GUIDE TO RETAINING WALL DESIGN

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

FOREWORD

This Geoguide presents a recommended standard of good practice for the design of retaining walls in Hong Kong. The first edition of the Geoguide was published in July 1982 and preparation for a second edition begun in 1985 with a consultation exercise to obtain the views of practitioners on the first edition. Work on the revision began in 1987.

The Geoguide covers the types of retaining walls which are commonplace in Hong Kong, including conventional reinforced concrete walls, gravity walls such as crib walls, gabion walls and mass concrete walls, and cantilevered retaining walls. It does not aim to deal comprehensively with maritime structures nor structures used to support the sides of deep excavations, such as sheet pile walls. The recommendations given herein are also not intended for retaining wall design which is outside existing experience derived from practice in Hong Kong. In giving recommendations on design this Geoguide adopts the limit state method with partial factors of safety.

Reference has been made in preparing this Geoguide to the results of research on retaining walls and to various national codes and guidance documents. These include the UK draft revised CP2, the Canadian Foundation Engineering Manual, the Danish Code of Practice for Foundation Engineering, the US Navy Design Manual 7.1 and 7.2, the then New Zealand Ministry of Works and Development's Retaining Wall Design Notes and the draft Eurocode No. 7.

The Geoguide was prepared by a team in the Special Projects Division of the Office led by Dr P.L.R. Pang under the overall supervision initially of Dr R. P. Martin and later Mr Y. C. Chan and including at various times Mr W. T. Chan, Mr W. K. Pun and Dr C. Y. Yung. The membership of the Steering Committee at various times over the past five years is given on the opposite page.

To ensure that the Geoguide would be considered a consensus document by interested parties in Hong Kong, a draft version was circulated locally and abroad for comment in early 1992. Those consulted included consulting engineers, contractors, academics, professional bodies and Government departments. Many individuals and organisations made very useful comments, which have been taken into account in finalising the Geoguide, and their contributions are gratefully acknowledged.

As with other Geoguides, this document gives guidance on good engineering practice, and its recommendations are not intended to be mandatory. It is recognised that experienced practitioners may wish to use alternative methods to those recommended herein. Practitioners are encouraged to comment at any time to the Geotechnical Engineering Office on the contents of this Geoguide, so that improvements can be made to future editions.

A. W. Malone
Principal Government Geotechnical Engineer
October 1993

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1. INTRODUCTION

1.1 PURPOSE AND SCOPE

The purpose of this Geoguide is to recommend a standard of good practice for the design of retaining walls in Hong Kong. The document is aimed at qualified civil, geotechnical and structural engineers who are conversant with the relevant engineering principles and procedures.

The geotechnical standards set out in this Geoguide are for new permanent earth retaining walls on land. Design of maritime structures and assessment of existing retaining walls are beyond its scope. Temporary structures which are used to support the sides of deep excavations (e.g. strutted and anchored walls formed with sheet piles, soldier piles, grouted piles or diaphragm wall panels) are also not covered. A review of the design methods for such structures is given in GCO Publication No. 1/90 : Review of Design Methods for Excavations (GCO, 1990).

General considerations in the design of retaining walls are outlined in Chapters 2, 3 and 4. The limit state approach incorporating partial safety factors has been adopted in this Geoguide (see Pang (1992) for an outline of the background development work). Appropriate partial safety factors for different types of loadings and material parameters are given in Chapter 4.

In carrying out the structural design for a retaining wall, the loadings due to soil, water and surcharge should be calculated using the principles and procedures given herein. If a limit state structural design code is adopted, appropriate partial safety factors on loading and structural material parameters as recommended in the code should be used. The loadings calculated in accordance with this Geoguide may also be used in conjunction with structural codes adopting the permissible stress approach to design.

Guidance on the evaluation of geotechnical parameters, earth pressures, and the effects of surcharge, seismic load and water is given in Chapters 5, 6, 7 and 8.

Many types of retaining walls are in use in Hong Kong. Those covered by this Geoguide are :

        (a) gravity retaining walls, which include mass concrete retaining walls, crib walls, gabion walls and reinforced fill structures,

        (b) reinforced concrete (R.C.) retaining walls, which include R.C. L-shaped and inverted T-shaped cantilever, counterfort and buttressed retaining walls, and

        (c) cantilevered retaining walls, e.g. bored pile walls.

Guidance on the choice of the type of wall to be used is given in Section 2.4. Selected design and construction aspects of the various types of retaining walls listed above

are dealt with in Chapters 9, 10 and 11 respectively.

Masonry retaining walls are not covered in this Geoguide.

As with other forms of construction, adequate supervision should be provided during construction of a retaining wall. Where necessary, the retaining wall should be monitored during and after construction. Retaining walls should be regularly maintained. Guidance on these subjects is given in Chapters 12 and 13.

There are a few terms used with specific meanings in this Geoguide or in Hong Kong. Their meanings are given in the Glossary of Terms at the end of this document.

1.2   GENERAL GUIDANCE

1.2.1   General

Recommendations on different aspects of design are given throughout this document, and frequent reference is made to relevant codes, textbooks and other published information. The reader should consult these original publications for more detailed coverage of particular aspects of the subject matter.

Engineering judgement should always be exercised in applying the theories and design methods presented. In particular, the practitioner should be aware of the limitations of the basic assumptions employed in a particular theoretical or computational method.

1.2.2   Personnel

The various stages of planning, investigation, design and construction of a retaining wall demand a range of skills. It is essential that personnel involved in the design have geotechnical knowledge appropriate to the project in hand. In particular, personnel involved in ground investigation should have appropriate specialized knowledge and experience, see Geoguide 2 (GCO, 1987).

It is essential that the relevant information and design assumptions are communicated clearly at all stages to safeguard against misunderstanding and the misuse of information. This is particularly important where the geotechnical and structural design are not carried out by the same person. The personnel responsible for supervision of site work must be aware of the critical design assumptions and check their validity on site.

1.2.3   Local Experience

In geotechnical designs, previous experience of the construction and performance of similar structures in similar conditions is often quoted. The term "local experience" refers to documented or other clearly-established information related to the ground conditions and type of structure being considered in design, involving the same geological materials and for which similar geotechnical behaviour is expected. Knowledge of local experience can be

invaluable in the design of retaining walls.

1.2.4 Geology

The geology of Hong Kong is very briefly summarised in Figure 1. Much of the accumulated local experience in geology has been derived within a fairly restricted area of the territory, broadly the older developed areas of urban Hong Kong Island, Kowloon Peninsula and Tsuen Wan. With spread of development into the Northwest New Territories and Lantau Island, areas with less familiar geology are being encountered. Due care must be exercised in carrying out geotechnical design in areas with less familiar geology and ground conditions.

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2. DESIGN CONSIDERATIONS

2.1 GENERAL

The purpose of a retaining wall is to withstand the forces exerted by the retained ground, and to transmit these forces safely to the foundation. Excessive deformation or movement of the retaining wall and of the adjoining ground is also to be avoided.

The cost of constructing a retaining wall is usually high compared with the cost of forming a new slope. Therefore, the need for a retaining wall should be assessed carefully during design and an effort should be made to keep the retained height as low as possible.

Special considerations are often necessary for retaining walls to be constructed close to land boundaries, particularly in urban areas. Land-take requirements often place limitations on the use of certain forms of earth retention. The permission of the adjacent land owner will need to be sought for excavation limits or structural elements to be extended into the adjacent land.

2.2 PERFORMANCE CRITERIA, LIMIT STATES AND DUCTILITY

2.2.1 Performance Criteria

A retaining wall, and each part of it, is required to fulfil fundamental requirements of stability, stiffness, durability, etc., during construction and throughout its design life.

The performance criteria which need to be considered in the design are indicated for each type of retaining wall in Chapters 9 to 11. Guidance on design calculation models and prescriptive measures which can normally be used to ensure that the performance criteria will be met are also given in these chapters. However, alternative design approaches can also be used where they can be justified (see Chapter 4). Such alternatives will usually necessitate additional geotechnical and soil-structure interaction analyses and calculations, or additional supervision and monitoring of site works. Wherever possible, the design should be critically examined in the light of local experience.

2.2.2 Limit State Method

When a retaining wall, or part of it, fails to satisfy any of its performance criteria, the wall is deemed to have reached a 'limit state'. This Geoguide adopts the 'limit state method' in which various limit states are considered separately in the design and their occurrence is either eliminated or is shown to be sufficiently unlikely. The limit states discussed cover, as far as possible, the full range of possible limit states for each type of retaining wall. However, the designer must judge whether additional limit states should be taken into account based on the particular site conditions or other requirements not covered herein.

The following two main classes of limit state should be considered :

(a) *Ultimate limit state* - a state at which a failure mechanism can form in the ground or in the retaining wall, or severe structural damage (*e.g. yielding or rupture*) *occurs in* principal structural elements.

(b) *Serviceability limit state* - a state at which specified serviceability criteria are no longer met.

