡ Rise in water table or increasing saturation and pore water pressure due to rainfall

Figure 4.3 - Typical Changes in Water Table, Degree of Saturation (s) and Pore Water Pressure (u) Due to Rainfall

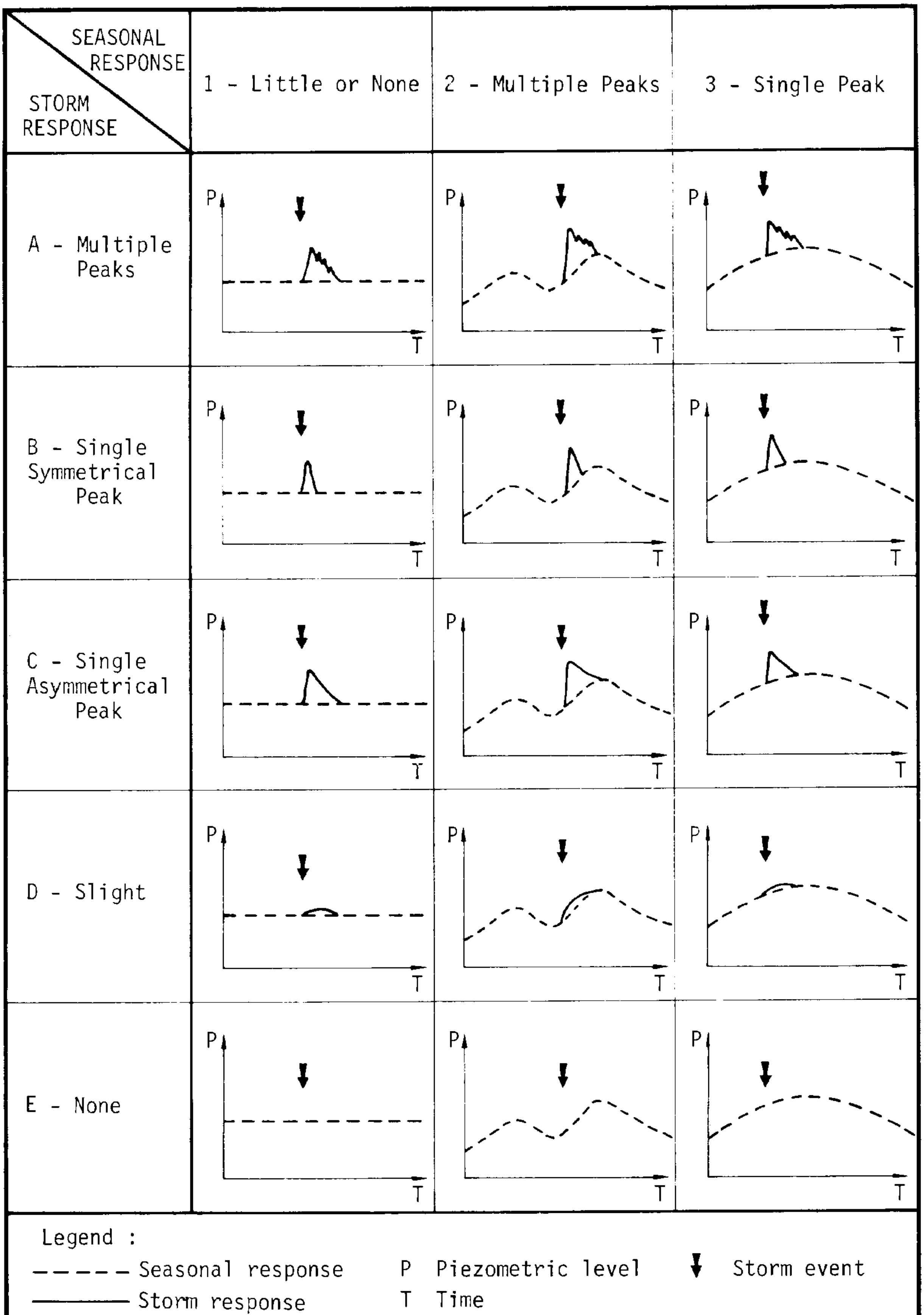
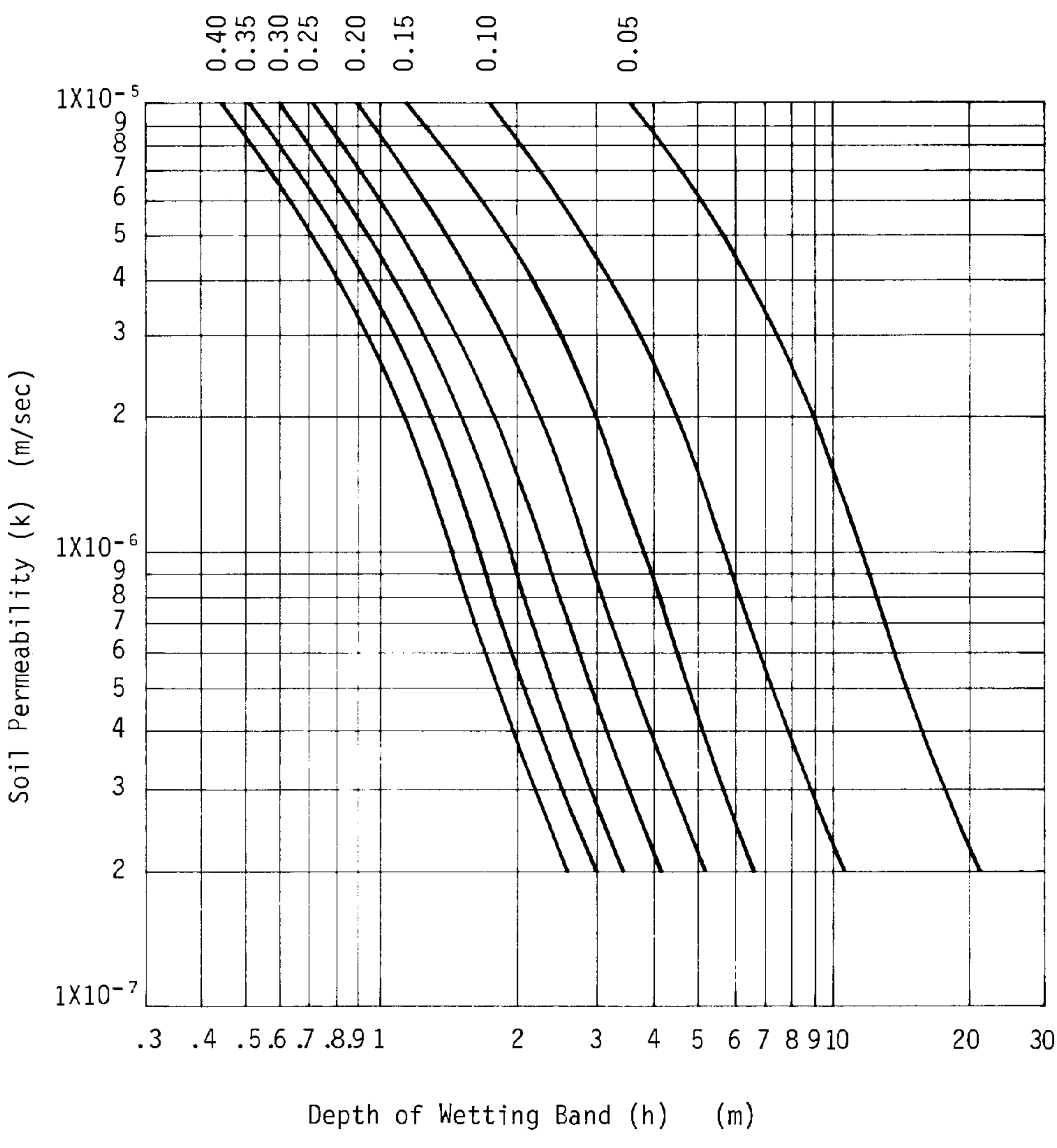


Figure 4.4 - Typical Piezometer Responses



Note : (1) This chart has been prepared assuming runoff is 50% and porosity (n) is 40%.

(2) Curves have been plotted for the various degrees of saturation (S_f - S_o) shown.

Figure 4.5 - Effect of Permeability and Degree of Saturation on Wetting Band Thickness for a Ten-year Return Period Rainfall Event

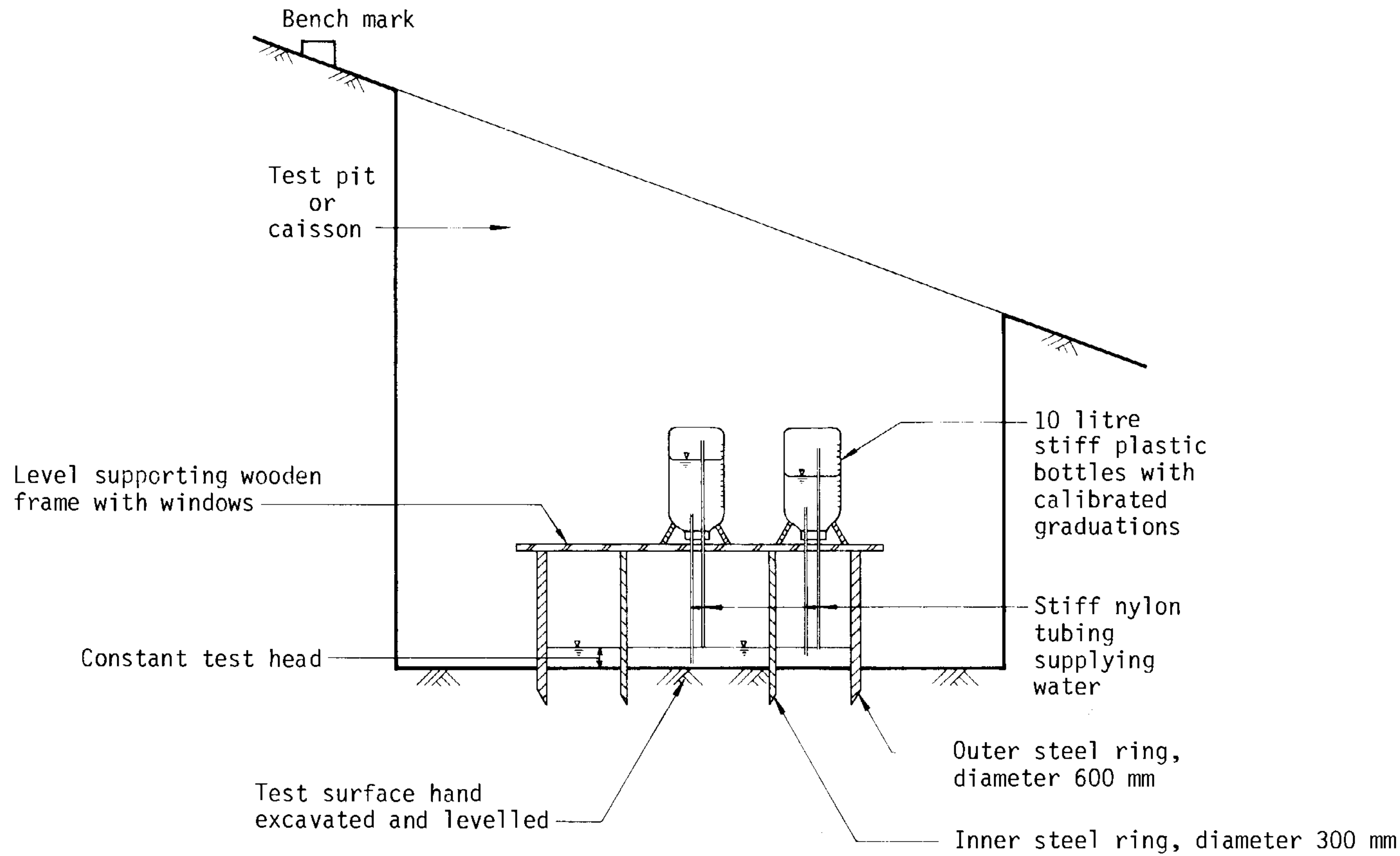


Figure 4.6 - Double-ring, Constant-head Field Infiltration Test Apparatus

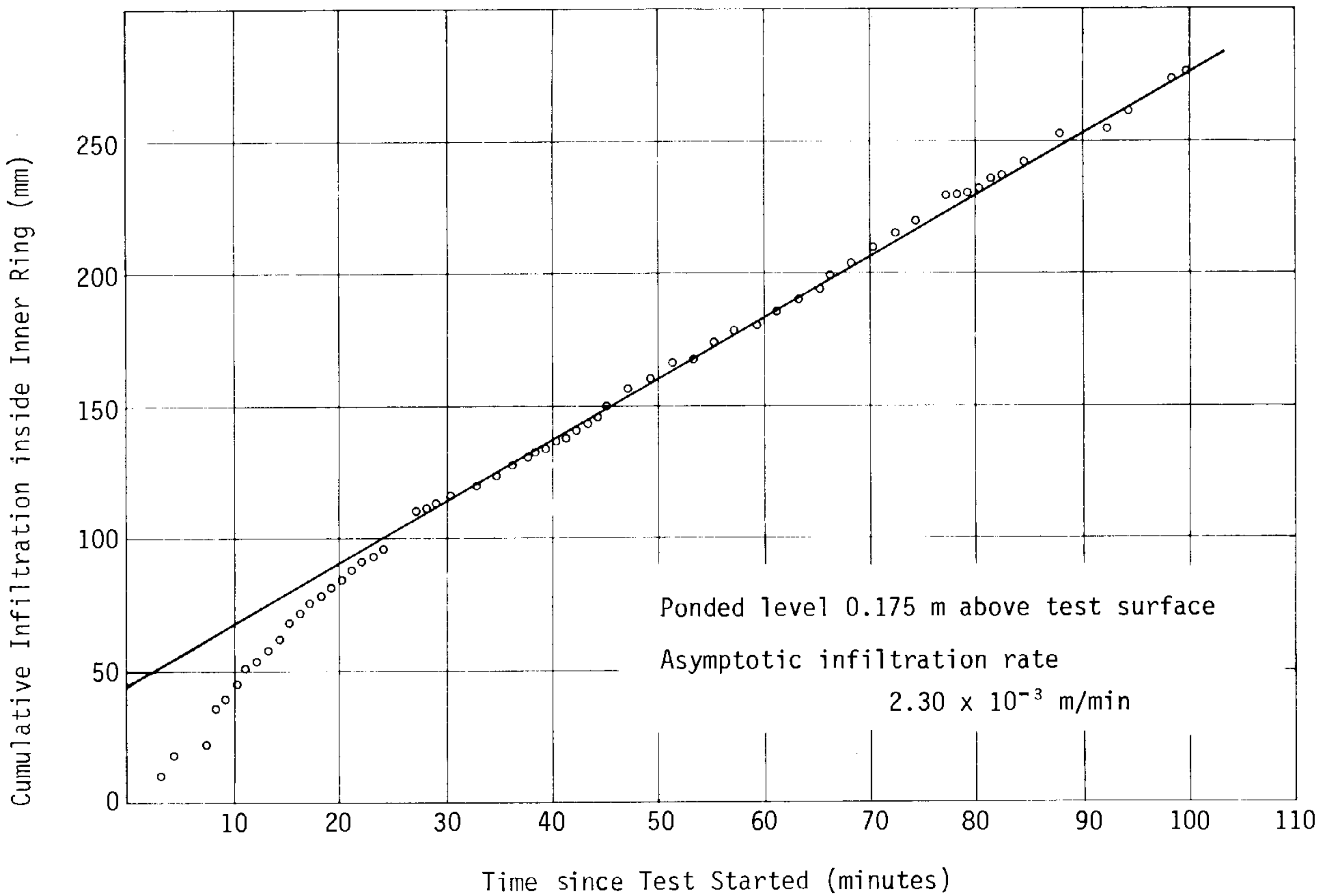


Figure 4.7 - Typical Results from Field Infiltration Test

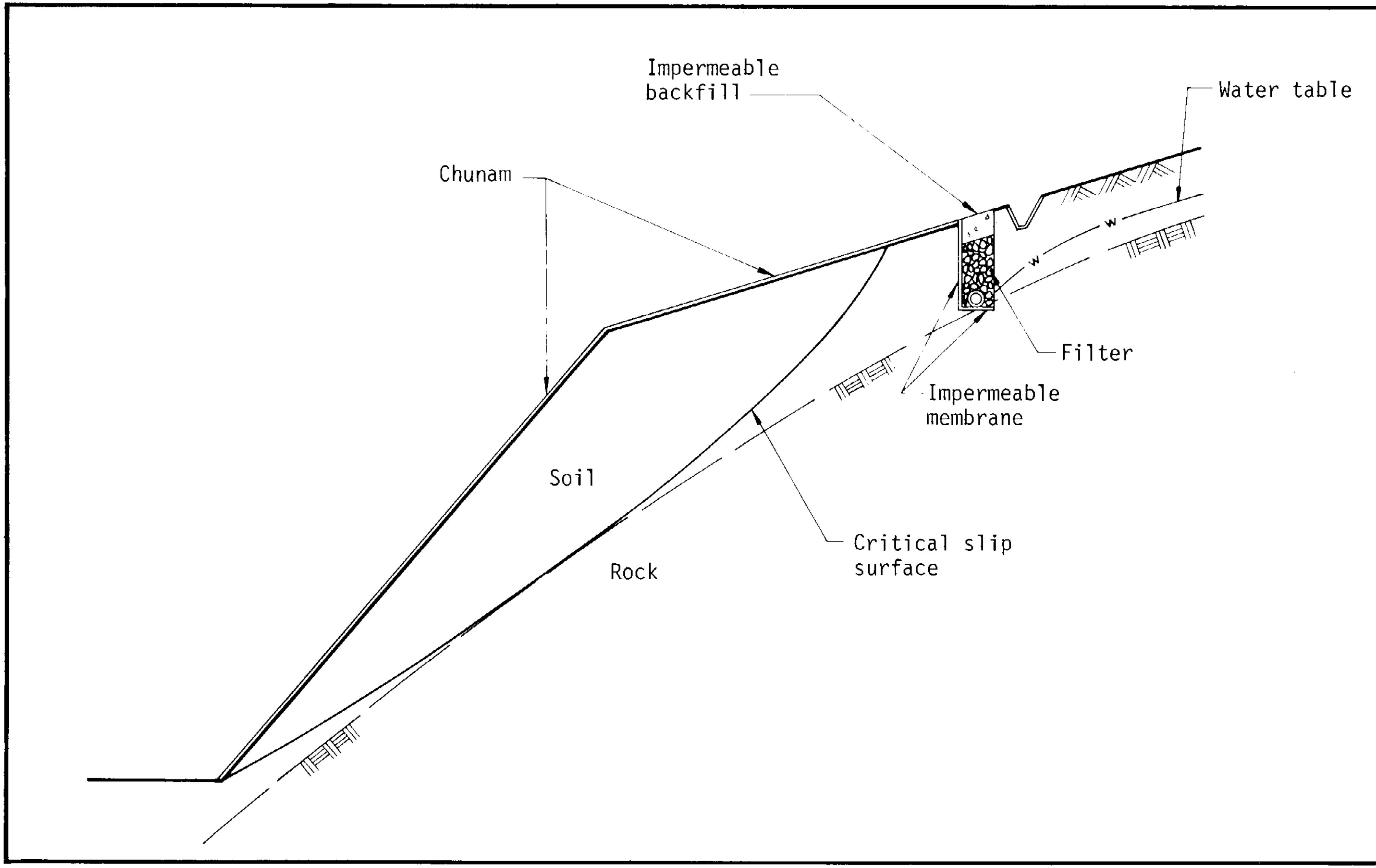
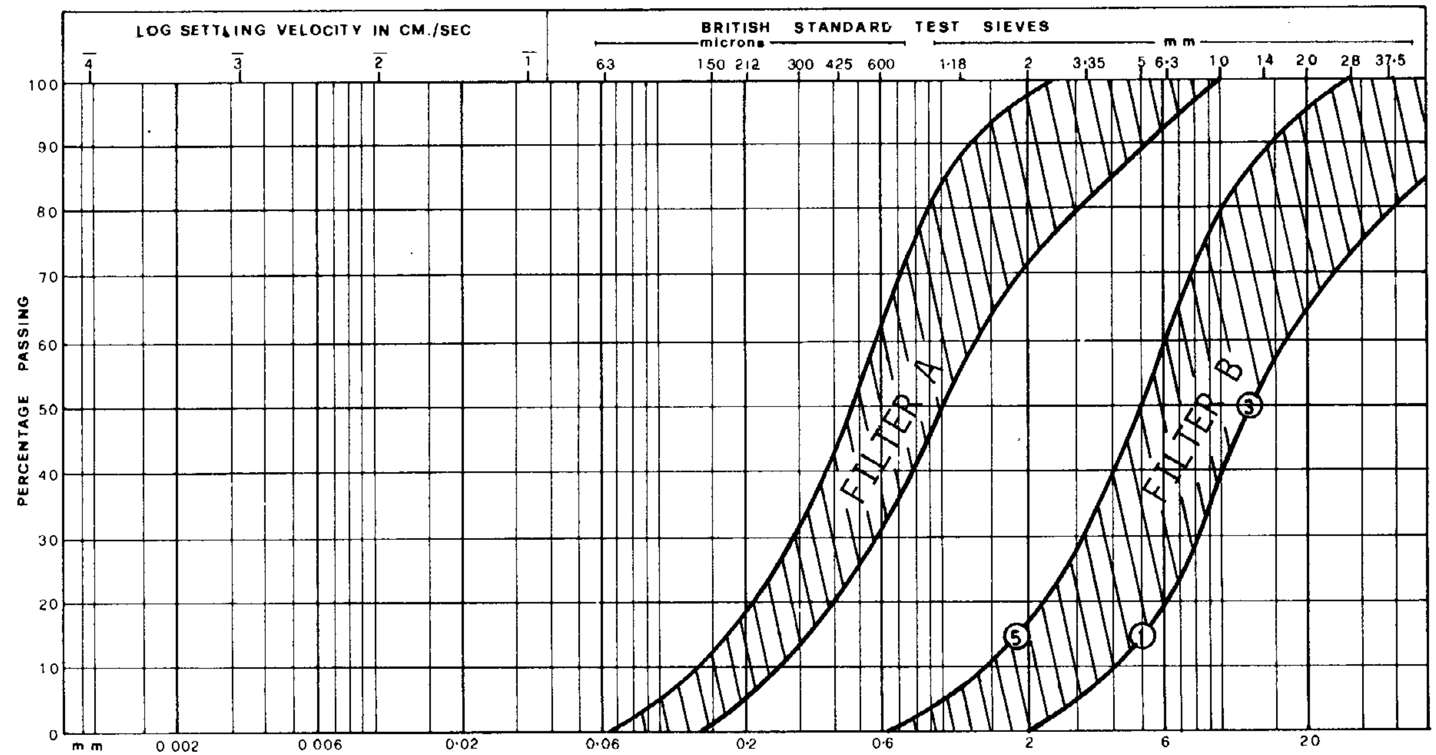