For simplicity in design, states prior to collapse are often considered in place of the collapse itself, e.g. forward sliding of a gravity retaining wall. These states are also classified and treated as ultimate limit states.

Serviceability limit states include strains or movements in a retaining wall which would render the wall unsightly, result in unforeseen maintenance or shorten its expected life. They also include deformations in the ground which are of concern, or which affect the serviceability of any adjacent structures or services.

2.2.3 Ductility

As far as possible, retaining walls should be designed in such way that adequate warning of danger (i.e. approaching an ultimate limit state) is given by visible signs. The design should guard against the occurrence of brittle failure, e.g. sudden collapse without conspicuous preliminary deformations. Retaining walls designed in accordance with this Geoguide should exhibit sufficient 'ductility' in approaching geotechnical limit states to give visible warning and no additional provision should be necessary.

2.3 SITE INVESTIGATION AND THE GEOLOGICAL MODEL

Guidance on the planning of site investigations and the conduct of desk study for retaining walls, and on the execution of ground investigations, is given in Geoguide 2 : Guide to Site Investigation (GCO, 1987).

A site investigation for retaining wall construction will normally proceed in stages, as follows : desk study; site reconnaissance; collection of field data for design, including ground investigation, topographic and hydrographic survey; and if necessary, follow-up investigations during construction.

The ground investigation should aim to establish the suitability of the site for the type of retaining wall being considered in the design, including the adequacy of the overall stability of the site with respect to the proposed works, the suitability of the foundation, and the suitability of materials on the site for fill. Preliminary design of the proposed works is often helpful in identifying parameters that need to be obtained from the ground investigation.

The investigation should identify specific groundwater and surface drainage conditions in the vicinity of the site, and their likely response to, for example, heavy rain or tidal action. In some cases, chemical testing of groundwater or surface water may be warranted (e.g. for salinity, sulphate ion content and pH). Further guidance on the chemical testing of

groundwater is given in Geoguide 2 (GCO, 1987).

To avoid confusion and misinterpretation in the communication of design information, it is essential that rocks and soils are adequately described. Guidance on this is given in Geoguide 3 : Guide to Rock and Soil Descriptions (GCO, 1988).

One of the end-products of a ground investigation is one or more geological models. It is important to state clearly in the geotechnical design report (see Section 4.6) the geological models considered in the design. In areas of insitu weathered rock in Hong Kong, a stratified model consisting of distinct layers may not be realistic.

2.4 SELECTION OF TYPE OF RETAINING WALL

The following factors should be considered in selecting a suitable type of retaining wall :

(a) location of the proposed retaining wall, its position relative to other structures and the amount of space available, including the necessity or otherwise to confine the support system within the site boundaries,

(b) height of the proposed retaining wall and the topography of the ground to be formed,

(c) ground and groundwater conditions,

(d) extent of ground movement acceptable during construction and in service, and the effect of movement of the retaining wall on nearby structures and services,

(e) availability of materials,

(f) time available for construction,

(g) appearance, and

(h) design life and maintenance.

A wide variety of different types of retaining walls is available (Figure 2). Their relative advantages and disadvantages are given in Table 1. Where several alternatives are suitable, an economic comparison should be made based on their initial construction and subsequent maintenance costs.

2.5 DURABILITY AND MAINTENANCE

Inadequate durability may result in disproportionate maintenance costs. It may also cause a retaining wall to reach either serviceability or ultimate limit states. Therefore, the

durability of the wall during its design life and the maintenance requirements must be considered in the selection and specification of materials for construction and in the evaluation of design parameters. The local climate and environment under working conditions should also be taken into account.

Some guidance on how concrete, steel and timber can be expected to deteriorate in different circumstances is given in BS 8004 (BSI, 1986a), which also indicates some of the steps which may be taken to prolong the life of these materials when used in foundations.

2.6 AESTHETICS

Retaining walls can be very dominant features on the urban and rural landscape. Careful design can make a considerable improvement to their appearance without leading to a significant increase in cost.

Apart from having to satisfy the functional requirements, a retaining wall should, as far as possible, be made to blend in with its surrounding environment and to be aesthetically pleasing. The aspects of a retaining wall that are important to its aesthetic impact are :

     (a) height of the wall and inclination of its front face,

     (b) curvature of the wall on plan (poor design can give the appearance of a 'kink' in the longitudinal elevation of the wall),

     (c) gradient and surface treatment of the adjacent ground,

     (d) surface textures of the front facing, and the expression and position of vertical and horizontal construction joints, and

     (e) the coping of the wall.

Aesthetic improvement can be achieved through the choice of a suitable structural form. Often, suitable types of vegetation can be incorporated to improve the appearance of the retaining wall. Guidance on the planting of grass, trees and shrubs in Hong Kong is given in the Geotechnical Manual for Slopes (GCO, 1984). Care should be taken to ensure that the proposed planting will not cause any damage to the wall in the long term. The appearance of a retaining wall may also be improved by providing features in the finished face or decorative facings. The advice of a landscape architect should be sought whenever appropriate.

2.7 PATENTS

Certain proprietary systems and components of retaining walls are covered by patents. Tender and contract documents should contain suitable clauses to ensure that no unforeseen liabilities are incurred by the employer with respect to the use of patented products.

3. CONSTRUCTION CONSIDERATIONS IN DESIGN

3.1 GENERAL

In the interest of safety and economy, designers of retaining walls should give careful consideration to methods of construction and materials to be used. This will help to avoid intrinsically risky designs and may result in significant economies, e.g., savings can often be achieved by incorporating part of the temporary works into the permanent structure.

3.2 SELECTION AND USE OF BACKFILL

3.2.1 General

The choice of materials for backfilling behind a retaining wall depends upon the materials available, the site conditions, the load to be placed on the backfill and the type of wall. The ideal backfill is a free-draining, durable material of high shear strength and stiffness, and which is free from any harmful matter. However, the final choice of materials often depends on the cost and availability of such materials balanced against the cost of providing more substantial support for lower-quality backfill materials.

Materials selected for use as backfill generally must not contain :

    (a) peat, vegetation, timber, organic or other degradable materials,

    (b) dangerous or toxic material or material susceptible to combustion,

    (c) metal, rubber, plastic or synthetic material,

    (d) material susceptible to significant volume change, e.g. marine mud, swelling clays and collapsible soils, or

    (e) soluble material.

Also, the backfill should not be chemically aggressive, e.g. the presence of excessive sulphate in soils can cause accelerated deterioration of concrete and steel (BSI, 1986a).

It should be demonstrated during the design stage or in the early stage of construction that the properties of the proposed source of backfill comply with the specification. The designer should specify the type, number and frequency of compliance tests, but there should be flexibility to allow the testing requirements to be increased during construction according to the heterogeneity of the material and the size of the retaining wall. Some minimum sampling frequencies are recommended in Table 2. Compliance testing of the proposed backfill should normally include determination of the particle size distribution, Atterberg limits, moisture content and compaction characteristics, as well as any electrical and chemical tests that are considered necessary. Shear strength testing may sometimes have to be

specified to check the design assumptions.

Determination of insitu densities and moisture contents should be specified for compaction control. Table 3 gives recommended minimum frequencies of compaction control tests for backfill placed behind retaining walls.

Where the contractor provides the backfill, the cost of a backfilled retaining wall can be minimized if the contractor is allowed the widest choice of materials, particularly where suitable materials can be found in the vicinity of the works site. Therefore, the specification of backfill should not be too restrictive.

3.2.2 Types of Backfill

The use of clay backfills is not recommended because of problems associated with swelling, shrinkage and consolidation of clay.

Backfills composed of uniform silts should not be used, as these materials are very difficult to compact.

For backfills composed of fine soils, adequate drainage should be provided to prevent the build-up of water pressure (see Section 8.4). Free-draining granular materials usually do not warrant the same amount of attention in this respect but may still require protection by properly designed filters.

For a retaining wall, the movement required to produce the active state in cohesive materials with a significant clay content is much greater than that for granular materials. This, together with the fact that the former generally have lower shear strengths, means that the amount of shear strength mobilised for any given wall movement is considerably lower for cohesive materials than for granular materials. The corresponding earth pressure on the 'active' side for a particular wall movement is therefore higher if cohesive soil is used for the backfill.

In Hong Kong, backfill for retaining walls customarily comprises selected soils derived from insitu weathering of granitic or volcanic (tuff and rhyolite) rocks (Figure 1). The sandy materials are suitable for backfill in most types of retaining walls, provided that they meet the requirements for soil backfill given in Table 4. Materials derived from weathered shales and mudstones and argillaceous facies within the volcanics and the meta-sediments of the Lok Ma Chau Formation may undergo considerable swelling by absorption of water and should not be used as retaining wall backfill.