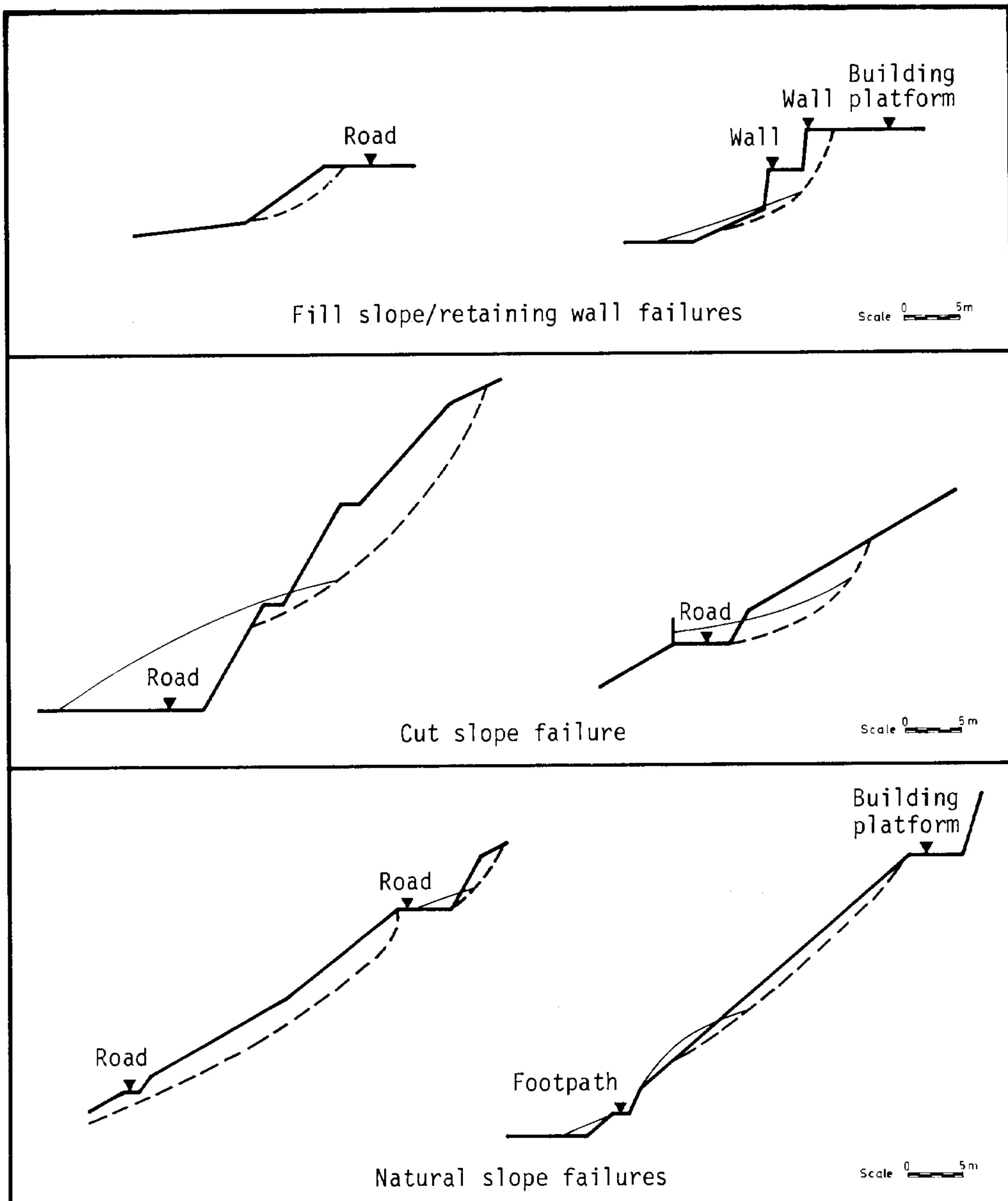
Figure 4.8 - Cut-off Drain



Note : (1) Filter A - This envelope describes the United States Corps of Engineers concrete sand which will act as a filter for all silts and finer soils. The grading of BS 882 Zone 2 natural sand is very similar to the United States Corps of Engineers concrete sand.

(2) Filter B - This envelope describes the drainage material associated with the sand filter (A) and has been derived using the filter rules of Table 4.1. The number of each rule used is shown on the diagram at the appropriate point on each grading curve.

Figure 4.9 - Grading of a Sand Filter Suitable for All Silts and Finer Soils



Legend :

Pre failure geometry
Post failure geometry
Extent of debris

Note : The sketch sections shown in this Figure are of failures that occurred in 1982.

Figure 5.1 - Typical Failure Profiles in Weathered Rocks and Soils in Hong Kong

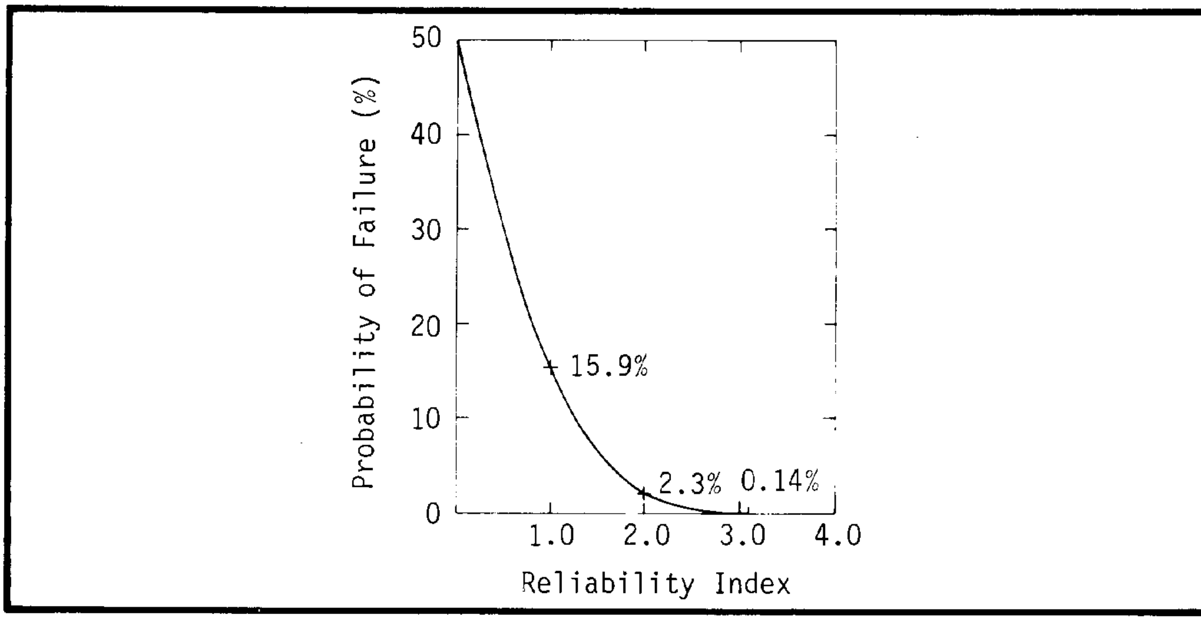


Figure 5.2 - Probability of Failure Versus Reliability Index

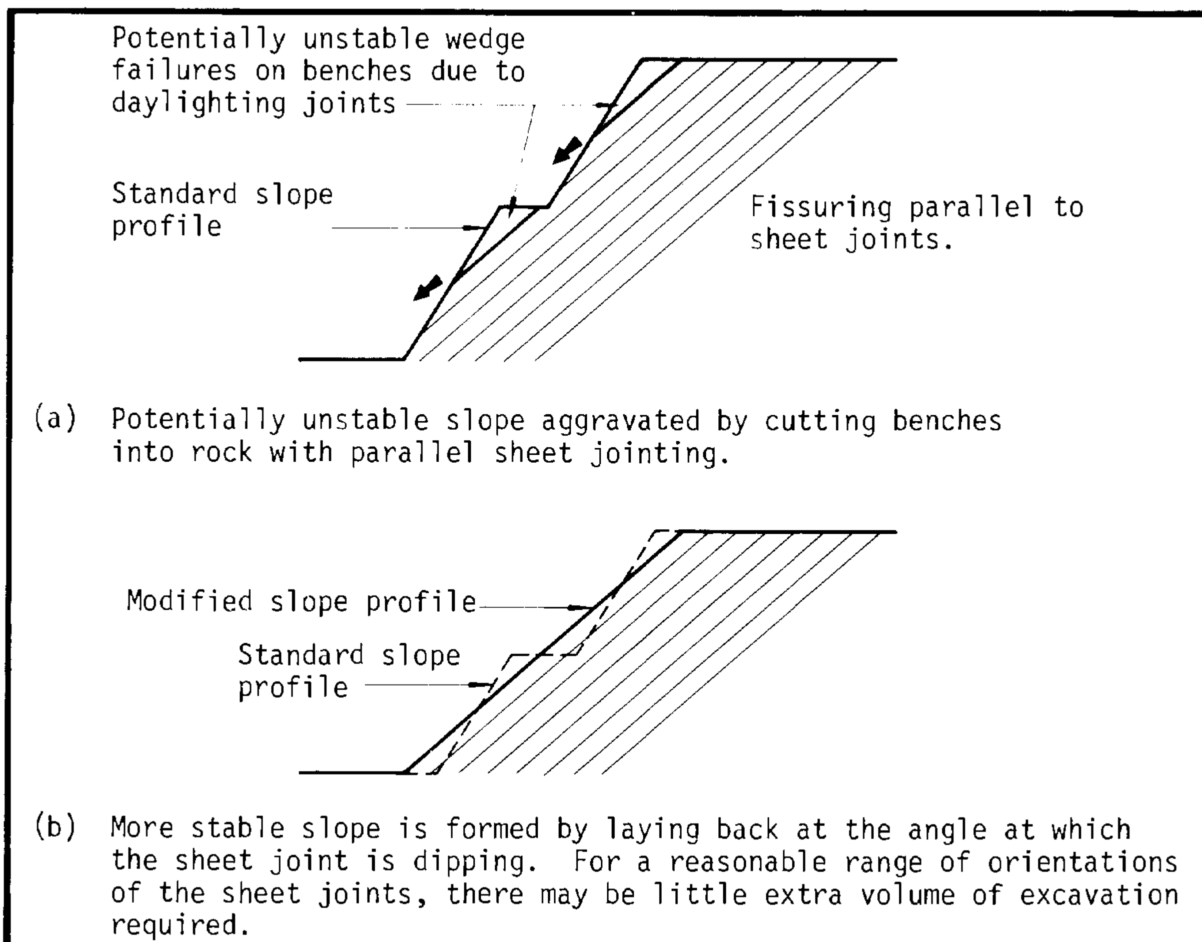


Figure 5.3 - Effect of Benches in Adversely Jointed Rock

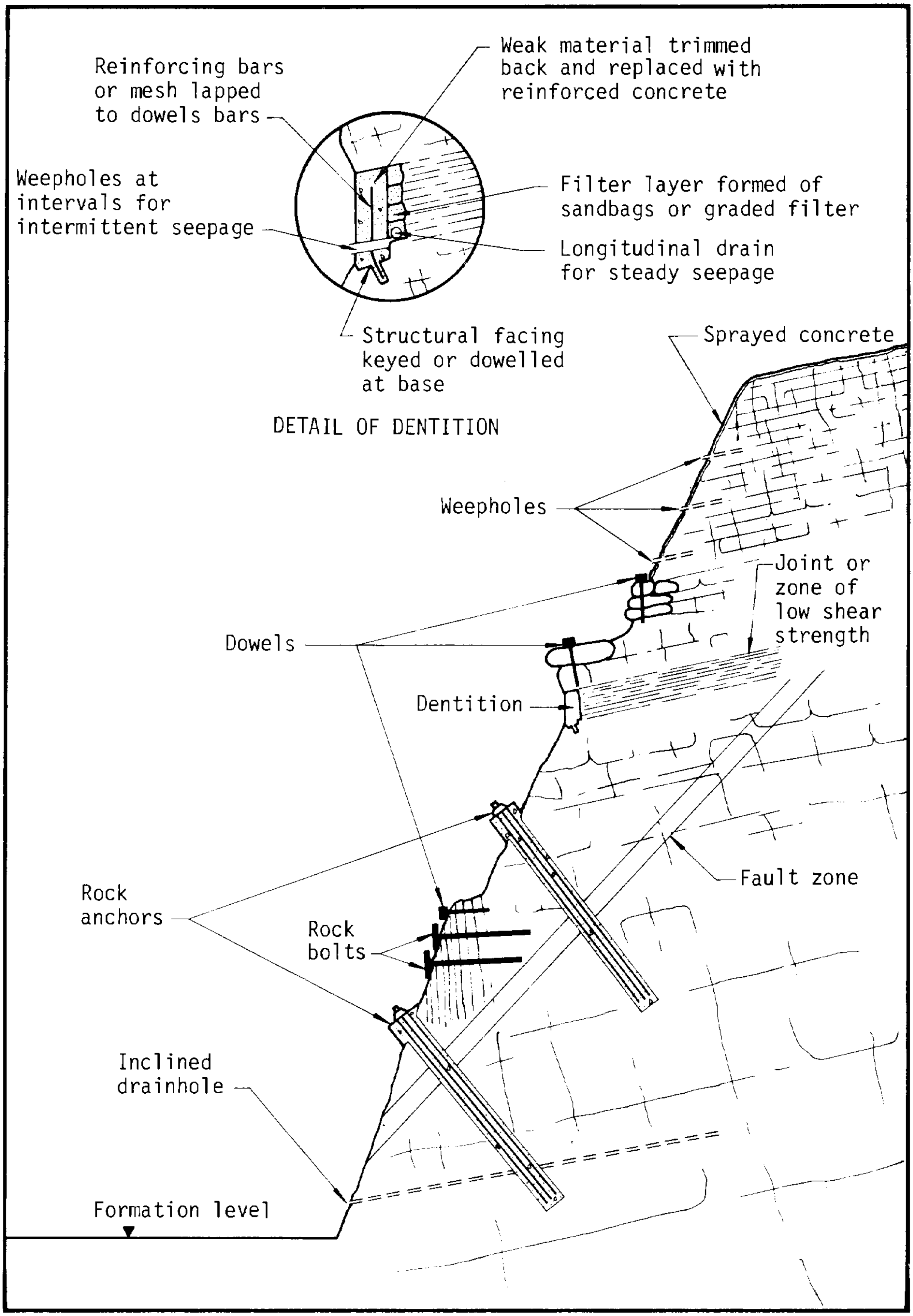


Figure 5.4 - Various Methods of Stabilising Rock Slopes

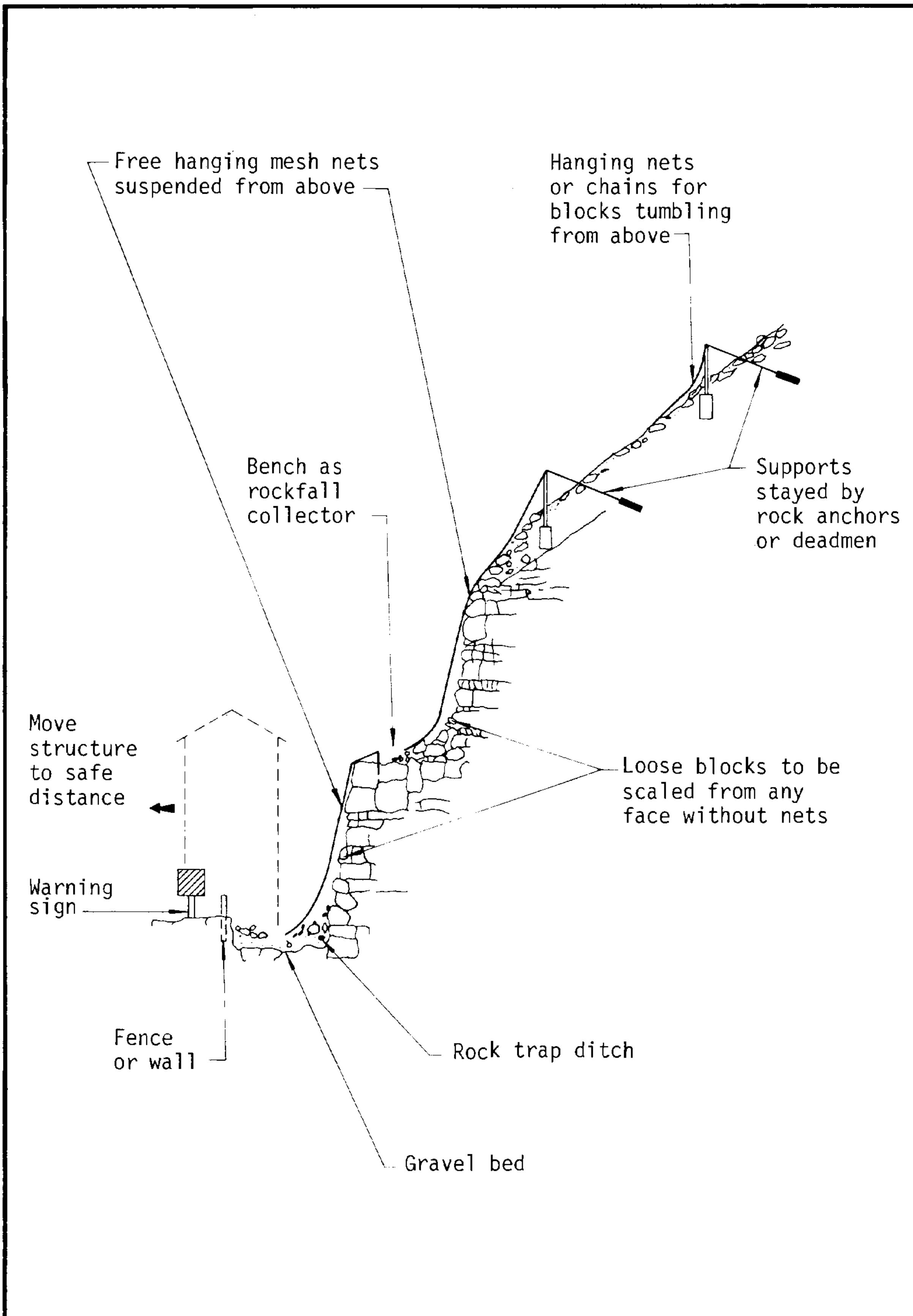


Figure 5.5 - Rockfall Control Measures

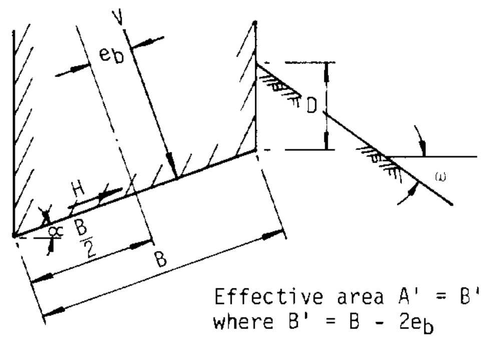
$$q_{ult} = cN_c\, S_c\, i_c\, t_c\, g_c + \tfrac{1}{2}\gamma B N_\gamma\, S_\gamma\, i_\gamma\, t_\gamma\, g_\gamma + qN_q\, S_q\, i_q\, t_q\, q_q$$

SHAPE FACTORS

$$S_c = 1 + \frac{B}{L} \cdot \frac{N_q}{N_c}$$

$$S_\gamma = 1 - 0.4\,\frac{B}{L}$$

$$S_q = 1 + \frac{B}{L}\tan\phi'$$



Effective area $A' = B'\,L'$
where $B' = B - 2e_b$
$L' = L - 2e_1$

INCLINATION FACTORS

$$i_c = i_q - \frac{1 - i_q}{N_c \tan\phi'}$$

$$i_\gamma = \left[1 - \frac{H}{V + BLc\cot\phi'}\right]^{m+1}$$

$$i_q = i_\gamma^{\frac{m}{m+1}}$$

where : $m = \dfrac{2 + \frac{B}{L}}{1 + \frac{B}{L}}$ provided the inclination of load is in the direction of B.