Fill composed of crushed rock products is a very suitable material for use as backfill to retaining walls. Ideally, the rock fill should consist of pieces of hard, durable rock of which not more than 30% by weight is discoloured or shows other evidence of decomposition. It should be well-graded and have a maximum particle size of 200 mm (Table 4). Movement of retained soil due to migration of fines into the rock fill needs to be prevented, and this requires the provision of a properly designed filter between the soil and the rock fill (see Section 8.5).

Free-draining materials are required for filling gabion baskets and for the interior filling between units in a crib wall. Guidance on suitable infilling materials for crib walls and gabions is given in Chapter 9.

Particular requirements for backfill are necessary for reinforced fill structures. Detailed specifications for fill for such structures are given in Geospec 2 : Model Specification for Reinforced Fill Structures (GCO, 1989a).

3.3 FILTER AND DRAINAGE MATERIALS

3.3.1 Granular Filter and Drainage Materials

Free-draining granular materials such as clean crushed rock products are often used as filter and drainage materials. Such materials should be durable and free from clay, organic materials and other impurities. The grading of granular filter and drainage materials should conform to the filter design criteria given in Section 8.5. No-fines concrete should not be used as a filter material.

The particle size distributions of the retained insitu soil or backfill should be determined during the construction stage and checked against the relevant filter design criteria which should be stated on the drawings. Where the grading of the insitu soil is known at the design stage, the designer may, as an alternative, give the design grading envelopes for the filter and drainage materials on the drawings.

Compliance testing such as determination of grading and plasticity indices and compaction control testing such as insitu density and moisture content tests, together with acceptance criteria, should be specified. The frequency of such tests should take into account the likely variability of the various materials.

The level of compaction specified for filter and drainage materials should be compatible with the shear strength and permeability of these materials assumed in the design.

3.3.2 Geotextile Filter Materials

Where site conditions are non-aggressive, geotextile filters composed of resistant synthetic polymers are suitable as alternatives to granular filters in permanent works. Geotextile filters are relatively cheap and easy to use in construction. They are factory-produced materials and the control over geotextile properties is generally better than for a granular filter. Relevant criteria for geotextile filter design are given in Section 8.5.

It is recommended that only geotextiles produced by established international manufacturers should be used in permanent works. Polypropylene, polyester, polyamide and polyethylene, each with a molecular weight that exceeds 5 000, may be regarded as resistant synthetic polymers. Some general data on the chemical and thermal stability of synthetic polymer fibres is given in Table 5. In order to provide the geotextile fibres with resistance to deterioration caused by the effects of exposure and burial, the base polymer of the fibres should contain suitable additives such as UV stabilizers and anti-oxidants.

Where geotextile filters are intended to be used in permanent works, the site investigation should assess the potential aggressiveness of the site. As a minimum, the pH of the natural soil, the groundwater, and the fill to be placed close to the geotextile should be checked. Where aggressive conditions (e.g. pH outside the range of 5 to 10) are found to exist, advice should be sought from the geotextile manufacturer and, if necessary, polymer specialists. Particular care should also be taken where the site is in a potentially aggressive area, e.g. within a former industrial area or close to factories or waste disposal facilities.

Geotextiles in the ground may be damaged by vegetation roots and burrowing rodents. Although such damage is unlikely when a geotextile is buried at depth, it should be considered in the design when there is only a shallow soil cover.

To ensure that a geotextile filter will perform satisfactorily in service, it must have adequate hydraulic and mechanical properties, and the installation must be carried out in such a manner that the fabric is not damaged or excessively strained during construction. The level of supervision to be provided during installation should be appropriate to the risk category of the proposed works (GCO, 1984).

Geotextiles can deteriorate fairly rapidly under ultraviolet light. The effect of exposure is particularly important in Hong Kong where both solar radiation intensities and ambient temperatures are relatively high (Brand & Pang, 1991). Therefore, geotextile rolls delivered to a site should be maintained in roll form and covered (e.g. with opaque tubular sleeves) until they are to be installed. Geotextiles should be covered as soon as possible after installation. If it is foreseen that a geotextile cannot be protected within a short time after laying, allowance should be made in the design to ensure that a geotextile of adequate strength and ductility is specified for the period of exposure anticipated. Alternatively, the specification or drawings should state that geotextile strength and ductility requirements shall be met immediately prior to backfilling and that allowance shall be made in the provision of geotextiles for any deterioration in properties due to exposure. Tensile testing of geotextile samples taken immediately prior to covering may be carried out to ensure that the geotextile installed meets the specified quality. The acceptance criterion for the tensile testing should take into account the variability of the materials.

Drainage materials placed adjacent to a geotextile filter should preferably not contain particles larger than 75 mm. If drainage aggregates of a relatively uniform size are used, the maximum size of the particles should be limited to 20 mm in order to minimize the 'span' of the geotextile across the particles.

It is important to ensure that the geotextile filter is installed in good contact with the adjacent soil. Additional guidance on the installation of geotextiles can be found in GEO Publication No. 1/93 (GEO, 1993).

Where appropriate, earth pressure calculations should consider the effect of a reduced shear strength at interfaces between soil and geotextile. In cases where the interface strength is likely to be a critical factor in a design, direct shear tests should be carried out to determine the interface shear strength. Alternatively, the shearing resistance can be improved by benching the insitu ground on which the geotextile is to be placed.

3.4 CONSTRUCTION METHOD AND SEQUENCE

Selection of the method and sequence of construction is usually undertaken by the contractor. However, there may be instances where a particular method or sequence of operations is dictated by the design. In such cases, it is necessary to check that the method and consequence of operations are not intrinsically risky. In other instances, the designer may wish or be required to stipulate the methods of construction. In all cases, there should be suitable controls, e.g. by requiring the submission of method statements, so that an assessment can be made during construction (see Chapter 12).

The tolerances of the completed wall should be specified and these should take account of possible construction methods, as well as any movements which may occur during construction (e.g. due to settlement of the foundation). The processes of excavation, filling, pumping, etc. should be so arranged as to avoid endangering the stability and reducing the strength of any portion of the retaining wall, including any partially-completed sections. For example, in the case of a retaining wall designed to be supported at the top, backfilling should not proceed until the support is provided. Alternatively, the wall should be designed for the partially-completed loading condition. Limits of permissible loadings (e.g. surcharge) on the retaining wall should be clearly stated on the drawings.

3.5 TEMPORARY WORKS - GENERAL ASPECTS

3.5.1 Responsibility for Temporary Works Design

It is important that the responsibility for the design of temporary works is clearly identified. Where temporary works are to be designed by the contractor, the amount of control which should be exercised to ensure the safety of the temporary works needs to be carefully considered, particularly where the ground conditions are complex and the consequences of failure are high.

3.5.2 Standard of Temporary Works Design

Temporary works associated with the construction of a retaining wall should be designed to the standard which would be adopted if the temporary works were to be permanent. However, allowance may be made in the design of temporary works for the shorter design life, and for less severe loading conditions which are likely to occur during the life of the temporary works, e.g. groundwater conditions.

3.5.3 Influence of Temporary Works on Retaining Wall Design

The influence which the temporary works may have on the design of a retaining wall should be fully considered. For example, the slope angle selected for the temporary excavation to be formed for the construction of a gravity or reinforced concrete retaining wall could influence the earth pressure that will act on the wall, depending on the relative strength of the insitu soil and the backfill. If weak insitu material exists, it may be economical to replace this so that the most likely potential failure surfaces lie wholly within the stronger

compacted fill.

Where the temporary works are to form part of the permanent works, the earth and water pressures which act on the partially-completed retaining wall may be different from those assumed in the design for the permanent state. Therefore, the loadings for the temporary and permanent conditions should be individually assessed.

Excavation and foundation works are often necessary in the construction of retaining walls. These aspects are discussed in more detail in the next Section.

3.6 TEMPORARY EXCAVATION AND FOUNDATION WORKS

3.6.1 Excavation Works

Excavations required for the construction of a retaining wall should be designed to have adequate stability. Also, the works should not lead to unacceptable movements in nearby structures, services and land.

Guidance on the design of slopes which form a part of temporary works, together with advice on the excavation programme and methods appropriate for Hong Kong conditions, are given in Chapter 9 of the Geotechnical Manual for Slopes (GCO, 1984). Particular care should be taken where the groundwater level is high. The surface of temporary slopes should also be adequately protected against erosion. Where the temporary slope is higher than say 7.5 m, the slope surfaces should be benched. Any weak material at the insitu soil interface should be removed prior to backfilling.

Temporary excavations with lateral support should be properly designed and the sequence of construction carefully planned. Stability should be ensured for each construction stage and movements should be estimated. Limits on allowable movements should be clearly stated on the drawings. A review of methods for assessing movements around excavations is given in GCO Publication No. 1/90 (GCO, 1990).

Where lateral support to an excavation is to be removed, this should be done progressively as placement and compaction of fill material proceeds. Lateral supports should be removed in such a way as to ensure that the stability of the adjacent ground is maintained and to avoid disturbance to compacted backfill, filter and drainage materials. Where lateral support is to be left within the permanent structure, the effect of local transfer of stresses onto the retaining wall should be carefully evaluated.