: $H_{max} = V\tan\phi' + Ac$

TILT FACTORS

$$t_c = t_q - \frac{1 - t_q}{N_c \tan\phi'}$$

$$t_\gamma = t_q = \left[1 - \alpha\tan\phi'\right]^2$$

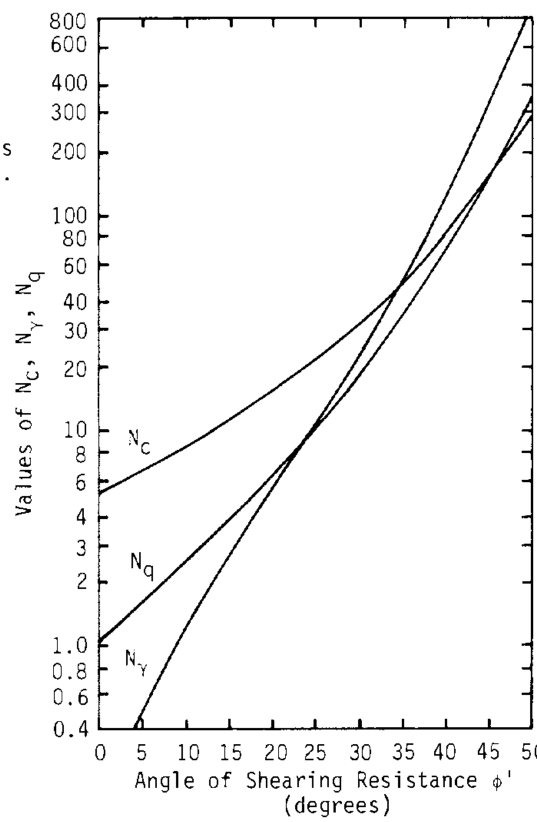
where : α is in radians

GROUND SLOPE FACTORS

$$g_c = g_q - \frac{1 - g_q}{N_c \tan\phi'}$$

$$g_\gamma = g_q = \left[1 - \tan\omega\right]^2$$

where : $\alpha < 45°$, $\omega < 45°$ and $\omega < \phi$



BEARING CAPACITY FACTORS

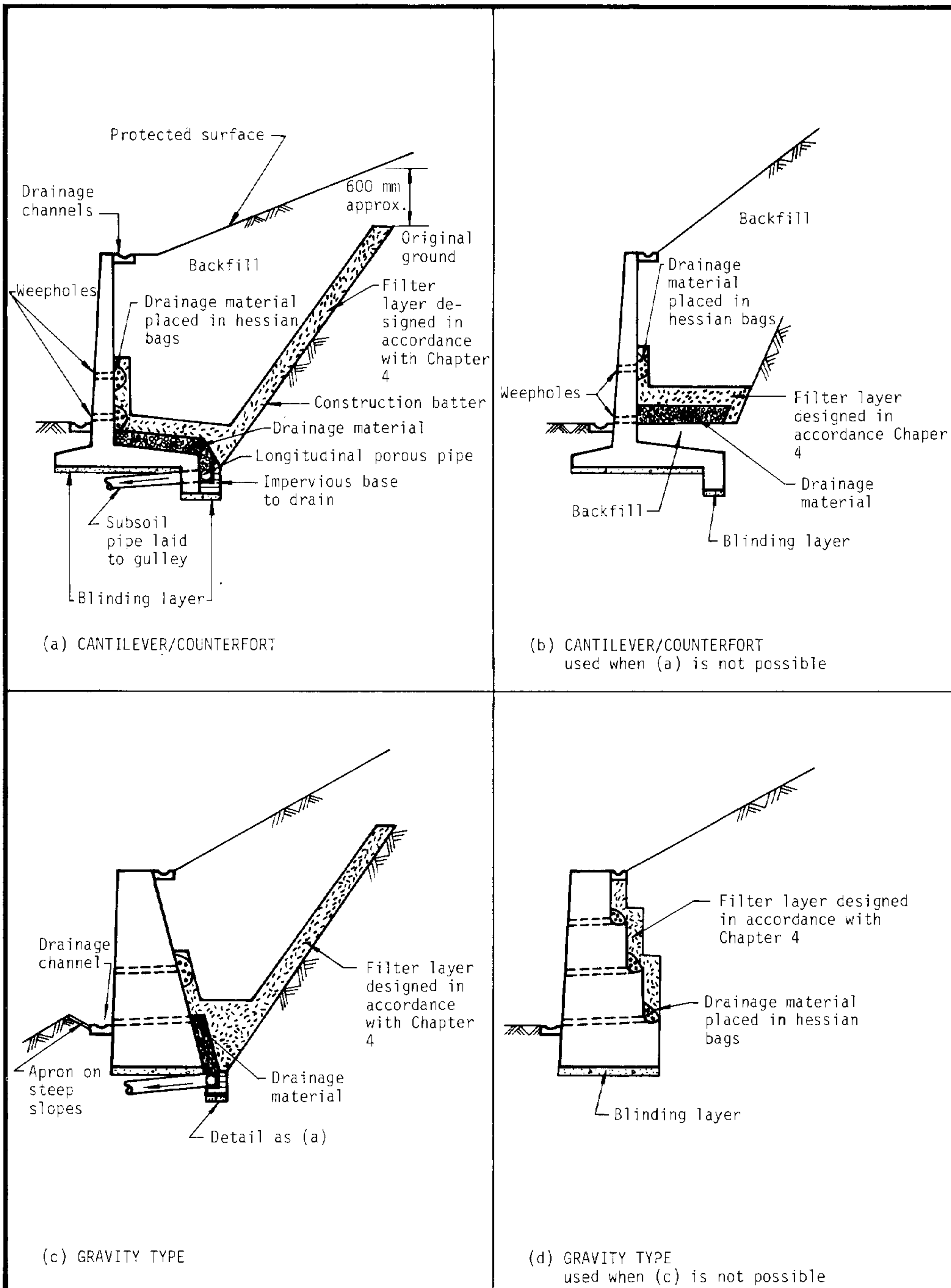
Note : (1) Data applies to shallow foundations only $D \leq B$.

(2) For $\omega \geq \dfrac{\phi'}{2}$ a check should be made for overall slope stability.

(3) For the effects of nonhomogeneous soil and soil compressibility and scale effects reference should be to Vesic (1975).

(4) Where the foundation is set back from the crest of the slope, refer to GCO (1982b).

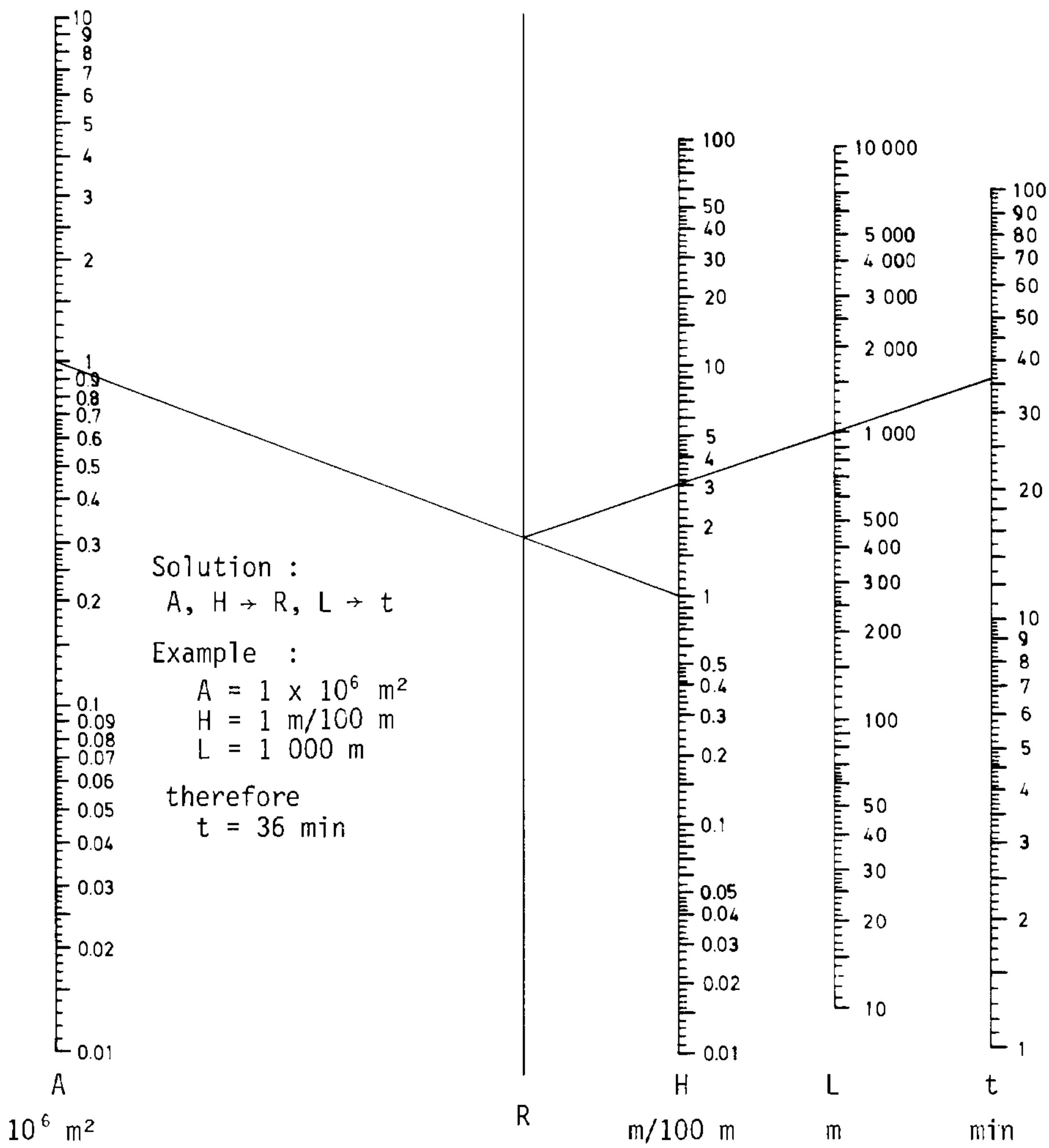
Figure 6.1 - Bearing Capacity Data



Note : (1) For ease of construction, where filter layers are constructed at a steep incline, filter material may be placed in hessian bags.

(2) Water pressure should be considered in design for types (b) and (d).

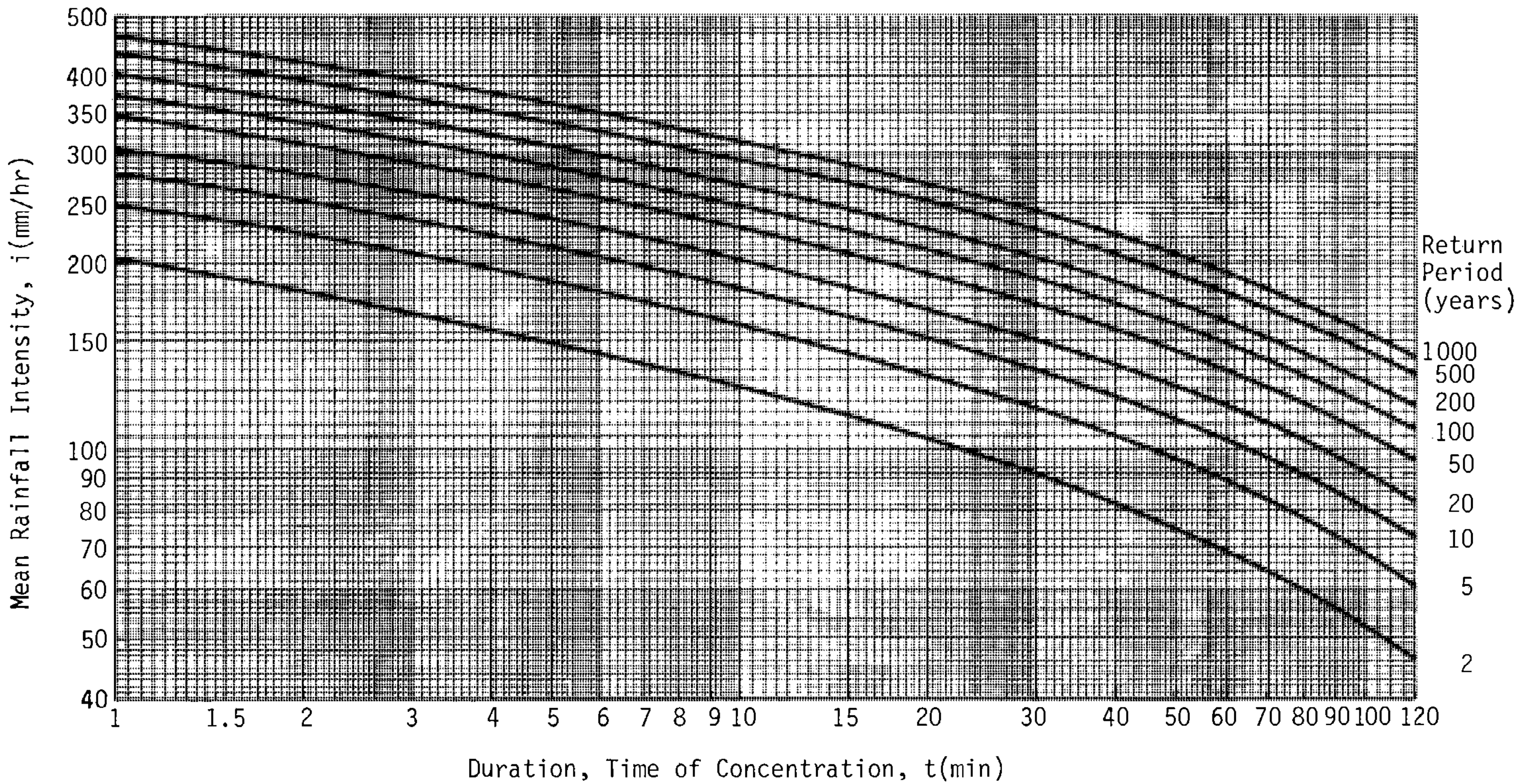
Figure 7.1 - Drainage Details for Retaining Walls



Legend :

t	Time of concentration (min)
H	Slope (m/100 m)
A	Catchment area (m^2)
L	Distance from catchment boundary to the design section (m), for which t is a maximum

Figure 8.1 - Nomogram for the Rapid Solution of the Bransby-Williams Equation



Note : The intensity-duration rainfall curves are from Peterson & Kwong (1981) - A design rainstorm profile for Hong Kong, Technical Note No. 58, Royal Observatory, Hong Kong. Data from tilting siphon records 1947-1980 (Royal Observatory) and instantaneous rate-of-rainfall records 1952-1980 (King's Park).

Figure 8.2 - Curves Showing Duration and Intensity of Rainfall in Hong Kong for Various Return Periods