3.6.2 Foundation Works

The extent of excavation for the foundation of a gravity or reinforced concrete retaining wall should be clearly shown on the drawings. The excavation should be protected from the effects of traffic, exposure to weather (rain and drying conditions) and the action of water (flow or ponding of surface water or the removal of groundwater). Foundations in clays or weak rocks, and foundations in soils derived from insitu weathered rocks (e.g. completely decomposed granites) below groundwater level, are particularly susceptible to

deterioration. In such cases, surface protection such as a layer of 'blinding' concrete should be placed immediately after excavation to the foundation level in order to protect the foundation and to keep the new construction clean. Blinding concrete is usually not placed for rock- or earth-filled gravity retaining walls such as reinforced fill structures. In these cases, backfilling of foundations that are susceptible to deterioration should take place as soon as possible. In all cases, any loose debris or slurry at the foundation level must be removed prior to placement of blinding concrete or backfilling.

3.6.3 Groundwater Control

In Hong Kong, there have been cases where inadequate groundwater control has led to failure of temporary slopes and weakening of the foundations of retaining walls, resulting in expensive remedial works. Therefore, adequate groundwater control measures should be implemented where appropriate.

Where excavations are required in permeable material below groundwater level, provisions should be made for adequate drainage control measures to prevent piping or a general weakening of the foundations due to the flow of water. This is particularly important where there are stringent requirements on the allowable movements of the retaining wall or on the differential movements between adjacent structures. Similar measures may also be needed for foundations in layered deposits of variable permeability. Excavations in compressible soils may result in heaving of the base of the excavation. This may give rise to problems of differential settlement where the retaining wall is required to span between excavated and unexcavated ground.

Groundwater levels may be controlled by dewatering. The dewatering method chosen should ensure the stability of the excavation and the safety of nearby structures. Reference may be made to Terzaghi & Peck (1967), NAVFAC (1982b), BS 8004 (BSI, 1986a) and Somerville (1986) for guidance on techniques of dewatering.

Groundwater control measures which result in a lowering of groundwater levels (e.g. pumping by means of dewatering wells) may cause settlement. The effects of dewatering settlement on structures, services and land in the vicinity of the site should be properly assessed.

3.7 PLACEMENT AND COMPACTION OF BACKFILL

All backfill materials placed behind retaining walls, including granular filter and drainage materials, should be compacted. The methods of placement and compaction of such materials are similar to those for any earthworks involving filling. General guidance on the compaction of fill materials is given in Chapter 9 of the Geotechnical Manual for Slopes (GCO, 1984).

In specifying the degree of compaction for backfill, granular filter and drainage materials, consideration should be given to the functions that the material is required to perform. The higher the degree of compaction, the higher are the shear strength and stiffness properties of the fill, and the lower is its permeability. It is recommended that the

degree of compaction specified should be at least 95% of the maximum dry density of the fill obtained using the appropriate test method given in Geospec 2 (GCO, 1989b). For level backfill, the top 1.5 m should be composed of fine soil compacted to at least 98% of its maximum dry density, in order to minimize infiltration through the surface. Such a compacted, low permeability layer should be provided irrespective of whether the surface is paved or not. Sloping backfill should also be compacted to the same standard for a vertical thickness of at least 3 m (GCO, 1984). The shear strength and other properties of the fill used in the design should be consistent with the degree of compaction specified.

Compaction can induce pressure on a retaining wall greater than that given by classical earth pressure theories. Therefore, unless a wall is very massive, heavy compaction equipment should not be used close to the back of the wall. The effect of compaction should be taken into account in the design, and the allowable loading due to compaction equipment should be clearly specified (see Section 6.8).

3.8 SERVICES

Where it seems possible that services will be placed in front of the retaining wall in the future, the effect of the trench excavation should be taken into account in the design. For example, it is prudent to design roadside retaining walls in built-up areas assuming the presence of a trench at least 1 m deep at the toe. Where appropriate, relevant authorities should be consulted on requirements to cater for future provisions. If a gravity or reinforced concrete retaining wall is used, the foundation should be set at a level such that the retaining wall will not be undermined during future trench excavation. If a cantilevered retaining wall is selected, the passive resistance should be reduced accordingly in the design.

3.9 USE OF PRESTRESSED GROUND ANCHORS

Prestressed ground anchors may be required solely for temporary support; they may form part of the permanent structure, or they may be designed to perform a dual function.

The use of permanent ground anchors in a project imposes a long-term commitment on the maintenance authority or owner, which usually involves appreciable recurrent cost. Therefore, permanent ground anchors should be used only when other methods of providing the required support are not practicable, and the agreement of the maintenance authority or owner should be obtained prior to their use.

Where ground anchors are to be used, an appraisal should be made of the suitability of the proposed anchor installation technique in relation to the ground conditions, the possible effects on adjoining structures and services, and the rights of adjoining owners whose structure or land may be affected. Corrosion protection and creep are two aspects which must be given special consideration. The standards set out in Geospec 1 : Model Specification for Prestressed Ground Anchors (GCO, 1989b) should be followed.

A review of the design methods for single-level and multi-level anchored or tied-back retaining walls is given in GCO Publication No. 1/90 (GCO, 1990).

4. VERIFICATION OF SAFETY AND SERVICEABILITY

4.1 GENERAL PRINCIPLES

The general approach which should be used to verify the safety and serviceability of a proposed retaining wall is described in this Chapter. Essentially, it consists of the following steps :

(a) Firstly, a list of required performance criteria should be compiled.

(b) Secondly, the relevant limit states at which the various performance criteria would be infringed should be determined.

(c) Thirdly, it must be demonstrated that the limit states are sufficiently unlikely to occur.

In compiling the list of required performance criteria and relevant limit states, the designer should consider the various design situations which can be foreseen during the construction and use of the proposed retaining wall. In the last step, the limit states must be shown to be sufficiently unlikely in each design situation. This is normally achieved by the use of calculation methods (see Section 4.3). In some cases, prescriptive measures (Section 4.4) or the observational method (Section 4.5) are applied. Sometimes it may be appropriate to use these methods in combination. Guidance is given in Chapters 9 to 11 on the methods to be used for checking limit states for the design of various types of retaining walls.

The guidance given in this Geoguide cannot possibly cover all situations. The designer should select performance criteria, limit states, loadings and design parameters appropriate to particular conditions.

4.2 DESIGN SITUATIONS

Situations considered in design should be sufficiently severe and varied so as to cover all reasonable conditions that may occur during construction and throughout the design life of the proposed retaining wall.

The following information is required in order to specify design situations :

(a) the nature of the environment within which the retaining wall is to be constructed over its design life, for example :

(i) the geological profile and variations in soil and rock properties,

(ii) changes in surface water and groundwater levels, and changes in pore water pressures,

(iii) effects of chemical attack and electrolytic corrosion,

(iv) excavation in front of and surcharge behind the retaining wall,

(v) effects of planned development in the vicinity,

(vi) effects of earthquakes,

(b) aspects of construction that are pertinent to the design, e.g. compaction behind the retaining wall, and

(c) users' functional requirements.

Each design situation should be clearly defined, together with the relevant types of loading.

Conceivable accidents should be considered in design and the retaining wall should be designed in such a way that it will not be damaged disproportionately or lose its stability under such accidental conditions. However, the effects of accidental loadings, e.g. pressures due to bursting of water mains, are often difficult to quantify, and consideration of combinations of accidental loadings usually results in costly designs. Hence, accidental loadings are usually best dealt with by prescriptive measures, e.g. re-routing or ducting of water services.

4.3 CALCULATION METHODS

4.3.1 Available Approaches to Design Calculations

Limit states relevant to a design situation are usually checked by calculation methods. Two approaches, viz. the direct approach and the indirect approach, may be followed in design calculations. Chapters 9 to 11 indicate where each of the following approaches is applicable.

In the direct approach, each limit state is checked by direct calculations which must show that the limit state would not be reached. Limit states involving the formation of failure mechanisms in the ground are often checked using this approach. For limit states involving considerations of deformations, the deformations may either be calculated or otherwise assessed if this approach is used.

In some cases, especially for routine design, rigorous analysis may not be justified and a simple indirect approach may be more appropriate. The indirect approach is particularly useful for serviceability checks, e.g. where the deformation criteria may be deemed to be satisfied by limiting the eccentricity of the reaction at wall base. An example of this approach is the application of the middle-third rule in the design of gravity retaining walls (see Section 9.2.4).

4.3.2 Design Calculation Models

A design calculation model generally consists of two elements :

(a) a method of analysis, often based on a theoretical approach including simplifications, and

(b) if necessary, a modification to the results of the analysis to ensure that the results of the calculation are either accurate or err on the side of safety.

Where an unconventional theory is to be used, or where a conventional theory is to be applied in an untried situation, the uncertainty in the applicability of the design calculation model should be assessed. Whenever possible, the method of analysis should be calibrated against observations of field performance of previous similar designs, load tests or more reliable analysis.

Design calculation models are given in Chapters 9 to 11 for the checking of limit states for various types of retaining walls.

4.3.3 Loadings, Geometric and Geotechnical Parameters

(1) *General.* Some general guidance is given in the following Sections. Specific guidance on the selection of values of geotechnical parameters for design, and on the evaluation of various types of loadings, viz. earth pressures, surcharge (and seismic) and water loads, is given in Chapters 5, 6, 7 and 8 respectively. Where appropriate, parametric studies of the effect of a variable loading or parameter should be carried out.

(2) *Loadings and Load Combinations.* For each design situation, concentrated or distributed loads which may result in forces acting on the retaining wall should be evaluated. The common types of direct loads are :

(a) weights of soil, rock and water,
(b) earth pressures,
(c) free water and groundwater pressures,
(d) seepage forces, and
(e) surcharge and seismic loads.

Indirect loadings that need to be tolerated by the retaining wall should also be considered and, where appropriate, their magnitudes should be determined. These include loadings arising from :

(a) geological factors, e.g. creep of slope masses, solution of soluble rock and collapse of sinkholes in cavernous rock,

(b) conceivable man-made activities, e.g. dewatering, excavation, tunnelling and blasting, and

(c) temperature effects.

No guidance is given on how to evaluate the magnitudes of these indirect loadings and major dynamic loads (e.g. from machine foundations), nor their effects on retaining walls. The designer should refer to specialist literature on these subjects.

In selecting values of loadings for design, the duration of the loadings should be considered with reference to possible changes in soil properties with time, especially the drainage properties and compressibility of fine-grained soils. It may be helpful to distinguish between very short-term transient loads (e.g. wind loads) during which soils are likely to display enhanced strength and stiffness, short-term loads (e.g. construction loads) during which soil drainage is likely to be negligible, and long-term loads.

It is sometimes necessary to check several load combinations in a design situation and to design for the most critical condition.

(3) *Geometric Parameters*. The most important geometric parameters in geotechnical design are usually the level and slope of the retained ground surface, the geometric characteristics of the geological model, and the levels of excavations. While small variations in such parameters are generally covered by the factors of safety applied, gross changes in levels should be allowed for in the design directly. For limit states with severe consequences, geometric parameters should represent the most unfavourable values which could occur in practice. In some cases, it may be economical to design for a range of anticipated levels, e.g. founding levels in the design of rock-socketed caissons, and to incorporate in the contract permissible variations in design subject to the actual conditions encountered during construction.

(4) *Geotechnical Parameters*. The parameters relevant to retaining wall design include unit weight, shear strength, permeability, insitu stresses and deformation parameters of soil and rock. Detailed guidance on the selection of such parameters for design is given in Chapter 5.

4.3.4 Factors of Safety

(1) *General*. The reliability of a geotechnical design depends not only on the factor of safety, but also on the methods of analysis, the calculation models, the way in which factors of safety are defined, the reliability of the geological model, the assessment of appropriate geotechnical parameters and the quality achieved in construction. Therefore, the minimum factors of safety recommended in this Geoguide should not be used out of context. Where the designer wishes to adopt different minimum factors of safety, the following factors should be considered :

(a) the consequences of the limit states being reached,

(b) possible slight increases in load beyond those covered by the design,

(c) reliability of the geological model,

(d) inaccurate assessments of loading and unforeseen stress redistribution within the retaining wall,

(e) the uncertainties in the method of analysis and the applicability of the design calculation model,

(f) possible differences between the strengths of the materials in the actual ground and structure and the strengths derived from test specimens,

(g) the level of supervision to be provided and the likely quality of workmanship, e.g. variations in dimensional accuracy, target compaction, etc., and

(h) the design life of the wall.

It should be noted that factors of safety cannot cover gross errors and non-compliance with specifications.

(2) *Partial Safety Factors.* The partial safety factor approach, allowing different safety factors for loading and material properties, commensurate with different reliabilities and consequences, is adopted in this Geoguide. Factored values of loading and geotechnical parameters, as defined below, should be used directly in the design calculations :

$$F_f = F \cdot \gamma_f \quad \ldots \ldots \ldots \ldots \ldots \ldots \quad (4.1)$$

$$X_f = X / \gamma_m \quad \ldots \ldots \ldots \ldots \ldots \ldots \quad (4.2)$$

where F_f, X_f = factored values of loading F and material parameter X respectively

γ_f, γ_m = partial load factor and partial material factor respectively

The recommended minimum partial factors of safety for ultimate limit state design of retaining walls are given in Tables 6 and 7. The use of the partial safety factor approach for calculating 'ultimate' values of earth pressures and sliding resistance is illustrated below.

The first step involves applying the partial material factor γ_m to the selected values (see Section 5.3 for guidance on its determination) of c' and tan ϕ' to obtain the factored shear strength parameters c_f' and tan ϕ_f' :

$$c_f' = c' / \gamma_m \quad \ldots \ldots \ldots \ldots \ldots \ldots \quad (4.3)$$

$$\tan \phi_f' = \tan \phi' / \gamma_m \quad \ldots \ldots \ldots \ldots \ldots \quad (4.4)$$

The angle of skin friction δ_s and wall friction δ are then assessed using ϕ_f' (see Section 5.11). Finally, the ultimate earth pressures should be obtained using the factored c_f' and ϕ_f' values and the δ_s or δ values assessed earlier.

The ultimate sliding resistance of a gravity or reinforced concrete retaining wall is assessed using the factored angle of base shearing resistance δ_{bf}. δ_{bf} should be calculated

from $\tan^{-1}[(\tan \delta_b)/\gamma_m]$ using the selected δ_b value, which should be based on the unfactored selected value of the relevant soil or interface shear strength parameters (see Section 5.12).

For the design of slopes, reference should be made to the Geotechnical Manual for Slopes (GCO, 1984), which gives the relevant recommended minimum factors of safety.

For serviceability limit state checks, all values of γ_f and γ_m should be set to unity. An exception is that the value of γ_f should be set to zero for those surcharge loads which produce a favourable effect, e.g. surcharge in the area between the wall stem of a R.C. L-shaped retaining wall and its virtual back.

Appropriate partial factors should be applied to the loadings and selected values of material parameters used in the various calculation models given in this Geoguide, depending on whether the calculation is to check against an ultimate or a serviceability limit state.

4.3.5 Structural Design

Structural design of retaining walls should be carried out in accordance with the requirements of relevant structural codes and standards.

In structural design, the loadings on retaining walls due to earth and water pressures, and the effects of surcharge and seismic load, should be evaluated in accordance with the principles given in this Geoguide. Generally, unfactored parameters should be used in the calculations, but this would depend on the requirements of the structural code adopted. In the commonly-used limit state structural design codes, the ultimate limit state loads are evaluated based on unfactored parameters. These are then multiplied by appropriate partial load factors in the checking of limit states.

4.4 PRESCRIPTIVE MEASURES

For certain limit states, e.g. failure due to chemical attack, calculation models are either not available or unnecessary. Instead, the limit state can be avoided by adopting 'prescriptive measures'. These may involve conventional or generally conservative details in the design, and attention to specification and control of materials, workmanship, protection and maintenance procedures.

4.5 THE OBSERVATIONAL METHOD

Because prediction of geotechnical behaviour is often difficult, it is sometimes appropriate to use the approach known as "the observational method" (Peck, 1969), in which several design options are provided and arrangements are made for design decisions during construction. When this approach is used, it is essential that the following requirements are met before the commencement of construction :

> (a) The limits of behaviour which are acceptable must be established.

(b) The range of possible behaviour should be assessed, and it should be shown that the actual behaviour is likely to lie within the acceptable limits.

(c) A monitoring plan which will reveal whether the actual behaviour lies within the acceptable limits must be arranged. The monitoring should be able to confirm this at a sufficiently early stage, and at sufficiently close intervals, to allow contingency actions to be undertaken promptly and successfully. The response time of the instruments needs to be sufficiently rapid in relation to the possible evolution of the system. Warning levels must be given and adequate staff suitably experienced to undertake monitoring should be provided.

(d) A plan of contingency actions to be adopted if the monitoring reveals behaviour outside the acceptable limits must be prepared.

During construction, the monitoring should be carried out as planned, and additional or replacement monitoring undertaken if necessary. The results of the monitoring must be assessed at appropriate stages, and the planned contingency actions should be put into operation if necessary.

The observational method is often used, for example, in the design of walls retaining the sides of embankments on soft ground. The parameters which are most frequently monitored are ground movements and pore water pressures. Contingency actions may include regrading of the surface of the natural ground or fill, installation of structural support or installation of drainage measures. In some cases, the action necessary may simply be a modification of the time scale for continued construction.