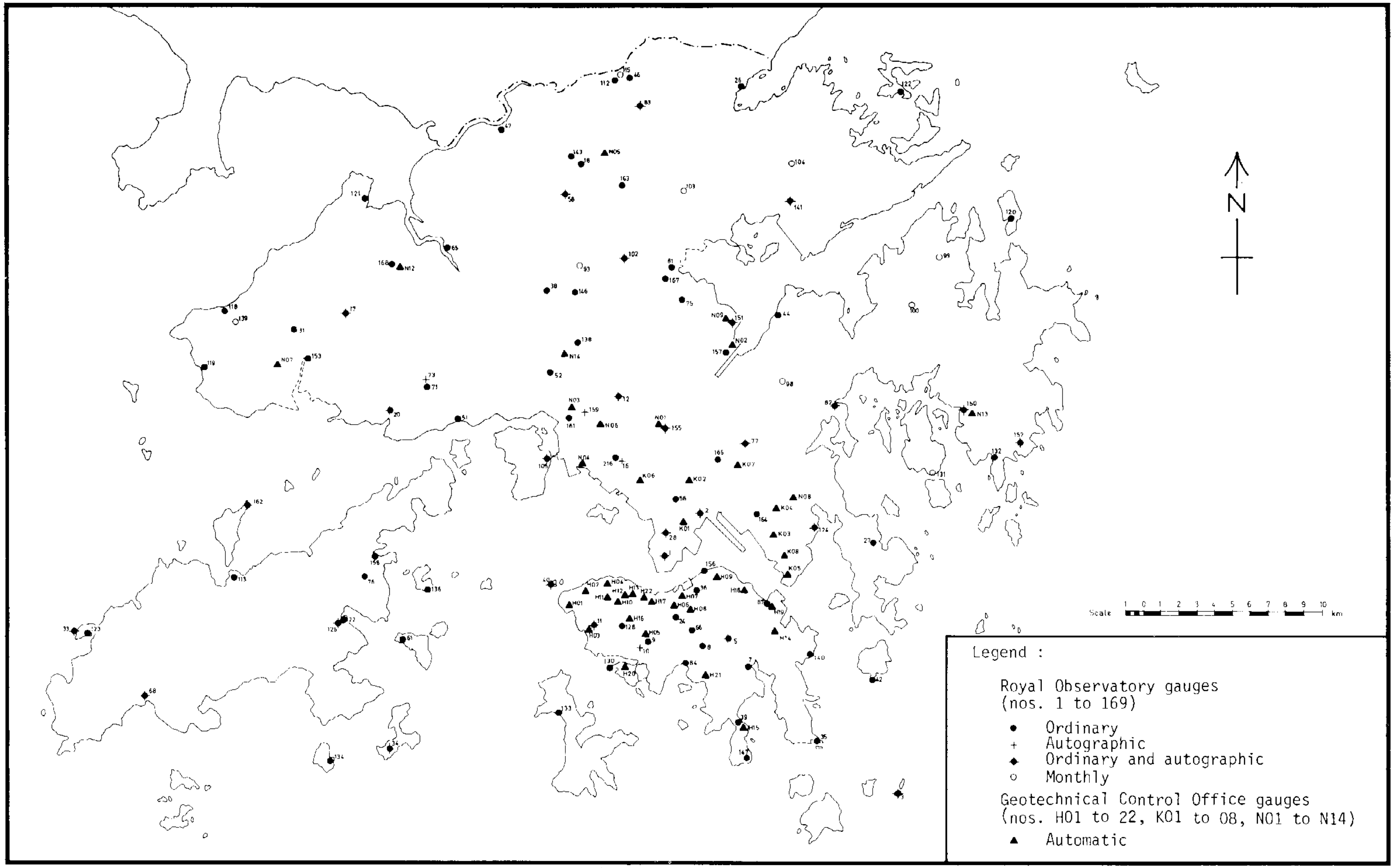
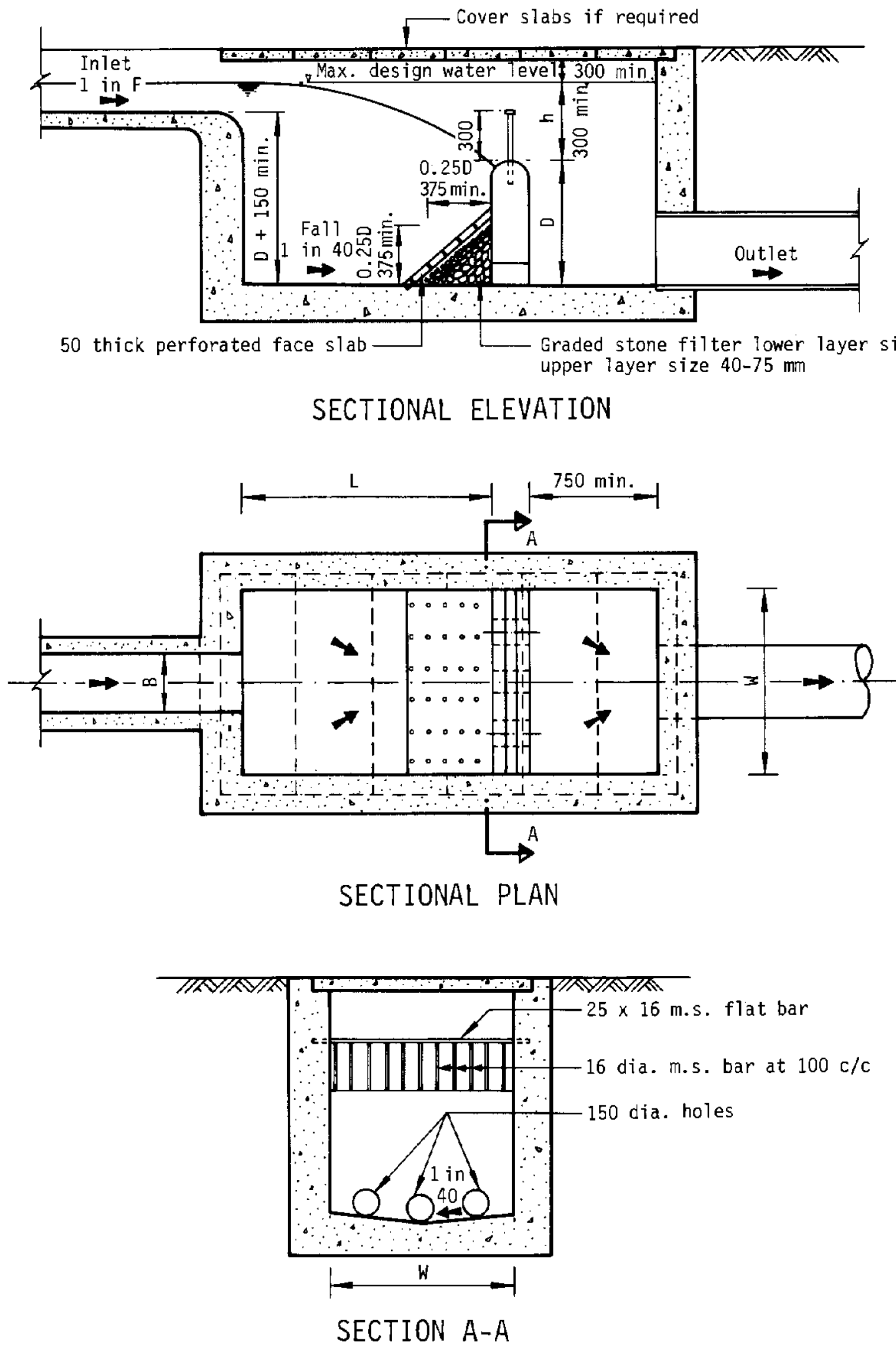


Figure 8.3 - Distribution of Raingauges in Hong Kong



Note :

(1) All dimensions in millimetres.

(2) Normally for drains of 900 mm dia. and below. For bigger drains and steep terrain, sand trap should be specially designed.

(3) Size

Depth : D $\not>$ 750

Width : W $\geq$ 3B

Length : $L = 4.8D^{0.67} h^{0.5} F^{-0.5} \geq 4B$

(4) Graded stone filter shall be crusher run granite aggregate.

(5) Capacity DWL to be according to size and nature of catchment, providing detention time not less than 5 minutes for max. design flow of inlet.

Figure 8.4 - Typical Sand Trap Arrangement

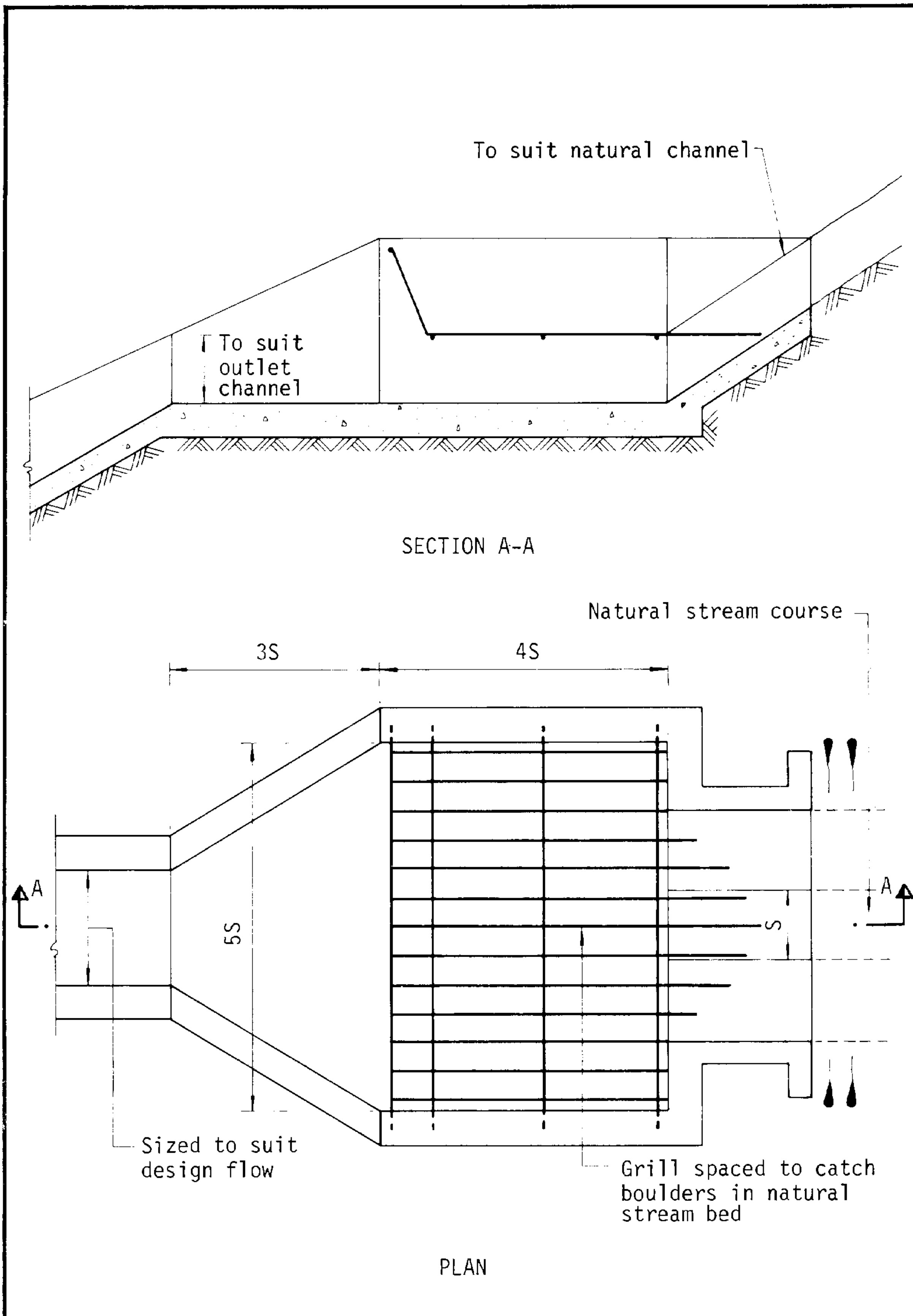


Figure 8.5 - Typical Rock Trap Detail

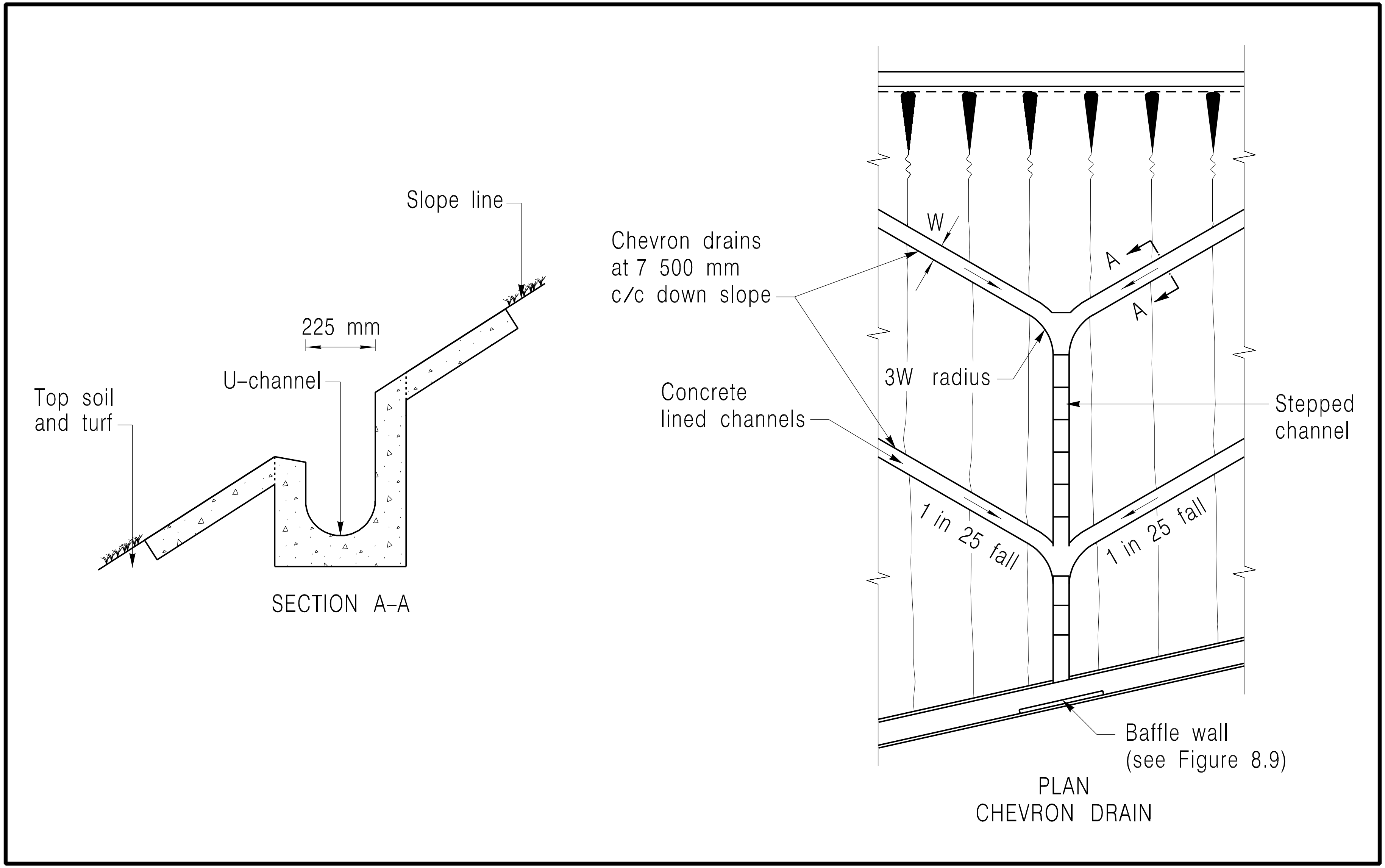
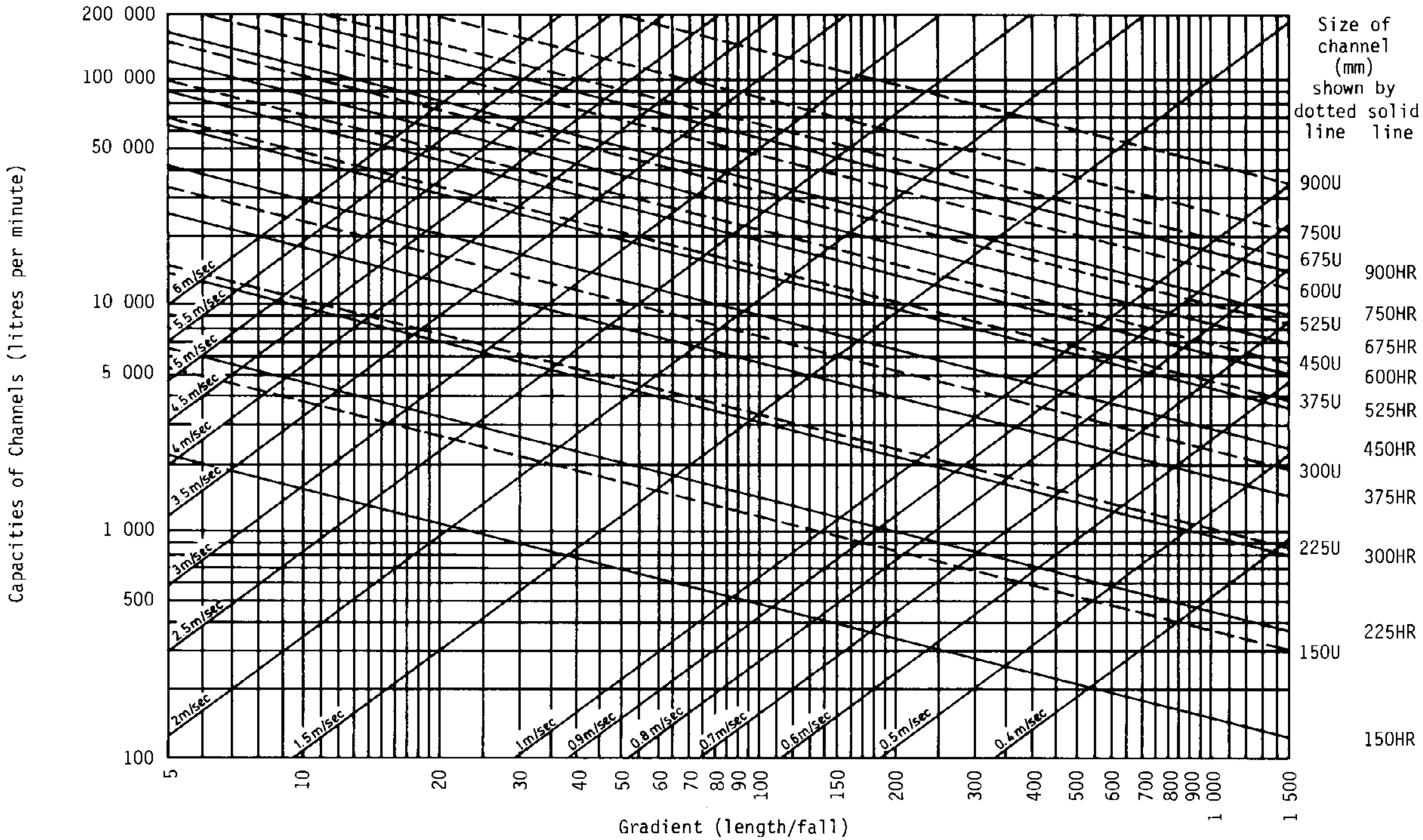
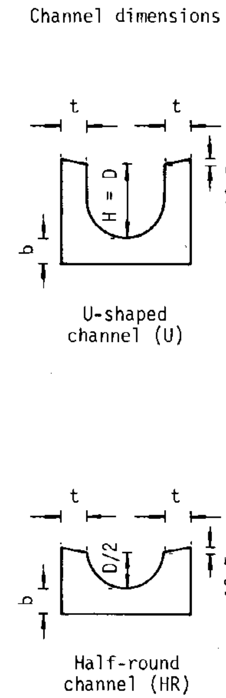


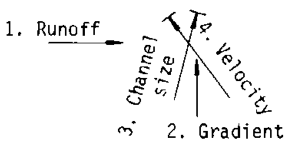
Figure 8.6 – Plan Showing Junction of Tributary Channels with Main Channels





DESIGN METHOO USING CHART

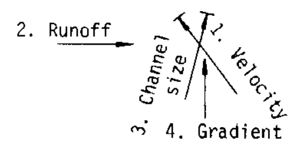
(a) Normal channel Solution



Example :

1. Enter Runoff = 4 000 litre/min.
2. Enter Gradient = 1 in 40
3. Read channel required = 225 U or 300HR
4. Read velocity = 2.2 m/sec. (<4 m/sec. ∴OK)

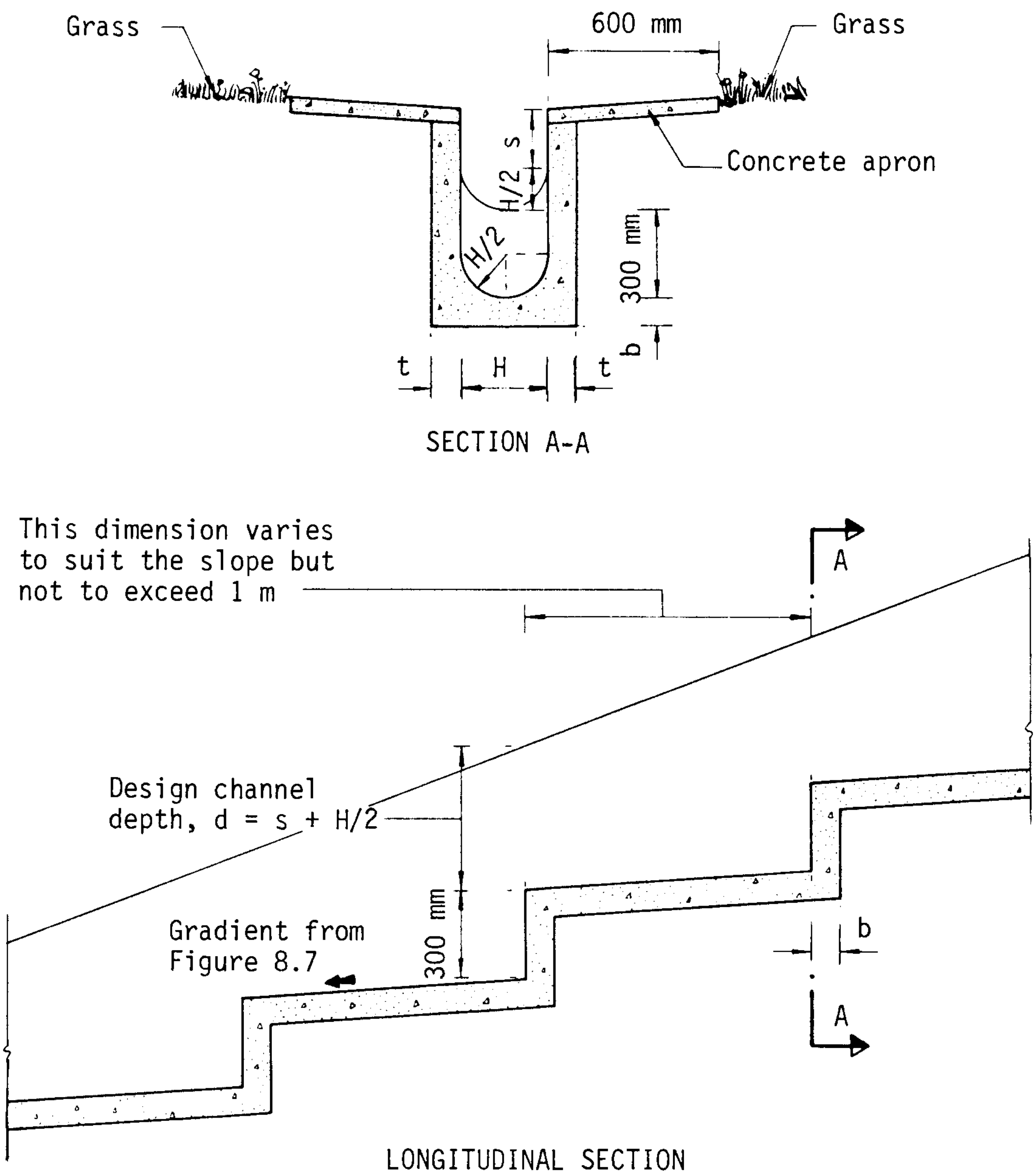
(b) Stepped channel Solution



Example :