4.6   GEOTECHNICAL DESIGN REPORT

The assumptions, data, calculations and results of the verification of safety and serviceability of a proposed retaining wall should be properly recorded in a geotechnical design report. The factual information should be separated from the interpreted information. Items which require checking, monitoring or testing during construction, or which require maintenance, need to be clearly identified in the report. When the required checks have been carried out, it is recommended that they should be recorded in an addendum to the report.

The report will normally include the following items, with cross-references to other documents which contain more details, e.g. the ground investigation report, see Geoguide 2 (GCO, 1987) :

(a) a critical examination and description of the site including topography, geology, groundwater and surface water conditions, site history, public utilities, drains and sewers and other services, and local geotechnical records,

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(b) a critical interpretation of the results of ground investigation,

(c) a critical interpretation of the results of site monitoring of groundwater conditions,

(d) a schedule of the geotechnical design assumptions, including the selected values of material parameters, loading and geometric parameters for design,

(e) a discussion of anticipated geotechnical problems and statements on the contingency actions to be taken,

(f) statements on geotechnical requirements for the design and construction of the retaining wall including testing, inspection and maintenance requirements, and

(g) stability analyses of the site and design calculations for the retaining wall.

5. GEOTECHNICAL PARAMETERS

5.1 GENERAL

This Chapter gives guidance on the evaluation of geotechnical parameters relevant to the design of retaining walls.

5.2 GEOTECHNICAL INVESTIGATION

5.2.1 Content of Site Investigation

Guidance on the content of site investigation for the design of retaining walls is given in Section 2.3. A common starting point in site investigation is the collection of information about ground conditions and properties determined in previous investigations. This is usually followed by ground investigations using techniques such as borehole drilling, trial-pitting and insitu testing.

For walls which are designed with retained heights of less than about 3 m, it is usually sufficient to select parameters for the backfill and the insitu ground on the basis of results of previous tests on similar materials. The materials should be carefully examined and described, particularly those at the proposed foundation level. Classification and other tests required to characterise the materials should be carried out both in design and during construction. This is to ensure that the assumed design parameters are consistent with the material types encountered. For walls designed with retained heights of more than about 6 m, values of geotechnical parameters should be determined from laboratory tests on samples selected by the designer, or from appropriate field tests, in addition to detailed description of the materials. For intermediate wall heights, the need for laboratory tests depends on the sensitivity of the wall dimensions to the design parameters. A preliminary design of the proposed wall using assumed parameters is often useful in identifying those parameters that need to be obtained from the ground investigation.

Laboratory tests are usually carried out to obtain values of geotechnical parameters. Good quality representative samples should be selected for such tests. Also, the test conditions should be specified in such a way as to model as closely as possible the conditions that are likely to exist at the limit state being considered. Field tests are very often carried out to obtain information to supplement results of laboratory tests. In situations where mass-scale geological features govern the behaviour of the ground, field tests can yield more reliable parameters than laboratory tests. For example, the permeability of the ground can often be determined more reliably by field permeability tests than by laboratory tests performed on small 'undisturbed' samples.

Testing schedules should include a sufficient number of tests to provide results representative of the variation of material properties relevant to the design. Where established empirical relationships exist between results of laboratory or field tests and geotechnical parameters, these may be used to take advantage of the valuable database available. However, the applicability of such relationships to the particular field conditions must be carefully scrutinized.

The Geotechnical Information Unit of the Civil Engineering Library, which is operated by the Geotechnical Engineering Office, holds records of previous ground investigations and laboratory tests carried out for building and civil engineering projects in Hong Kong. Such records may allow rapid assessment of design parameters which are suitable for preliminary designs.

5.2.2 Evaluation of Geotechnical Parameters

Only a discussion of general principles is given below as it is not possible to provide a set of rules for the evaluation of relevant parameters from geotechnical data.

Designers should always look critically at material descriptions given in the ground investigation report. Where there is doubt, the designer should examine the samples or consult the people who supervised the ground investigation and logged the samples.

Techniques based on statistical methods (e.g. the Least Squares Method) are often applied to the test results to obtain 'best-fit' parameters (e.g. see Lumb, 1970). Such techniques should not be used uncritically. In the grouping of test results for evaluation, account should be taken of all available information, especially the geological conditions and the descriptions of the materials. Any anomalous test result should be considered carefully in order to determine whether it is misleading or represents a real phenomenon that should be accounted for in the design.

'Best-estimate' parameters are those which are representative of the properties of the materials in the field. They should be determined from a careful analysis of all relevant information. Statistical best-fit parameters should not be taken directly as best estimates without due consideration being given to the factors which can affect the representativeness of the test results.

Sampling bias can affect the representativeness of the test results. For example, sampling of very weak material is difficult and test specimens that can be successfully prepared tend to be the stronger portions of the material. The same problem of sampling and specimen preparation occurs for insitu soil containing a significant proportion of gravel- or cobble-size particles, but with the opposite effect. Specimens containing a few or no coarse particles are often tested, giving strength values lower than the average shear strength of the insitu soil. The problem of sampling bias is not limited to insitu soil. The shear strength of compacted fill is sometimes obtained by laboratory tests on specimens compacted to the minimum degree of compaction specified by the designer. Such tests are likely to yield lower bound values of shear strength because over-compaction of fill usually occurs on site.

Whenever possible, the derived parameters should be compared with relevant published data, and local and general experience. Published correlations between parameters should be examined and deviations from established relationships should be critically evaluated. Where possible, the derived parameters (e.g. soil stiffness) should also be checked against values which have been back-analysed from measurements taken from comparable full-scale construction in similar ground conditions (e.g. Humpheson et al, 1986).

Commentary on the evaluation of test results to obtain 'best-estimate' geotechnical

parameters and an explanation of the basis of all assumed parameters should be clearly presented in the geotechnical design report (see Section 4.6). Relevant references should be included. If in the opinion of the designer the data are defective, inaccurate or insufficient, this should be pointed out in the geotechnical design report. Proposals for any additional investigation and testing required prior to the start of construction should also be given.

5.3 DETERMINATION OF SELECTED VALUES OF DESIGN PARAMETERS

The determination of selected values of geotechnical parameters for design should be based on a careful assessment of the range of values of each parameter which might govern the performance of the retaining wall during its design life, with account taken of the conditions representative of the ground and the nature of the environment. Many geotechnical parameters are not true constants (e.g. soil shear strength parameters). It may be necessary to adopt different selected values for a parameter in different limit states and design situations. For example, different design strengths may need to be used when assessing potential shear failure in a soil containing relict joints, depending on whether the shear surface is free to follow the joints or is constrained to intersect intact material. Strain levels and compatibility should be considered in the assessment of strengths in materials through which a presumed failure surface passes. Also, different (upper or lower) selected values are sometimes used for the calculation of loading and of resistance.

Selected values should usually be based on 'best-estimate' parameters (see Section 5.2.2), taking into account the following factors :

(a) the quality of the ground investigation, viz. quality of retrieval, transportation and storage of samples, sample disturbance and quality of testing, in relation to the level of technical supervision provided,

(b) the appropriateness of the test methods in relation to the likely field conditions, viz. geology, presence of strong or weak inclusions and specimen size relative to the mass characteristics of the ground,

(c) the adequacy of test data, in relation to the inherent variability of the materials encountered, the possible existence of weaknesses, sample disturbance and sampling bias,

(d) the appropriateness of the test conditions in relation to scale, viz. size of ground features relative to size of the retaining wall and in relation to the limit states being considered, viz. failure mechanism and mobilized strain levels,

(e) the effects of construction activities on the material properties, especially for insitu soils (e.g. excavation, preparation of the foundation, upward hydraulic gradients

and dewatering),

(f) the influence of workmanship for compacted fill materials, and

(g) time effects (e.g. consolidation).

If the design is sensitive to certain parameters (e.g. the ultimate bearing capacity of a spread foundation is sensitive to ϕ'), the designer should take special care in assessing the reliability of the selected values of those parameters. Reasonably conservative selected values should be adopted where the level of confidence in any of the above factors is low. In this respect, sensitivity checks of design parameters are useful.

At the design stage, the source of the backfill is usually not known. Assumed values of backfill parameters (e.g. unit weight and shear strength parameters) should not be overly optimistic, in order not to limit the choice of materials unnecessarily. Account should be taken of the specification of the backfill and the compliance testing requirements adopted.

In the following Sections, typical ranges of values of various parameters are given in the form of Tables. These are for general guidance only. Appropriate values of parameters within the upper and lower limits given may be used in preliminary designs. For detailed design, the basis for selecting the value of each design parameter should be justified in the geotechnical design report.

5.4 UNIT WEIGHT

Typical ranges of values of unit weight for selected Hong Kong soils are given in Table 8. The unit weight used for a submerged soil should correspond to the saturated state.

For backfill, the unit weight may be determined from standard laboratory compaction tests on representative samples or estimated from records of field compaction tests on similar fills. The selected values should correspond to the compaction and moisture conditions which will apply in the field. Laboratory compaction tests are indispensable when fill materials from new sources are used. This is important because the new fill materials may have unit weights much higher than those normally encountered, e.g. soils containing metallic minerals such as the magnetite- or haematite-bearing soil found at Ma On Shan. There has also been a case history of collapse involving a wall which retained soil-like industrial wastes with unit weight much higher than that of soil.

For insitu soils, the unit weight may be determined from laboratory measurements made on 'undisturbed samples' taken for shear testing. Where the soil is granular in nature or is weakly cohesive, disturbance in sampling renders laboratory determination of unit weight unreliable. In such cases, conservatively assumed values are normally adequate for design. However, where accurate values of unit weight are needed, advanced sampling techniques using special drilling fluid may be used.

5.5 SOIL SHEAR STRENGTH PARAMETERS

5.5.1 General

Table 8 gives typical ranges of values of shear strength parameters for soils commonly found in the granitic and volcanic (tuff and rhyolite) areas of Hong Kong. Soils derived from the meta-sedimentary and argillaceous rocks may have markedly different typical values of shear strength parameters (see Section 1.2.4).

5.5.2 Design Soil Strength Model and Parameters

The shear strength of a soil may be represented graphically on a Mohr diagram. In general, the shear strength (failure) envelope of a soil is curved. However, for simplicity of analysis, it is conventional to use a c' - ϕ' soil strength model (i.e. a linear strength envelope) in design. In this model, the shear strength of the soil, τ_f, is expressed in terms of effective stress by the parameters c' and ϕ' using the following equation :

$$\tau_f = c' + \sigma_{nf}'\tan \phi' \quad\quad\quad\quad\quad\quad (5.1)$$

where c', ϕ' = apparent cohesion and angle of shearing
resistance of the soil in terms of effective stress

σ_{nf}' = effective normal stress at failure

It should be noted that the values of c' and ϕ' are not intrinsic soil properties, but are merely coefficients in the simplified design model. As such, they should only be assumed constant within the range of stresses for which they have been evaluated. Figure 3 shows two linear strength envelopes for a soil for two stress ranges. It should be noted that the linear envelopes are not tangents to the curved failure envelope. Also, it is worth noting that c' is lower and ϕ' is higher for stress range I than for the wider stress range II.

The shear strength parameters of a soil are often interpreted from laboratory test results. It should be noted that where the Least Squares Method is used to obtain values of c' and ϕ', problems can arise where there are too few test results and the soil is very variable. Figure 4 illustrates some of the pitfalls of using the Least Squares Method. When interpreting laboratory shear test results, account should be taken of the amount of data available within the relevant stress range, extrapolation required in the low stress region, possibility of curvature of the shear strength envelope and the likely variability of the soil. The value of c' in particular should be selected with care. A suggested procedure for determining the values of c' and ϕ' of a soil from laboratory test results is given in Figure 5. Considerable engineering judgement is required when selecting shear strength parameters for design, and account should be taken of the various factors given in Section 5.3.

The 'critical state' strength of a soil (ϕ_{cv}', see Section 5.5.3) can serve as a guide in the selection of the soil's shear strength parameters. ϕ_{cv}' delineates the lower limit of shear strength in a τ_f - σ_{nf}' (or p_f' - q_f) plot (see Figure 3), where $p_f' = (\sigma_{1f}' + \sigma_{3f}')/2$, $q_f = (\sigma_{1f}' - \sigma_{3f}')/2$, and σ_{1f}', σ_{3f}' are effective major and minor principal stresses at failure respectively. As a guide, the lower bound value of ϕ_{cv}' is about 34° and 30° for Hong Kong granitic and volcanic (tuff and rhyolite) soils respectively.

'Undrained' conditions are sometimes assumed in design, e.g. in checking the bearing capacity of foundations on cohesive soils of low permeability. In such cases, total stress analyses are carried out, using a $\phi = 0$ soil strength model which is based on the parameter s_u, the undrained shear strength of the soil. Guidance on the selection of s_u values for cohesive soils is given in Section 5.5.7.

5.5.3 Compacted Fill

Soil undergoes significant remoulding and degradation in the process of excavation, deposition, spreading and compaction. The shear strength of a remoulded soil consists of two components, one depending on the intrinsic frictional properties of the soil (viz. the 'critical state' or 'constant volume' angle of shearing resistance ϕ_{cv}'), and the other on density, stress level and stress history (Rowe, 1962; Schofield & Wroth, 1968). The former component can be considered constant whilst the latter will vary depending on whether the soil dilates or contracts upon shearing. For backfills which satisfy the grading and plasticity requirements of Table 4, it is generally sufficient to adopt a $c' = 0$ soil strength model for design purposes. Such a model gives a conservative estimate of the shear strength of the backfill and is analytically simple to apply in design.

The shear strength parameters of a fill material can be determined from laboratory tests on representative specimens where degradation is replicated and remoulding undertaken to the density specified by the designer. The minimum dry density specified by the designer for compaction purposes is normally adopted. The tests should be conducted at appropriate stress levels, preferably covering a range of stresses up to a level which exceeds the maximum theoretical normal stress acting on the critical failure surface. The value of the maximum normal stress should therefore be estimated in each limit state check, to allow proper specification of stress levels for shear strength testing.

Both triaxial compression and direct shear tests can be used to determine the shear strength parameters of a fill material. Guidance on methods for carrying out such tests and on the interpretation of test results can be found in the Geotechnical Manual for Slopes (GCO, 1984), Head (1986) and BS 1377 : Part 8 (BSI, 1990a). A detailed description of a direct shear test procedure suitable for a range of fill materials is given in Geospec 2 : Model Specification for Reinforced Fill Structures (GCO, 1989a). It should be noted that the test conditions, including specimen size, method and time of soaking or saturation, as well as the stress path, rate of shearing and drainage conditions, can affect the test results.

For crushed rock fill with 50% or more of the component particles greater than about 20 mm, it is not meaningful to determine shear strength by means of laboratory shear tests on small specimens. The empirical method given by Barton & Kjaernsli (1981) is suitable for obtaining an estimate of the shear strength of such materials. When applying the empirical method, consideration should be given to the nature of the constituent rock (e.g. strength, size and angularity) and the porosity of the rock fill that can be achieved in the field. The porosity of the rock fill will depend on the method of placement and compaction. Therefore, when selecting shear strength parameters for design, the method of compaction control specified should be taken into account.

5.5.4   Soils Derived from Insitu Rock Weathering

These soils are of two types, viz. saprolites, which retain evidence of the original rock texture, fabric and structure, and residual soils, in which the original rock features have been destroyed by weathering. For residual soils in Hong Kong, a c' = 0 soil strength model is often used for design purposes.

For saprolites, the 'material' shear strength is a function of the nature of the parent rock, its grain size distribution, mineralogy and microfabric, the state of weathering, the degree of saturation, and changes induced by changes in moisture content. The state of the soil (viz. void ratio and effective confining pressure) is also very important. In order to assess the 'mass' shear strength, consideration should be given to the material strength as well as to the orientation, spacing, persistence and unevenness of relict structures and discontinuities present in the soil mass. The nature of any infilling or coating on the discontinuities and the proportion, distribution and nature of any less weathered rock (e.g. corestones or rock bands) within the relatively more weathered matrix should also be considered.

Features such as corestones, even if identified during ground investigation, are not easy to quantify in terms of their contribution to mass strength. Conservative practice is to use the matrix strength only in design. A recent study by the GEO has provided data which allow quantification of the contribution of corestones to mass shear strength (Brand, 1992). However, a rigorous investigation is required to determine the proportion, distribution and nature of the coarse fragments in the ground.

The presence of relict structures and discontinuities in saprolites can result in failure by mechanisms different from those assumed in conventional soil mechanics theories. Therefore, as a general rule it is prudent to inspect exposures. For walls which can be affected by the existence of persistent adversely-oriented relict structures and discontinuities, such as bored pile (or caisson) walls, detailed geological mapping should be carried out. If such features exist, then an appropriate calculation model and relevant shear strength parameters should be used in design. It is also recommended that exposures should be mapped during construction and the design should be reviewed should adverse features be revealed at that stage.

For the more sandy saprolites derived from weathering of granite, tuff and rhyolite in Hong Kong, which are generally relatively permeable, only an effective stress analysis using a c' - ϕ' soil strength model together with postulated water pressures needs to be carried out. However, consideration should be given to performing a total stress analysis using undrained shear strength for the finer-grained residual soils and saprolites and for cases where a layer of cohesive soil of low permeability (e.g. a decomposed dolerite dyke or a kaolin vein) exists within the saprolite mass.

Triaxial compression tests are normally used to determine the shear strength parameters of saprolites. The direct shear test may also be used, for which a specimen size of 100 mm square by 44 mm thick has been found to give satisfactory results for completely decomposed granites with grains up to 10 mm in size (Cheung et al, 1988).