1. Enter Velocity = 5 m/sec.
2. Enter Runoff = 20 000 litre/min.
3. Read required channel size = 300U
4. Read required gradient = 1 in 14

Figure 8.7 - Chart for the Rapid Design of Channels



Dimensions of Stepped Channel			
Nominal size of channel H(mm)	Thickness t(mm)	Thickness b(mm)	Splash allowance s(mm)
225 to 300	100	100	200
375 to 675	100	150	350
750 to 900	125	200	400

Figure 8.8 - Typical Details of Stepped Channel

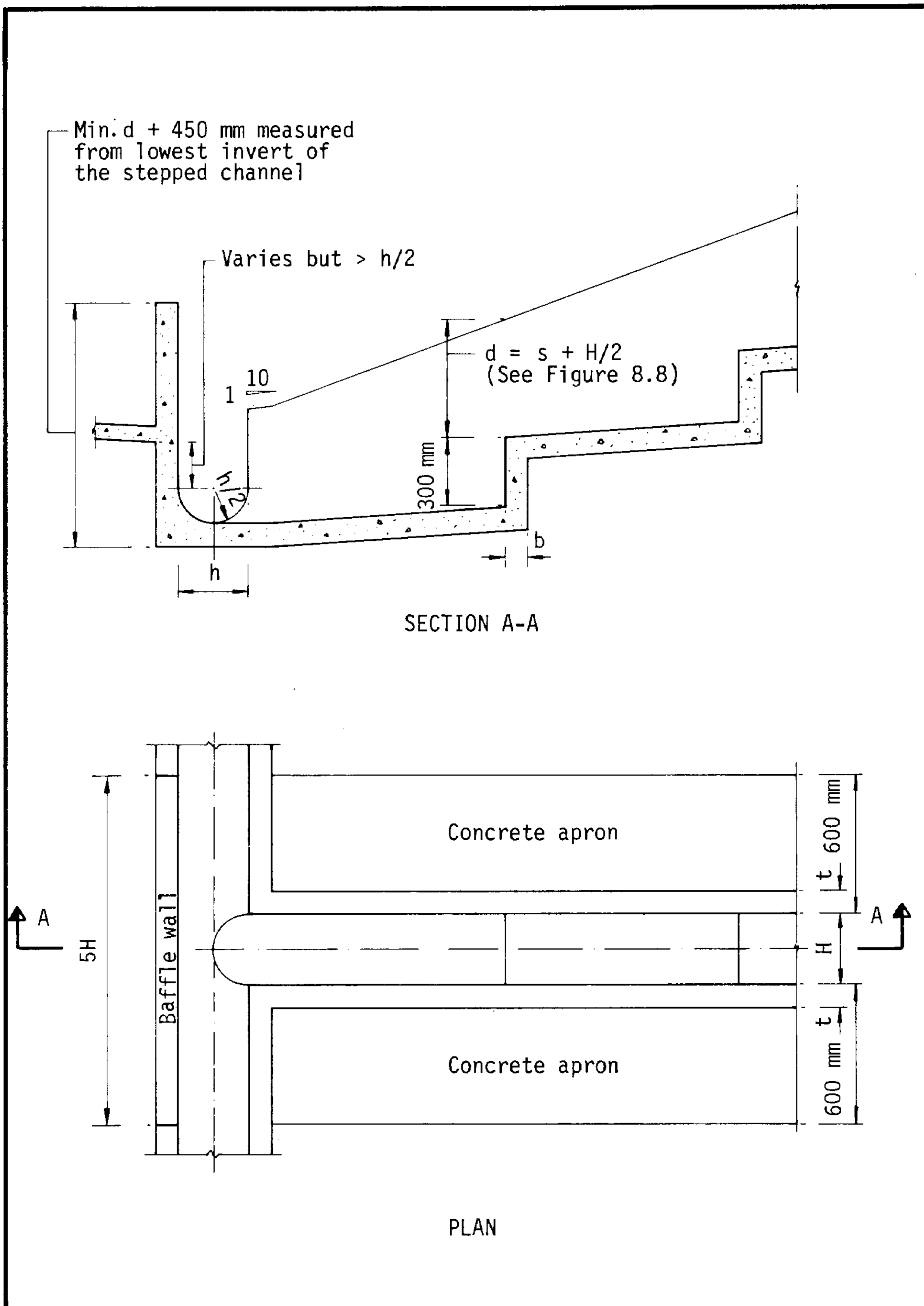
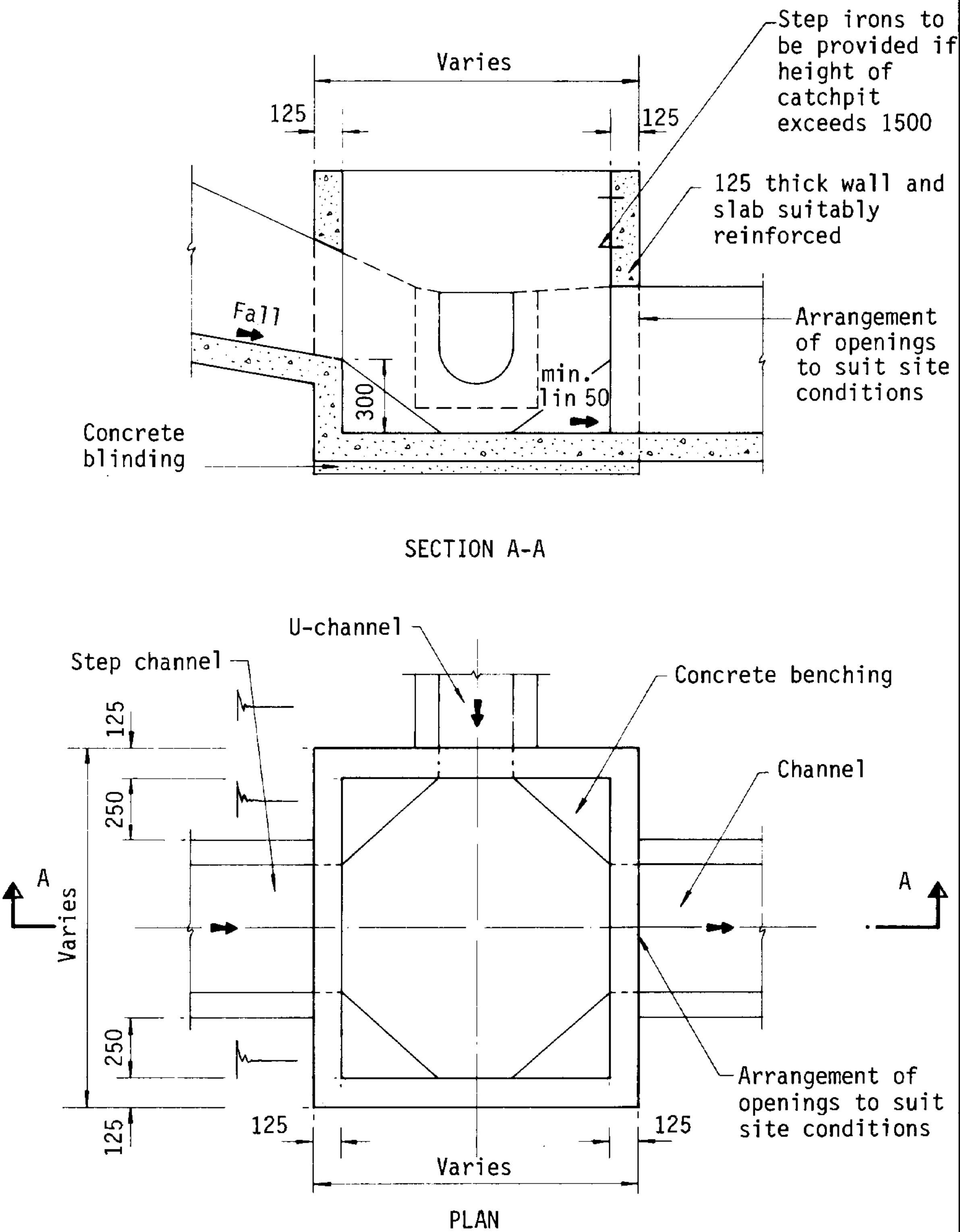
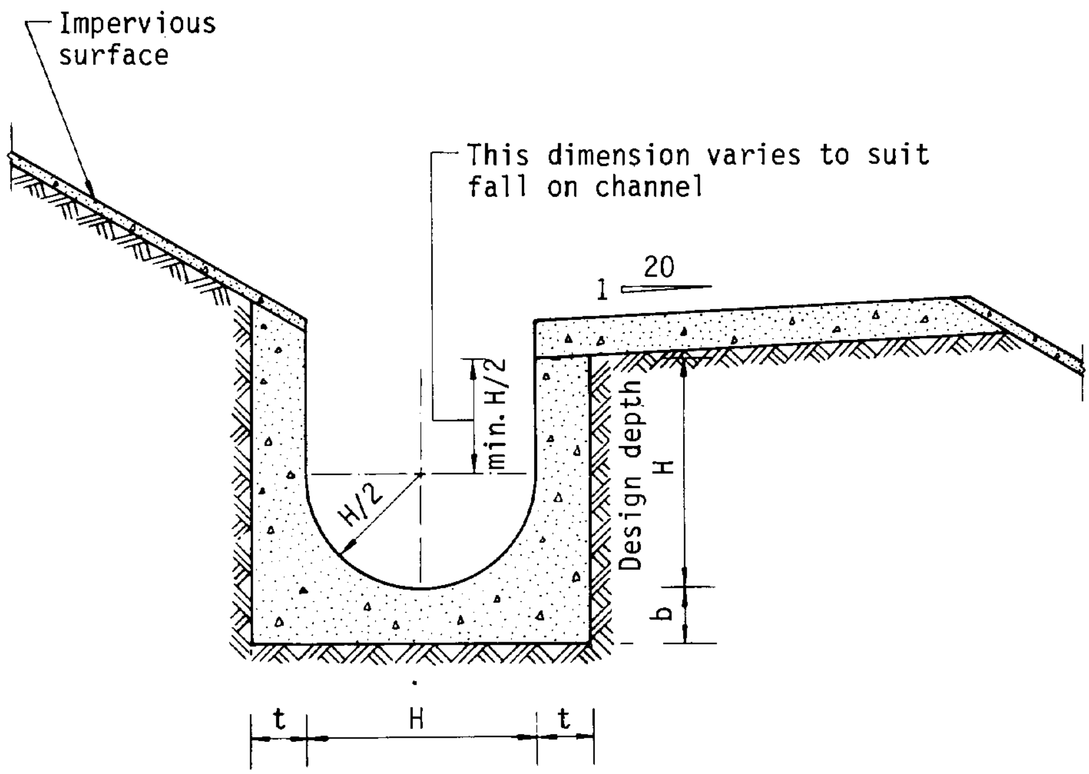


Figure 8.9 - Typical Details of Junction of Stepped Channel and U-channel at Toe of Slope



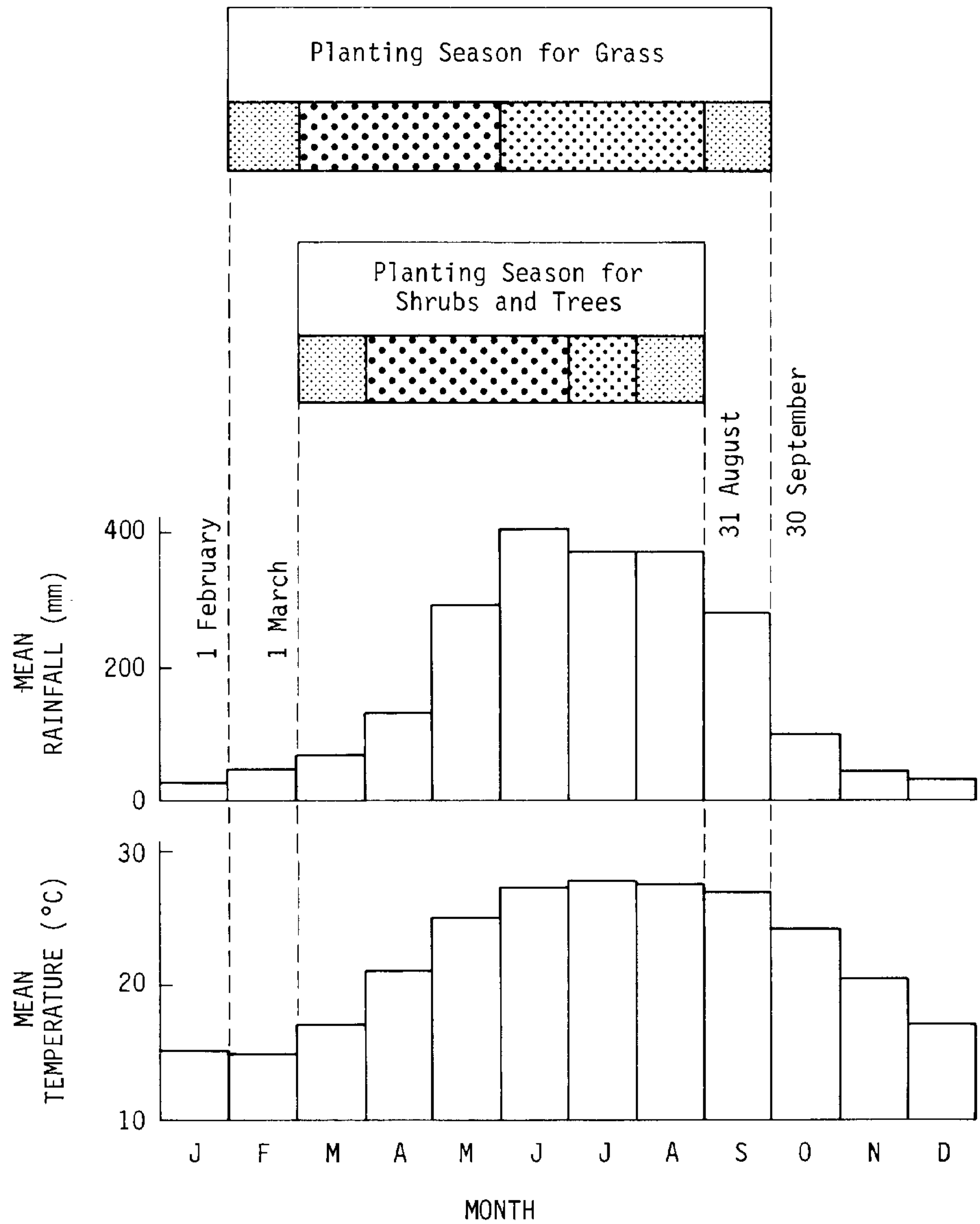
Note : All dimensions in millimetres

Figure 8.10 - Typical Details of Catchpits



Dimensions of U - channel		
Nominal size of channel H (mm)	Thickness t (mm)	Thickness b (mm)
225 to 600	150	150
675 to 1200	175	225

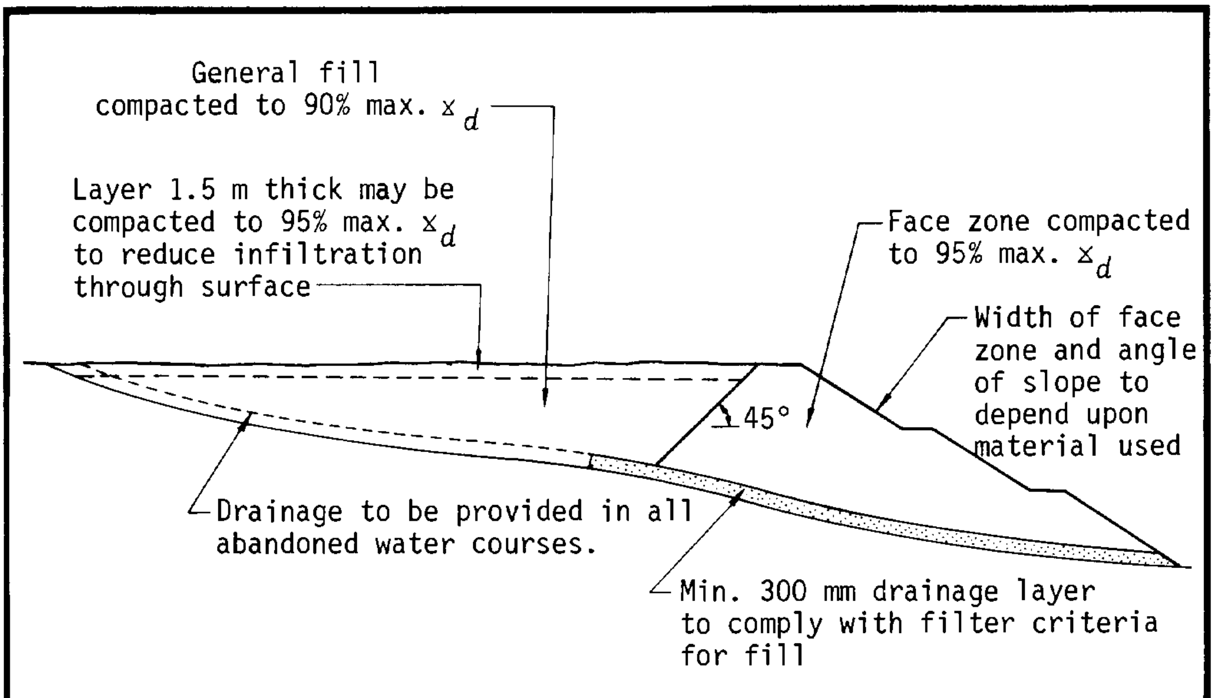
Figure 8.11 - Typical U-channel Details



Legend :

Optimum time
Good time
Fair time

Figure 8.12 - Planting Season in Hong Kong



Note : Maximum dry density (max. γ_d) from B.S. 1377, Test 12.

Figure 9.1 - Typical Detail of General Fill Area Employing Dual Compaction Standards

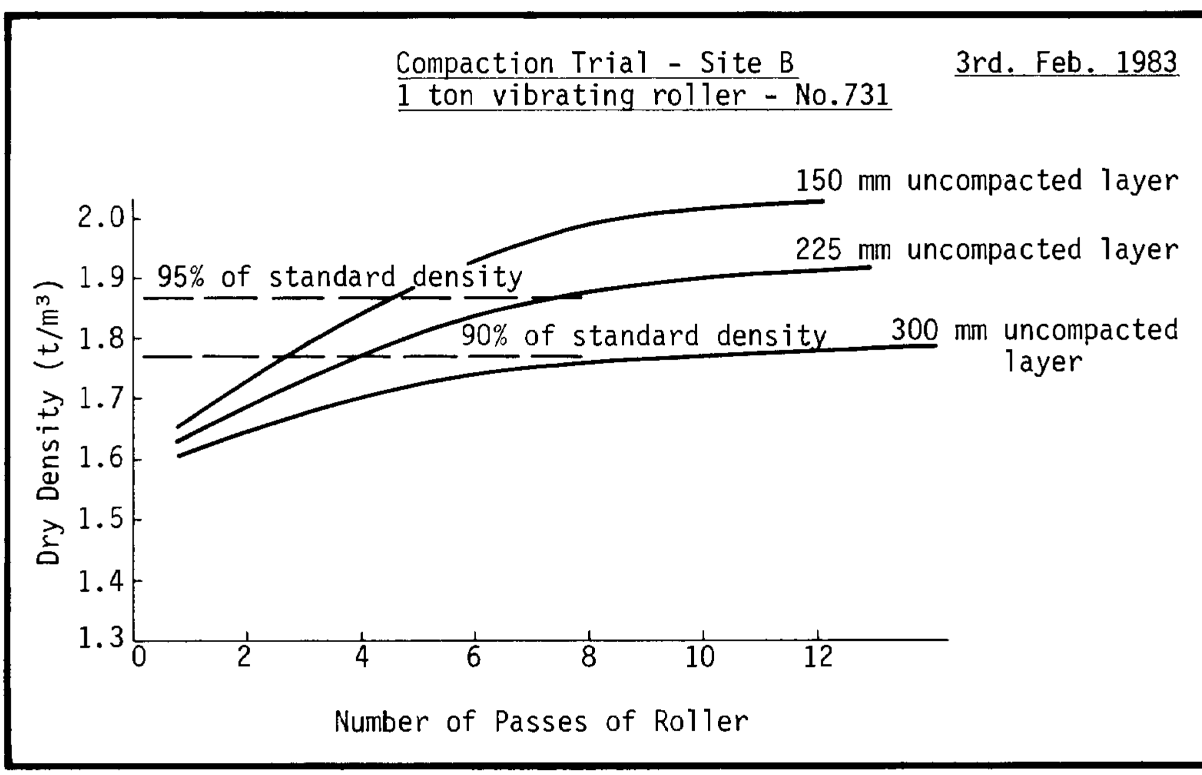


Figure 9.2 - Typical Compaction Trial Results

From to

FILL COMPACTION RECORD

Project

Fill location

Specification requirements :
(a) Relative compaction (%)
(b) Moisture content (W_o - W_t) + % to %

Method of control :
(a) Conventional ☐
(b) Rapid compaction ☐

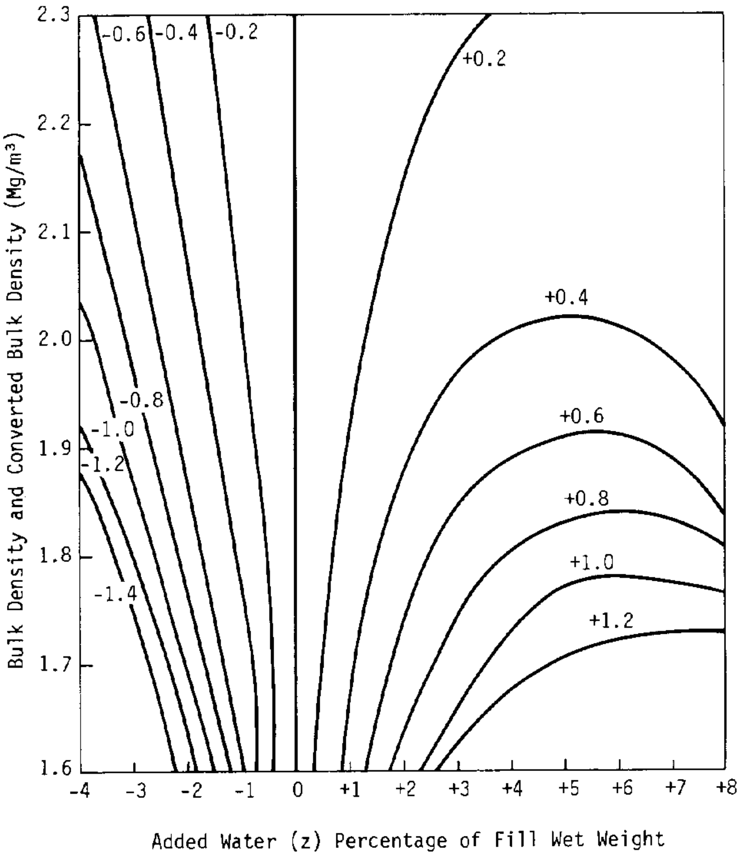
Date	Insitu Density Test				Laboratory Compaction Test Ref. No.	Fill Characteristics			Compaction Plant		Density (Mg/m^3)				Moisture Content (%)			Accept or Reject	Remarks
19	Plan Ref.	Location No.	Layer No.	Method (1)		Type (2)	Source	Compacted Layer Thickness (mm)	No. of Passes	Roller (3) Type	Insitu Bulk γ_{ft}	Insitu Dry γ_{fd}	Max. DD γ_{td}	Rel. Comp. (%)	Insitu W_f	Opt. W_o	Rel. Moist. W_o-W_f		

Note : (1) SR - Sand replacement
RB - Rubber balloon
WR - Water replacement

(2) Unified soil classification

(3) PN - Pneumatic tyred
ST(V) - Steel drum } (V) if vibrating
SF(V) - Sheepsfoot }
Specify roller static weight

Figure 9.3 - Record Sheet for Density Tests in Compacted Fill



Fill bulk density	γ_{ft}	Mg/m^3
Max. converted bulk density	γ_{cm}	Mg/m^3

$W_o - W_f$	m	%
Degree of compaction	$\frac{\gamma_{ft} \times 100}{\gamma_{cm}}$	%

Fill moisture content	W_f	%
Optimum moisture content	W_o	%
Fill dry density	γ_{fd}	Mg/m^3
Lab. max. dry density (derived from Hilf's method)	γ_{ld}	Mg/m^3

Description of Soil/Remarks

Determination of Converted Bulk Density							
Added water (z)	%	0					
Mass mould + base + sample	g						
Mass mould + base	g						
Mass of sample	g						
Volume of mould	cm^3						
Bulk density γ_b	Mg/m^3						
Converted bulk density γ_c	Mg/m^3						

Determination of Fill Moisture Content		
Wet soil + container	g	
Dry soil + container	g	
Container number	g	
Moisture loss	g	
Dry soil	g	
Moisture content W_f	%	

$$\text{Converted bulk density} = \frac{\gamma_{bulk}}{\left(1 + \frac{z}{100}\right)}$$

$$\text{Optimum moisture content } W_o = W_f + Z_m \left(\frac{100 + W_f}{100}\right)$$

Figure 9.4 - Field Sheet for the Hilf Method of Rapid Compaction Control

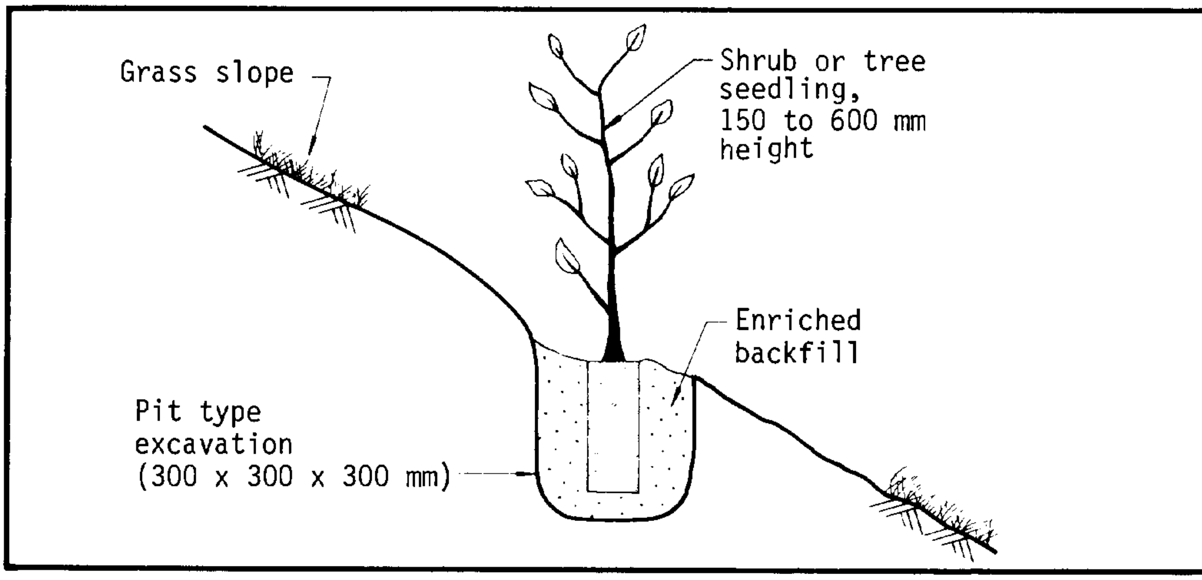
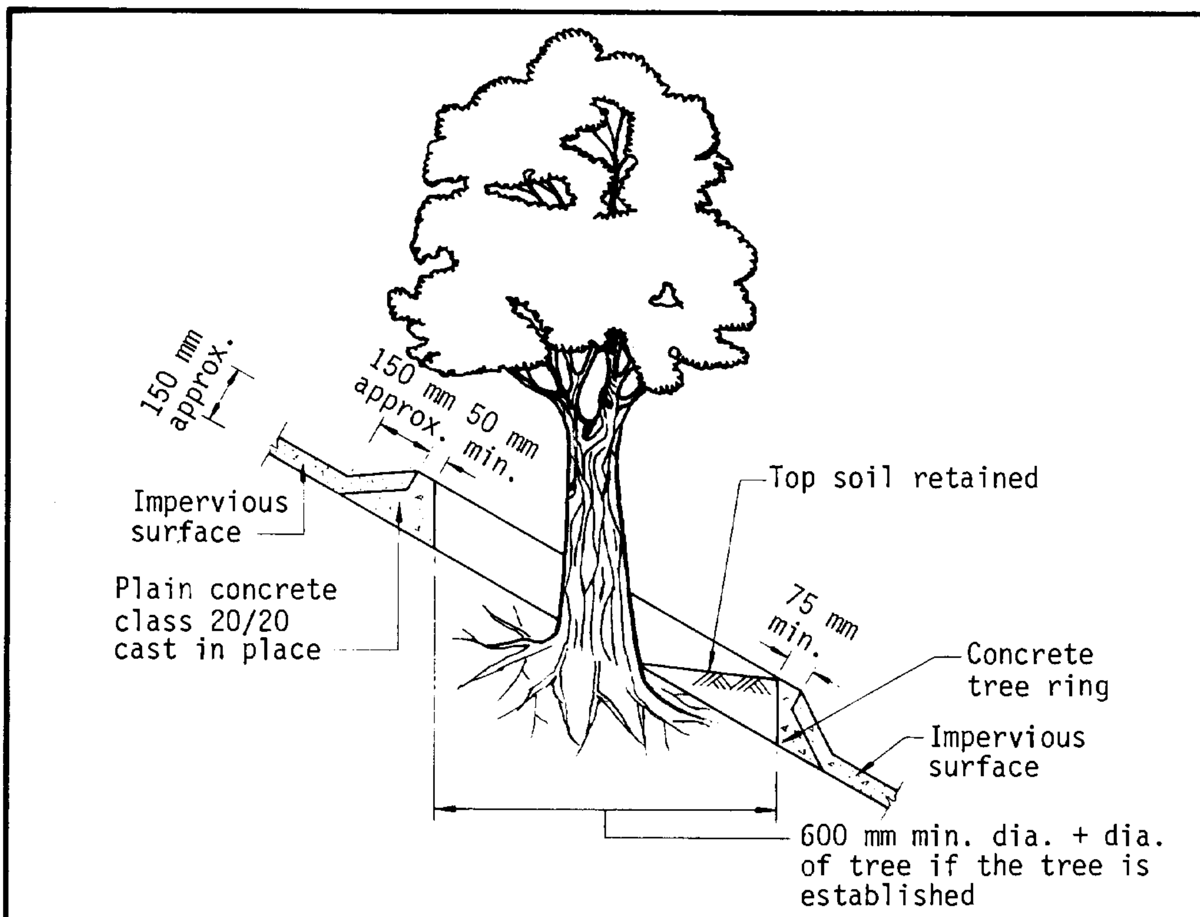


Figure 9.5 - Seedling Shrub or Tree Planting



Note : Precast concrete tree rings may also be used.

Figure 9.6 - Detail of Tree Rings on Impervious Slope Surface

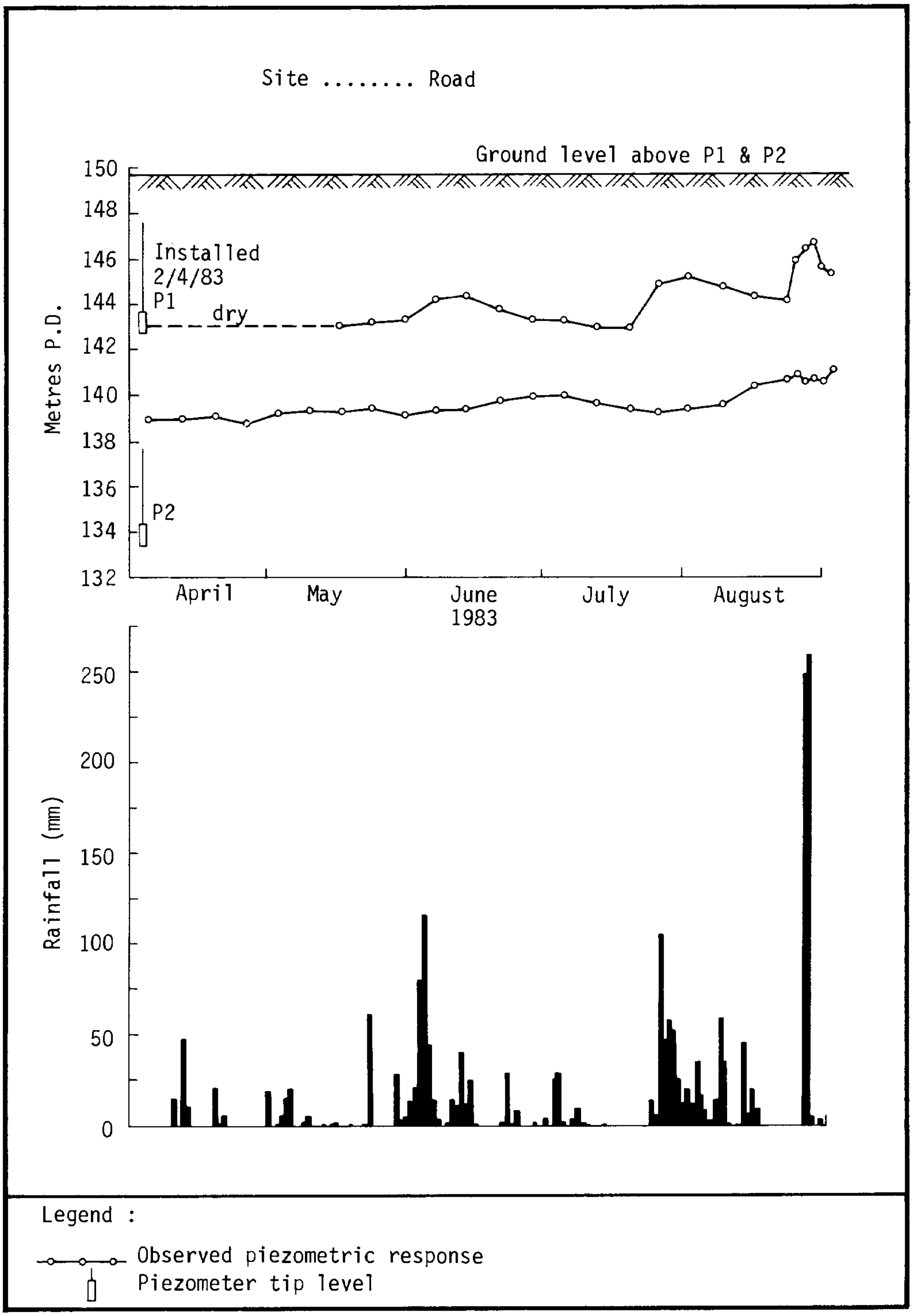


Figure 10.1 - Example of a Piezometer Record Sheet

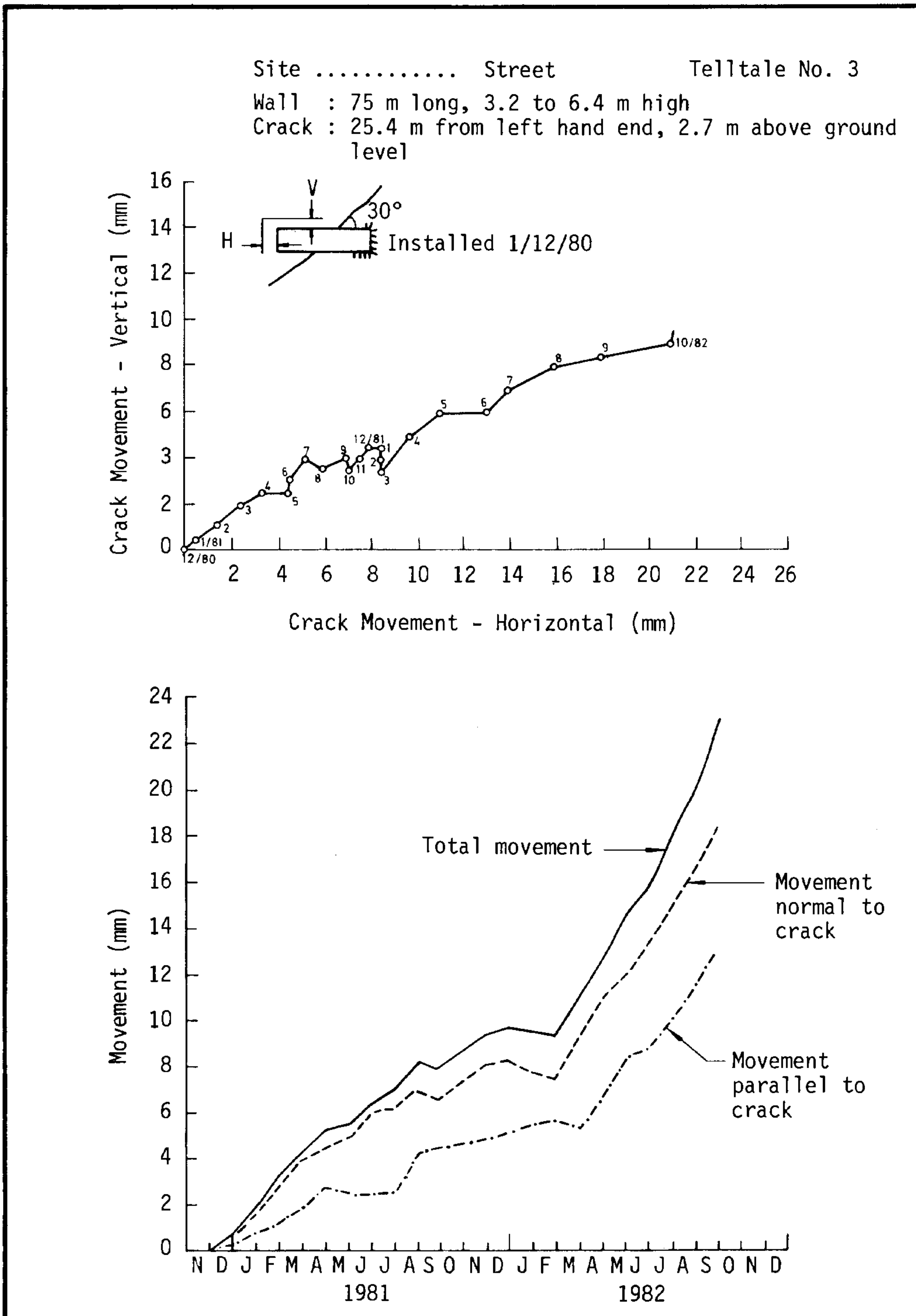


Figure 10.2 - Example of a Telltale Record Sheet

Site : Blank Road

Anchor : No. C4 | Load cell : Vibrating wire
Free length 22.5 m | Deakin No. 738241
Fixed length 6.0 m | Capacity 1 000 kN
Strands : 7 No., 12.5 mm

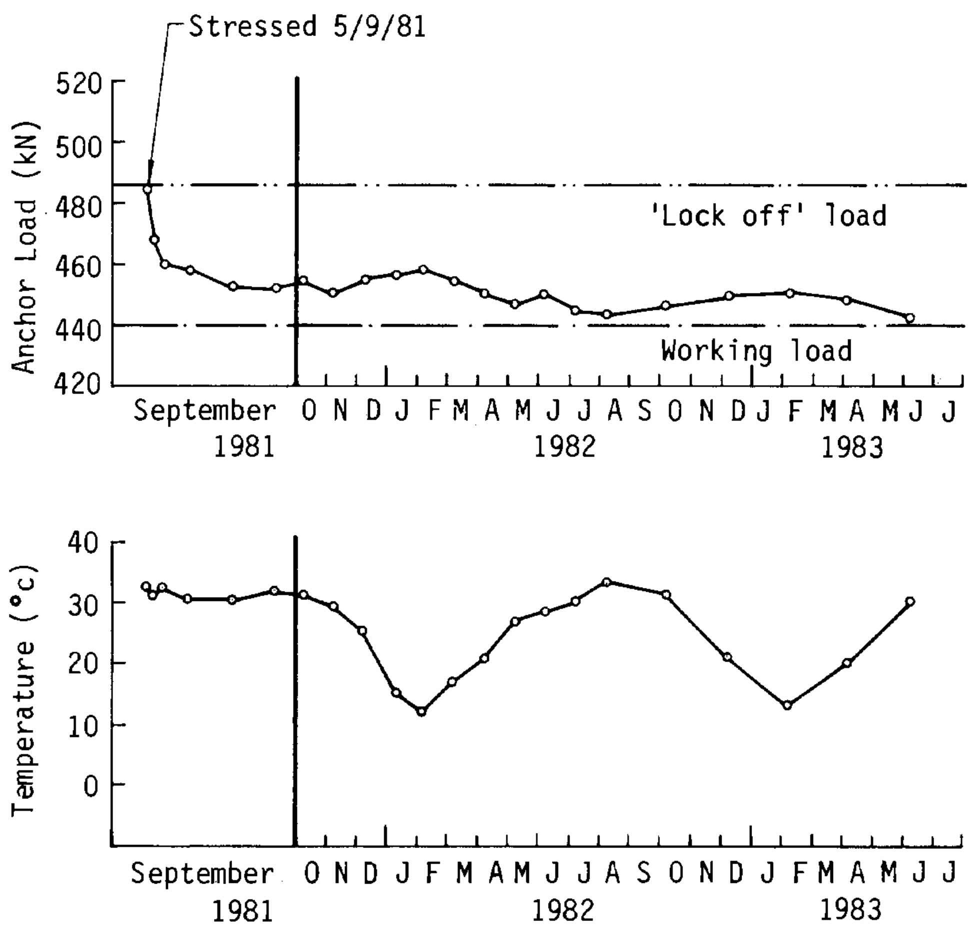


Figure 10.3 - Example of an Anchor Load Record Sheet

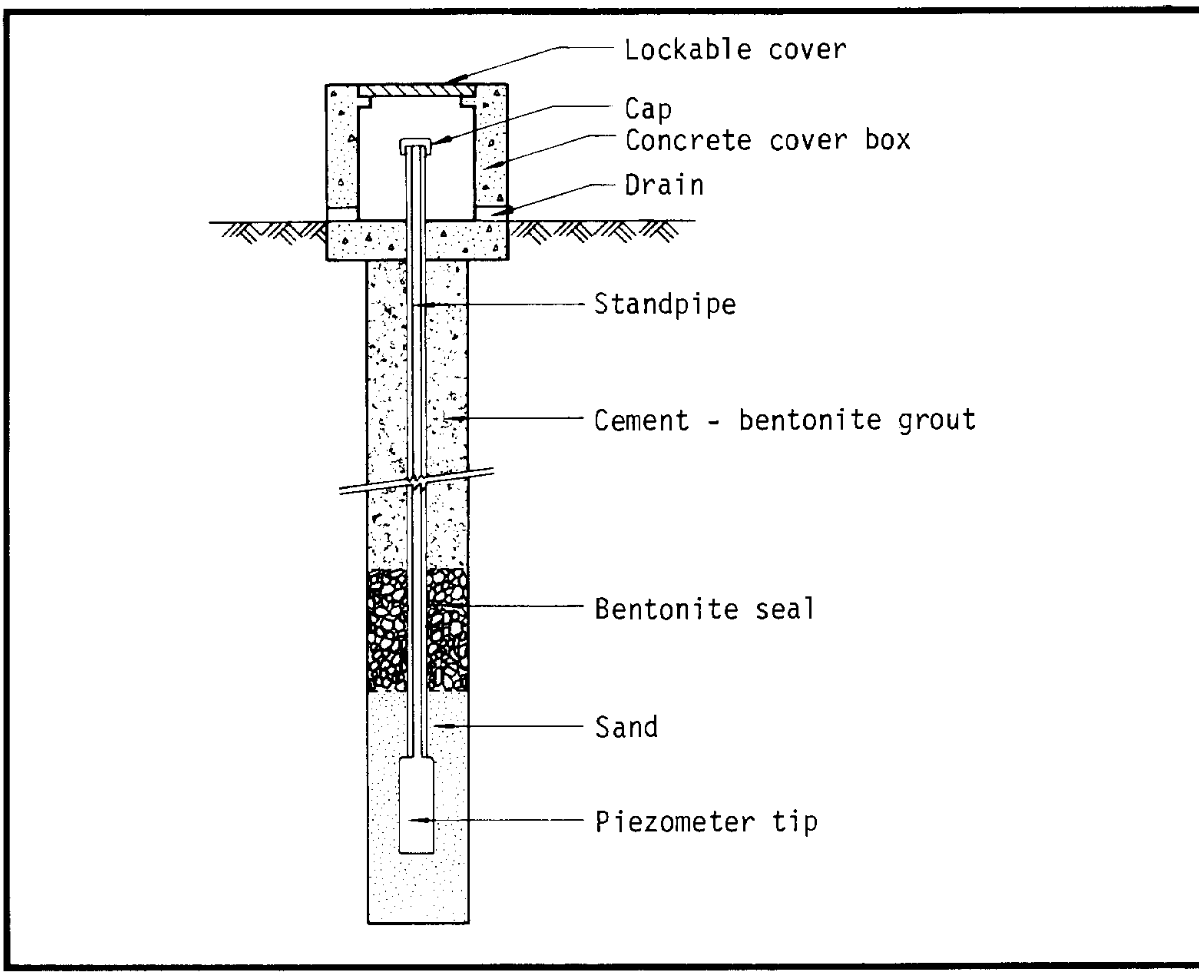


Figure 10.4 - Open Hydraulic (Casagrande) Piezometer

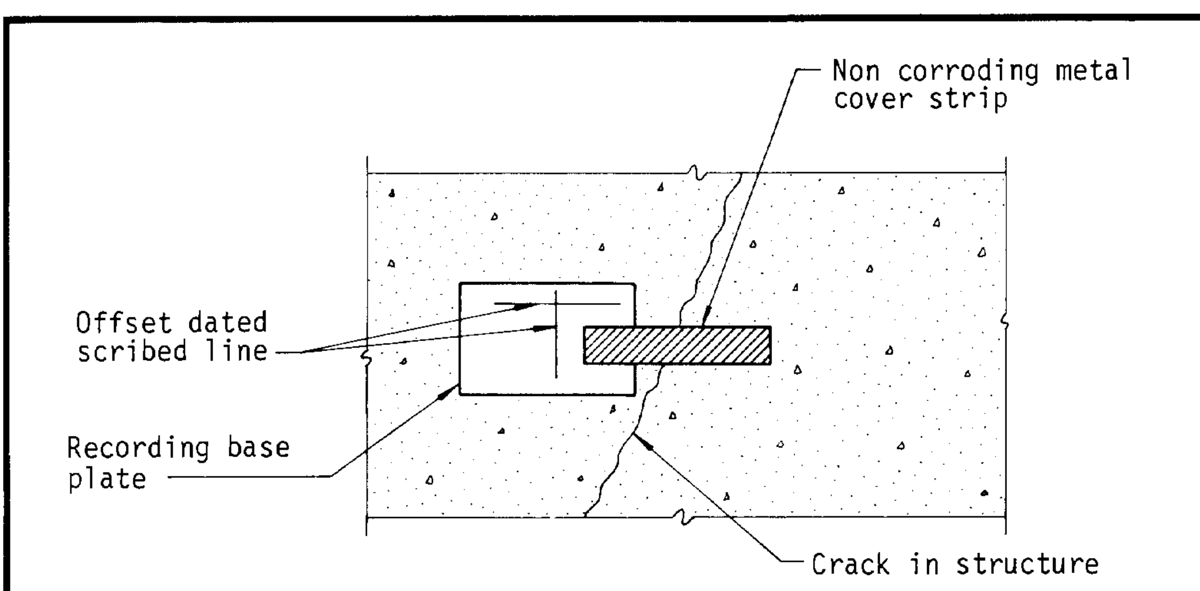


Figure 10.5 - Telltale

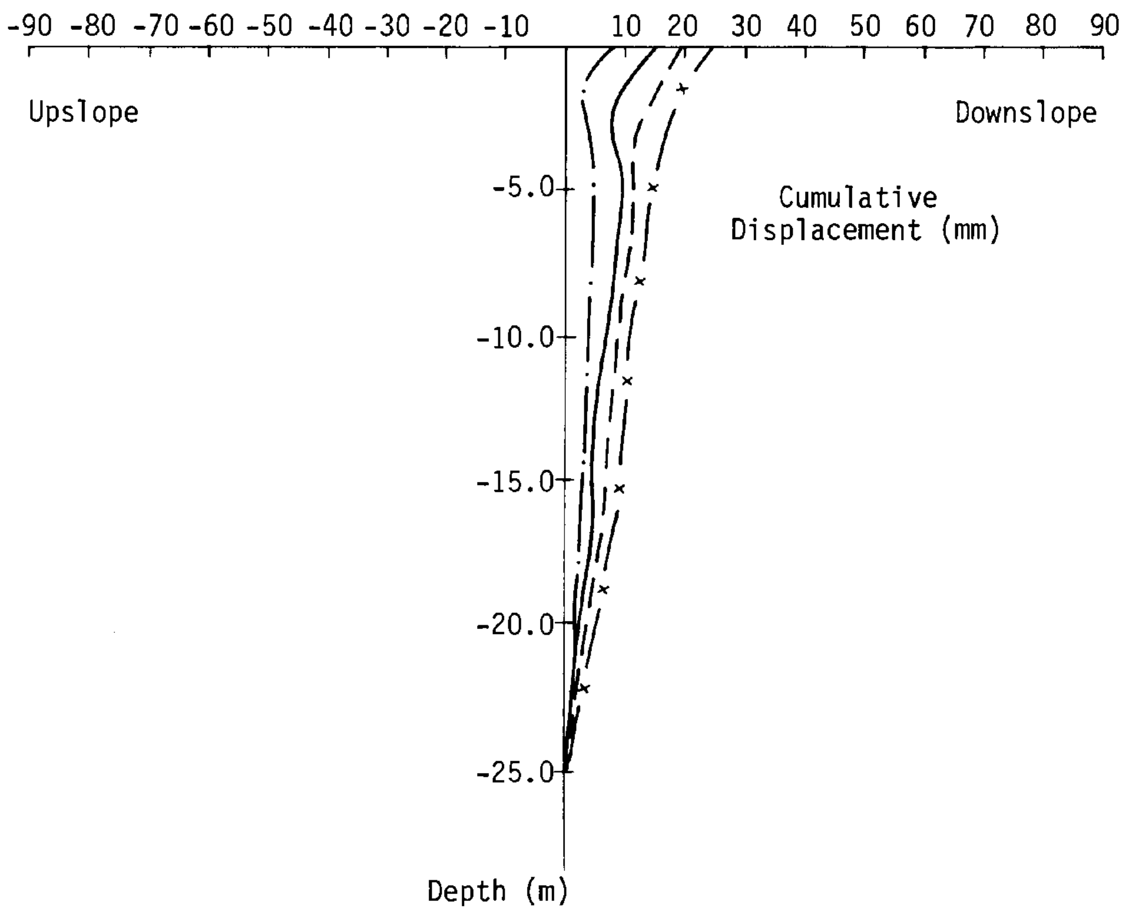
Job :

Movement Profile on Axis : Normal to wall ☒ / Parallel to wall ☐

Site : Retaining Wall A   Inclinometer No. : A1

Base Level of Inclinometer Tube :   m.P.D.

Date of First Observation :   Observer :



Legend :

-·——·-	40211	(9.12.82)
———	40221	(15.12.82)
- — — -	40231	(22.12.82)
— x —	40241	(2. 1.83)

Note : (1) For orientation of inclinometer guide axes, see monitoring key plan

(2) The displacement profile is measured relative to the initial profile

(3) The file code is formed as follows : digit 1 for inclinometer number, digit 2, 3&4 for visit no. and digit 5 for axis notation (1 is axis AB, 2 is axis CD).

Figure 10.6 - Example of an Inclinometer Record Sheet

<table>
<tr><td colspan="4">Piezometer Readings</td></tr>
<tr><td colspan="2" rowspan="2">Development at :
13 Blank Street</td><td>Critical water level</td><td>Depth of critical W.L. below top of standpipe</td></tr>
<tr><td>269.55 m P.D.</td><td>0.75 m</td></tr>
<tr><td colspan="2">Piezometer No. P3</td><td colspan="2">Level of top of standpipe (m)
270.30</td></tr>
<tr><td>Tip level :
254.8 m P.D.</td><td>Ground level :
270.36 m P.D.</td><td colspan="2">Depth of tip below top of standpipe (m)
15.50</td></tr>
</table>

Date	Depth from top of standpipe to water surface(m)	Weather	Comments	Recorded by
26/8/82	5.36	Heavy rain	Cover box clear of water - drain free.	CLM
28/8/82	0.95	Heavy rain	Cover box flooded - drain blocked. Cleared before removing cap.	CLM

Figure 11.1 - Example of a Slope Maintenance Record Sheet for Piezometer Readings

Slope Maintenance Inspection

Slope Location

Page 1 of 3

Slope Number | Weather
Inspecting Officer | Position | Date

Risk Category   Low/Medium/High
Last Inspection Date
Is interval between inspections OK?   Yes/No
Previous Risk Category   Low/Medium/High
Have past recommendations been carried out?   Yes/No

Summary

Major works required	Yes/No
Minor works required	Yes/No
Investigation needed	Yes/No
Slope satisfactory	Yes/No

ACCESS

Is there good maintenance access?	Yes/No
Is it difficult for the public to gain access?	Yes/No
Has the inspecting officer gained access to the crest, the toe, and all berms?	Yes/No

Comments

INSTRUMENTATION

Have all instrumentation systems been checked?	Yes/No
Have all instrumentation results been plotted?	Yes/No
Are all readings acceptable?	Yes/No
Is there a need for new instrumentation?	Yes/No

Comments

P.S. This inspection sheet is to be read together with the slope data sheet/file.

Figure 11.2 - Example of Sheet One of a Maintenance Inspection Record

Slope Maintenance Inspection

Slope Location

Page 2 of 3

CONDITION OF SLOPE

	Status of Feature				
	None	Good	Satis-factory	Works Needed Minor	Works Needed Major
Condition of impermeable surface					
Extent of impermeable surface					
Condition of weepholes					
Capacity of weepholes					
Condition of vegetated surface					
Capacity of surface drainage					
Condition of U-channels & step channels					
Condition of catchpits & sandtraps					
Condition of raking drains					
Condition of associated culverts & nullahs					
Condition of artificial support					
Condition of toe fence/toe barrier					

Comments

		Works Needed Minor	Works Needed Major
Has there been a recent slope failure?	Yes/No		
Has there been any recent erosion?	Yes/No		
Has there been any recent movement?	Yes/No		
Are there any tension cracks at the crest?	Yes/No		
Is there adequate protection against infiltration above the crest?	Yes/No		
Has there been any recent seepage?	Yes/No		

If seepage give details :-

If movements give details :-

Comments

Figure 11.3 - Example of Sheet Two of a Maintenance Inspection Record

Slope Maintenance Inspection

Slope Location

Page 3 of 3

ASSOCIATED RETAINING WALLS

		Works Needed: Minor	Works Needed: Major
Have there ever been wall movements?	Yes/No		
Has there been recent wall settlement?	Yes/No		
Has there been recent wall cracking?	Yes/No		
Has there been recent wall titling?	Yes/No		
Has there been recent wall bulging?	Yes/No		
Is the capacity of the weepholes adequate?	Yes/No		
Are the weepholes clear?	Yes/No		
Are the mortar joints/pointing satisfactory?	Yes/No		
Is vegetation adversely affecting the wall?	Yes/No		

Comments

SERVICES & DRAINAGE

Are services adversely affecting the slope? Yes/No
Do any services need testing? Yes/No
Have the appropriate authority been informed? Yes/No

GENERAL & COMMENTS

Does the slope need upgrading to meet the present risk category? Yes/No

RECOMMENDATIONS

Signature :

Figure 11.4 - Example of Sheet Three of a Maintenance Inspection Record

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PLATES

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LIST OF PLATES

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Plate 1.1 - Granite Soil within Zone A

Plate 1.2 - Volcanic Soil within Zone A

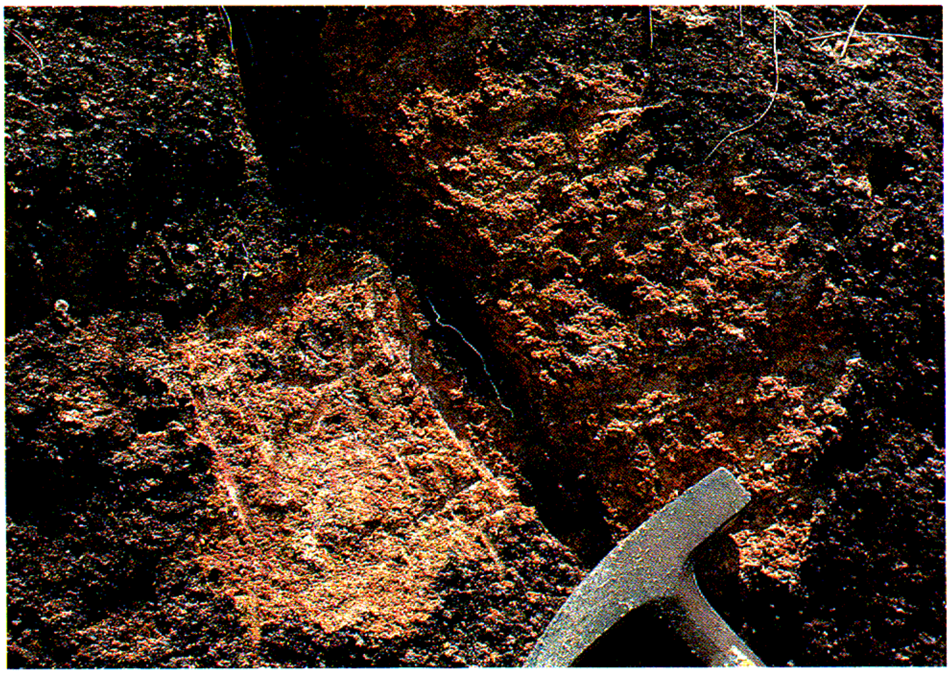


Plate 1.3 - Weathered Granite in Zone B Showing Well-defined Relict Jointing

Plate 1.4 - Weathered Volcanic Rock in Zone B Showing Well-defined Relict Jointing

Plate 1.5 - Typical Exposure of Weathered Granite of Zone B Showing Relict Jointing and Corestones : Zone A is Absent

Plate 1.6 - Junction between Zone B and Zone D Volcanics : Zone C is Absent

Plate 1.7 - Granite Showing Intense Weathering along Joints : Zone C

Plate 1.8 - Cut Slope through Weathered Volcanic Rock Showing Zone B Overlying Zone D : Zones A and C are Absent

Plate 1.9 - Granite Showing Slight Staining on Joints : Blasting Fractures Can Be Seen at Centre Right : Zone D

Plate 1.10 - Volcanic Rock with Staining along Joints : Zone D

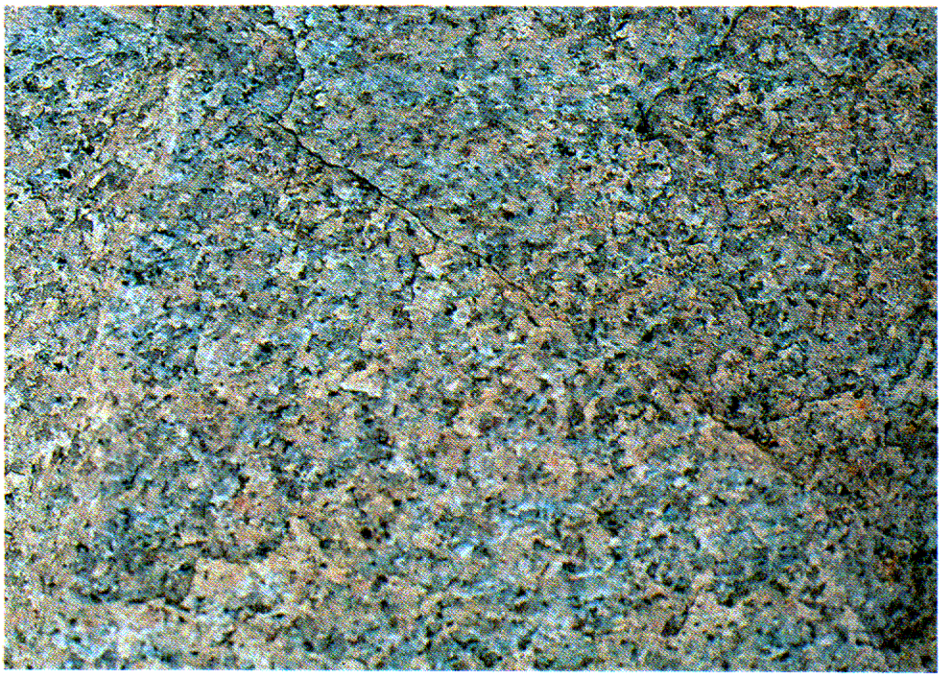
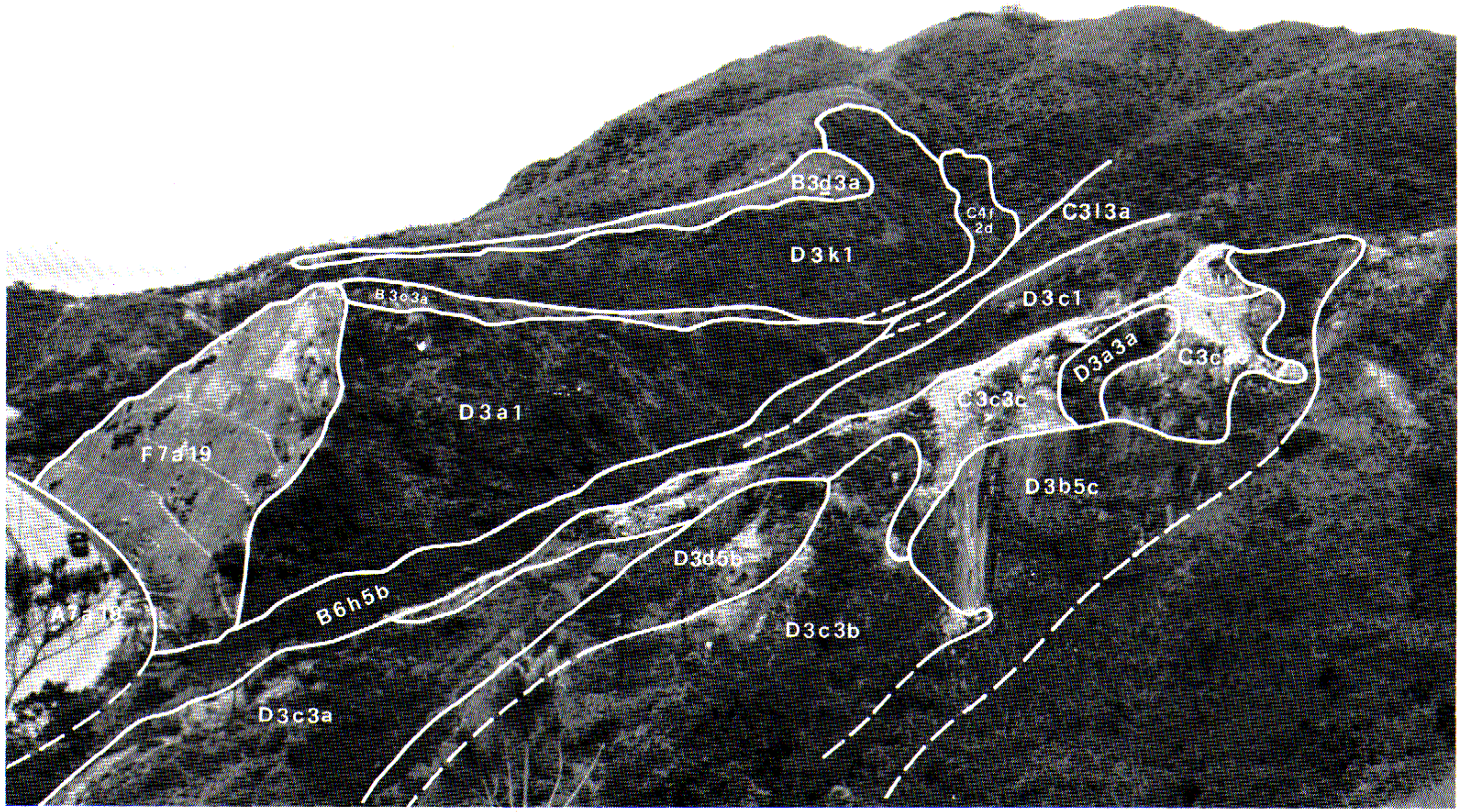


Plate 1.11 - Fresh Granite within Zone D

Plate 1.12 - Fresh Volcanic Rock within Zone D

Plate 1.13 - Granitic Fill Showing Layering Parallel to the Slope

Plate 1.14 - Volcanic Colluvium Showing Boulders Contained in a Structureless Soil Matrix



Note : The terrain classification in this example uses a four or five figure code to represent the observed terrain :

a) The first character represents the slope gradient (e.g. 'D' is a slope gradient of 30-40 degrees).

b) The second character is the terrain component (e.g. '3' is a sideslope, '6' is an incised drainage line and '7' is disturbed terrain).

c) The third character is the terrain morphology and material (e.g. 'c' is convex insitu terrain and 'h' is concave colluvial terrain).

d) The fourth character is an erosion and instability classification; the number indicating the type of erosion or instability, the letter indicating the severity (e.g. '1' is no appreciable erosion and 3b is severe gully erosion).

e) The final character is the depth of cut or fill (e.g. '9' is a cut slope of greater than 30 metres).

There are numerous other codes within this terrain classification system which are used to describe land surface features.

Plate 2.1 - Oblique Photograph that Illustrates the Use of Aerial Photograph Interpretation in Terrain Evaluation

Plate 2.2 - Volcanic Residual Soil (Grade VI), 0 to 0.24 m
Completely Decomposed Volcanic Rock (Grade V), 0.24 to 4.20 m

Plate 2.3 - Completely Decomposed Volcanic Rock (Grade V), 4.20 to 9.21 m
Highly Decomposed Volcanic Rock (Grade IV), 9.21 to 10.05 m
Moderately Decomposed Volcanic Rock (Grade III), 10.05 to 11.04 m

Plate 2.4 - Slightly Decomposed Volcanic Rock (Grade II), 11.04 to 17.33 m

Plate 2.5 - Slightly Decomposed Volcanic Rock (Grade II), 17.33 to 23.39 m

Plate 2.6 - Slightly Decomposed Volcanic Rock (Grade II), 23.39 to 25.08 m

Plate 2.7 - Volcanic Residual Soil (Grade VI) : Detail A

Plate 2.8 - Completely Decomposed Volcanic Rock (Grade V) : Detail B

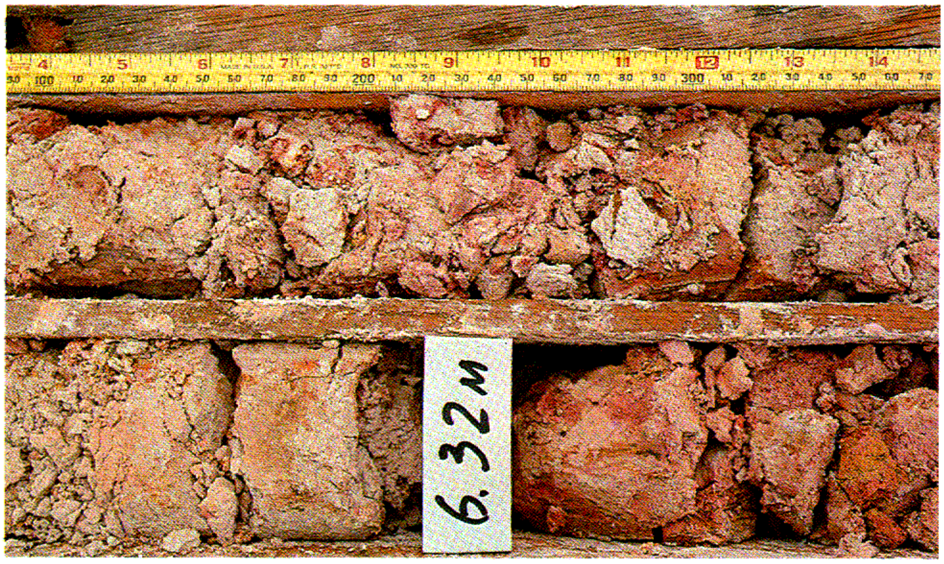


Plate 2.9 - Completely Decomposed Volcanic Rock (Grade V) : Detail C

Plate 2.10 - Completely Decomposed Volcanic Rock (Grade V) with Stained Discontinuities : Detail D

Plate 2.11 - Moderately Decomposed Volcanic Rock (Grade III) : Detail E

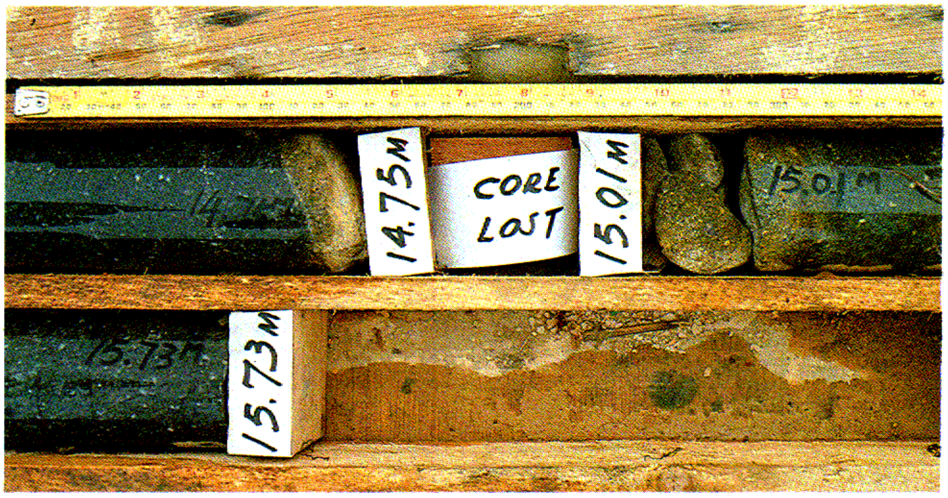


Plate 2.12 - Completely Decomposed Seam (Grade V) in Slightly Decomposed Volcanic Rock (Grade II) : Detail F

Plate 2.13 - Slightly Decomposed Volcanic Rock (Grade II) : Detail G

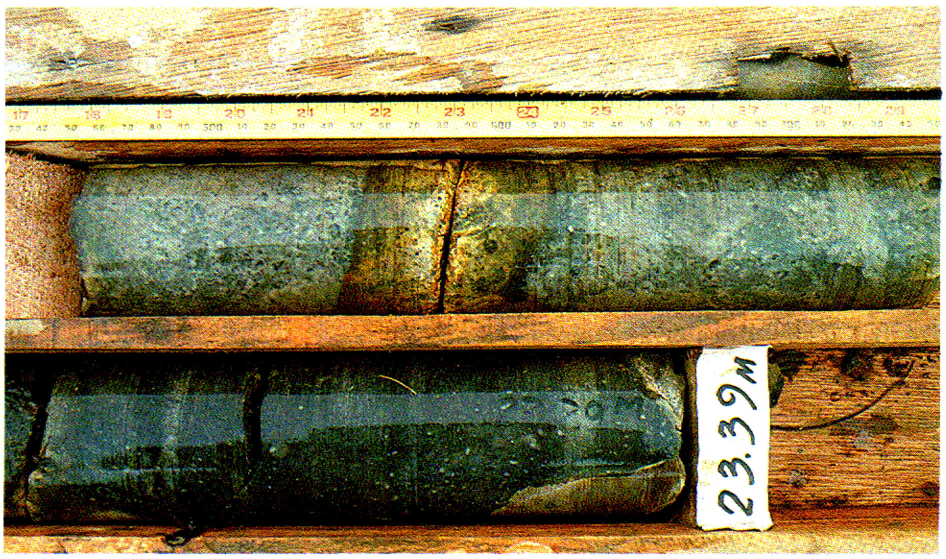


Plate 2.14 - Moderately Decomposed Seam (Grade III) in Slightly Decomposed Volcanic Rock (Grade II) : Detail H

Plate 2.15 - Completely Decomposed Granite Rock
(Grade V), 0 to 1.00 m
Highly Decomposed Granite Rock
(Grade IV), 1.00 to 3.59 m
Moderately Decomposed Granite
Rock (Grade III), 3.59 to 4.65 m

Plate 2.16 - Moderately Decomposed Granite Rock
(Grade III), 4.65 to 5.55 m
Highly Decomposed Granite Rock
(Grade IV), 5.55 to 10.71 m

Plate 2.17 - Slightly Decomposed Granite Rock
(Grade II), 10.71 to 13.71 m

Plate 2.18 - Highly Decomposed Granite Rock (Grade IV), 13.71 to 16.45 m
Moderately Decomposed Granite Rock (Grade III), 16.45 to 17.95 m

Plate 2.19 - Highly Decomposed Granite Rock (Grade IV), 17.95 to 20.17 m
Slightly Decomposed Granite Rock (Grade II), 20.17 to 21.23 m
Moderately Decomposed Granite Rock (Grade III), 21.23 to 21.48 m

Plate 2.20 - Moderately Decomposed Granite Rock (Grade III), 21.48 to 22.00 m
Slightly Decomposed Granite Rock (Grade II), 22.00 to 24.48 m

Plate 2.21 - Slightly Decomposed Granite Rock
(Grade II), 24.48 to 25.95 m

Plate 2.22 - Highly Decomposed Granite Rock
(Grade IV) : Detail A

Plate 2.23 - Moderately Decomposed Granite Rock
(Grade III) : Detail B

Plate 2.24 - Highly Decomposed Granite Rock
(Grade IV) : Detail C

Plate 2.25 - Slightly Decomposed (Grade II) Becoming
Moderately Decomposed Granite Rock
(Grade III) : Detail D

Plate 2.26 - Highly Decomposed Seam (Grade IV) in Slightly
Decomposed Granite Rock (Grade II) : Detail E

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ADDENDUM

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Readers should note that the following Chapters/Sections of this Manual have been supplemented or superseded by later publications:

(a) Chapter 1 and Section 2.3.3 are superseded by the Geological Survey Maps and Memoirs and Geoguide 3 (GCO, 1988), which should be read in conjunction with GEO Publication No. 1/2007 (GEO, 2007a) and GEO Technical Guidance Note (TGN) No. 10 (GEO, 2009a).

(b) Chapter 2 (except Section 2.3.3), Sections 3.5 and 10.2 are superseded by Geoguide 2 (GCO, 1987), which should be read in conjunction with GEO TGN Nos. 3 (GEO, 2004d), 5 (GEO, 2009b), 6 (GEO, 2004a) and 10 (GEO, 2009a).

(c) Section 4.6 is superseded by GEO Publication No. 1/93 (GEO, 1993a).

(d) Section 5.2.2 is supplemented by GEO TGN No. 6 (GEO, 2004a).

(e) Sections 5.2.2, 5.3.6, 5.5.1, 5.5.2, and 9.5 are supplemented by the Works Bureau Technical Circular No. 13/99 (Works Bureau, 1999), which should be read in conjunction with GEO TGN Nos. 7 (GEO, 2004b) and 15 (GEO, 2007b). Section 5.2.4 is superseded by this Technical Circular, and Tables 5.1 to 5.4 are superseded by Tables 1 to 4 of the same Technical Circular.

(f) Section 5.3 is supplemented by GEO Report No. 138 (Ng et al, 2003) and GEO Publication No. 1/2007 (GEO, 2007a).

(g) Section 5.4 is supplemented by Geoguide 7 (GEO, 2008), GEO Publication No. 1/2009 (GEO, 2009c) and GEO TGN No. 10 (GEO, 2009a).

(h) Sections 5.5 and 9.5 are supplemented by GEO TGN No. 7 (GEO, 2004b).

(i) Section 5.5.1 is supplemented by Geoguide 6 (GEO, 2002).

(j) Section 6.2.1 is supplemented by Appendix A.3 of Geoguide 1 (1993b).

(k) Chapter 7 (except the parts relevant to the design of remedial or preventive works to existing gravity retaining walls as given in Section 7.3.3) is superseded by Geoguide 1 (GEO, 1993b), GCO Publication No. 1/90 (GCO, 1990) and GEO Circular No. 33 (GEO, 2004c).

(l) The last paragraph of Section 8.3.4 is superseded by GEO TGN No. 27 (GEO, 2006), and Figures 8.6 and 8.8 of this Manual have been updated.

(m) Sections 8.4, 9.6 and 11.4.2 are supplemented by GEO Publication No. 1/2000 (GEO, 2000) and GEO TGN No. 20 (GEO, 2007c).

(n) Chapter 11 is superseded by Geoguide 5 (GEO, 2003).

(o) The addresses of organisations referred to in Section 12.5 have been updated.

(p) References to BS 1377:1975 concerning Phase 1 tests described in Works Branch Technical Circular 6/94 are replaced by Geospec 3 (GEO, 2001).

(q) "The Hong Kong Bibliography" referred to in the Manual is the Bibliography on Geology and Geotechnical Engineering of Hong Kong (Brand, 1984), the updated version of which can be found at the Hong Kong Slope Safety website on the Internet: http://hkss.cedd.gov.hk/hkss/eng/education/bb_geology_gehk/lib_query.htm.

The Works Bureau Technical Circular referred to in (e) above, together with the latest information on the list of GEO publications including the complete list of the series of Hong Kong Geological Survey Maps and Memoirs, can be found at the following websites: http://www.devb.gov.hk and http://www.cedd.gov.hk/eng/publications/geo/index.htm on the Internet respectively.

Copies of the GEO Circular No. 33 referred to in (k) above can be obtained from the Technical Secretary of the Geotechnical Engineering Office, Civil Engineering and Development Department, Civil Engineering and Development Building, 101 Princess Margaret Road, Homantin, Kowloon, Hong Kong, (Tel: (852) 2762 5087, Fax: (852) 2715 0501, E-mail: enquiry@cedd.gov.hk).

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(a) Geotechnical Control Office (1987). Guide to Site Investigation (Geoguide 2) (Reprinted, 2000). Geotechnical Control Office, Hong Kong, 359 p.

(b) Geotechnical Control Office (1988). Guide to Rock and Soil Descriptions (Geoguide 3), (Reprinted, 2000). Geotechnical Control Office, Hong Kong, 186 p.

(c) Geotechnical Control Office (1990). Review of Design Methods for Excavations (GCO Publication No. 1/90) (Reprinted, 2003). Geotechnical Control Office, Hong Kong, 187 p.

(d) Geotechnical Engineering Office (1993a). Review of Granular and Geotextile Filters (GEO Publication No. 1/93). Geotechnical Engineering Office, Hong Kong, 141 p.

(e) Geotechnical Engineering Office (1993b). Guide to Retaining Wall Design (Geoguide 1), (Second Edition) (Reprinted, 2007). Geotechnical Engineering Office, Hong Kong, 258 p.

(f) Geotechnical Engineering Office (2000). Technical Guidelines on Landscape Treatment and Bio-engineering for Man-made Slopes and Retaining Walls (GEO Publication No. 1/2000). Geotechnical Engineering Office, Hong Kong, 146 p.

(g) Geotechnical Engineering Office (2001). Model Specification for Soil Testing (Geospec 3). Geotechnical Engineering Office, Hong Kong, 340 p.

(h) Geotechnical Engineering Office (2002). Guide to Reinforced Fill Structure and Slope Design (Geoguide 6). Geotechnical Engineering Office, Hong Kong, 236 p.

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