Using appropriate techniques, "Class 1" quality block samples for shear strength

testing can be obtained from trial pits (GCO, 1987). Trial pits are particularly useful for assessing the mass characteristics of the ground, as the fabric of the saprolite as well as any relevant features in the ground can be examined at the time of the pit excavation. It is therefore recommended that trial-pitting is carried out wherever possible, especially for the purpose of foundation design.

Saprolitic soils are easily disturbed by sampling and specimen preparation procedures. The effects of any disturbance should be taken into account when analysing the test results. An estimate of the lower bound strength of a test specimen may be obtained by examining the portion of the stress-strain data in a triaxial compression test which corresponds to the critical state. Theoretically, the approach of such a state is indicated by the effective stress ratio reaching a constant value in an undrained test, or the dilation rate approaching zero in a drained test. However, it has been found in practice that the critical state is often not achieved even at high strains for the saprolitic soils commonly encountered in Hong Kong. For such soils, an estimate of the critical state angle of shearing resistance may be more conveniently obtained by carrying out shear tests under high confining pressures on specimens which have been remoulded to a loose state. Care is required to interpret the test results, which may be influenced by the effects of any relict structures or fabric not destroyed by the remoulding process and by the effects of grain crushing. Knowledge of the critical state strength, if considered together with the results of laboratory shear tests on 'undisturbed' intact samples, can assist in the selection of soil shear strength parameters for design (Figure 5).

5.5.5   Colluvium

Colluvium, the product of the downslope movement of earth materials essentially under the action of gravity, can be very heterogeneous even within a small area. In characterizing a colluvial deposit, consideration should be given to the age of the deposit, the nature of the matrix material (including soil type and degree of cementation) and the proportion, distribution and nature of the coarse fragments. Where the coarse fragments in colluvium can be characterised with confidence, the mass strength of the colluvium can be assessed according to Brand (1992). Otherwise, the shear strength of the matrix only should be used in a design.

A large number of tests are required to evaluate the shear strength parameters of a variable colluvial deposit, for which a c' - ϕ' effective stress soil strength model is usually appropriate for design purposes.

5.5.6   Granular Sedimentary Soils

For cohesionless granular soils which are of relatively high permeability (e.g. gravels, sands and silty sands such as those that may be found in the beach deposits of coastal areas, river channel deposits and in reclamations formed of marine sand), a c' = 0 effective stress soil strength model is often used.

It is not usually possible to retrieve high-quality 'undisturbed' samples of granular soils from the ground for laboratory shear strength testing. Therefore, to determine shear

strengths, laboratory tests are sometimes carried out on remoulded specimens that have been prepared to a range of densities and stress levels. However, this approach is very time-consuming. In practice two alternatives are commonly used to obtain approximate values of strength parameters. These are usually adequate for design purposes.

The first approach involves an estimation of the relative density of the soil by means of insitu tests, e.g. using established correlations between standard penetration test (SPT) results and relative density (Figure 6). The angle of shearing resistance is then determined from an empirical correlation such as that given by Bolton (1986). The second approach is to use a direct empirical relationship between insitu tests and angle of shearing resistance, such as that given by Schmertmann (1975). Figures 7 and 8 illustrate the application of these approaches. It should be noted that the value of ϕ' derived using such approaches may be in error by up to $\pm 5°$, due to scatter in the original database, inaccuracies in the standard penetration test and errors in the estimation of relative density.

5.5.7   Cohesive Sedimentary Soils

For cohesive sedimentary soils of low permeability (e.g. clays), it is conventional to express the shear strength of the soil in terms of total stress by the undrained shear strength, s_u. Laboratory undrained triaxial compression tests can be used to determine s_u values. These should be carried out on samples which have been consolidated to the correct confining stresses. Insitu tests such as the cone penetration test and the vane test can also provide an estimate of shear strength. The use of a vane borer is preferable to the borehole vane for soft clays (GCO, 1987). Interpretation of results of cone penetration and insitu vane tests is discussed by Simpson et al (1979).

In evaluating results of shear tests on clays and in selecting s_u values for design, the influence of sample disturbance, soil fabric, stress state, anisotropy, shear rate, degree of saturation, the effects of large strain, and time and chemical effects should be considered. Brand & Brenner (1981) provide detailed discussions on these factors in relation to soft clays. Useful advice on the measurement and selection of shear strength parameters for the design of retaining walls embedded in stiff clays is given by Padfield & Mair (1984).

5.6   SOIL DEFORMATION PARAMETERS

5.6.1   General

The following factors should be considered when evaluating soil deformation parameters :

(a)   the effect of construction activities on bonding in saprolitic soils : the soil may yield before application of the design loading,

(b)   the effect of preloading, either due to removal of overburden or lowering of groundwater level : the stiffness of a soil on reloading is several times higher than that on

virgin loading,

(c) the rate of application of loading, particularly in relation to drainage and consolidation of the soil mass (see Section 5.7.2),

(d) the magnitude of the average shear stress induced in the soil mass in relation to the soil shear strength, i.e. the extent of plastic behaviour,

(e) the magnitude of the average strain induced in the soil mass, i.e. the applicability of small-strain stiffness,

(f) the size of the specimen being tested in relation to its natural variability and any fabric and structure present, and the effects of the scale of such features in relation to the size of the retaining wall, and

(g) the effect of non-linear stress-strain relationship.

Reliable measurements of soil deformation parameters are usually very difficult to obtain from field or laboratory tests. Measurements obtained from laboratory specimens may underestimate the stiffness of the insitu soil mass due to sample disturbance. Also, there can be other difficulties, e.g. choice of an appropriate model for interpreting the test results. Where possible, soil deformation parameters should be assessed from back analysis of the behaviour of previous work in similar ground conditions.

The stress-strain behaviour of soil is generally non-linear. However, it is often convenient in design to assume a linear or log-linear relationship between stress and strain for the behaviour of soil within a limited stress range. The soil models described in Sections 5.6.2 and 5.6.3 below both assume linear elastic behaviour. While these are generally adequate for estimating the order of magnitude of the deformation of retaining walls, in some cases it may be necessary to use more refined models to check the results (e.g. when serviceability criteria are being approached and the consequence of the serviceability limit state being reached is severe). An alternative approach is to calibrate the simple models using large-scale field tests.

5.6.2 Deformation Parameters for the Linear Elastic Continuum Model

For simplicity in computation of deformation, the ground is often modelled as a homogeneous isotropic linear elastic medium. Typical ranges of values of Young's modulus and Poisson's ratio for various types of soils considered as linear elastic media are given in Table 9.

For most saprolite of granite, tuff and rhyolite in Hong Kong, effective stress parameters are appropriate for design. The Young's modulus of granitic saprolite in terms of effective stress, E_s', has been correlated with SPT 'N' values without correction for depth. Such correlations, which are based on back analysis of case histories of buildings, plate

loading tests, pile tests and pumping tests, give E_s' values ranging from 0.2N to 2N MN/m^2 (Whiteside, 1986). These correlations are comparable to the range of results for sands given by Yoshida & Yoshinaka (1972), Parry (1978) and Burland & Burbidge (1985). The wide range of E_s' values is due to the different influence of the individual factors given in Section 5.6.1 in the various methods used to derive the values. For large retaining walls, field plate loading tests may be carried out to obtain E_s' values suitable for estimating the deformations of retaining walls. However, the factors mentioned in Section 5.6.1 and the problems associated with plate loading tests, e.g. softening of the ground, should be considered when interpreting results and selecting design parameters.

The one-dimensional or constrained modulus E_o', which is relevant to the case of one-dimensional compression, is often sufficient for estimating settlement. E_o' is related to E_s' and ν_s', the Poisson's ratio in terms of effective stress, by the following equation :

$$E_o' = \frac{E_s'(1 - \nu_s')}{(1 + \nu_s')(1 - 2\nu_s')} \quad \ldots \ldots \ldots \ldots \quad (5.2)$$

For values of ν_s' in the range 0.1 to 0.3, E_o' varies between $1.11E_s'$ and $1.35E_s'$. It should be noted that, by definition, $E_o' = 1/m_v$, where m_v is the coefficient of volume change of the soil. Values of m_v over the relevant stress range can be obtained from oedometer tests or from the consolidation stage of triaxial compression tests. However, experience indicates that values derived from conventional triaxial compression tests often overestimate the compressibility of Hong Kong granitic saprolite, and that values based on correlation with SPT 'N' values can give better predictions of overall field performance in such soils.

For cohesive soils of relatively low permeability, it is necessary to estimate immediate (i.e. short-term) as well as long-term deformations. In assessing immediate deformations, 'undrained' parameters in terms of total stress should be used. The Young's modulus corresponding to 'undrained' loading, E_u, may be calculated from E_s' and ν_u, the Poisson's ratio corresponding to 'undrained' loading, using the following equation :

$$E_u = \frac{(1 + \nu_u)E_s'}{(1 + \nu_s')} \quad \ldots \ldots \ldots \ldots \ldots \quad (5.3)$$

The value of ν_u for a saturated clay can be taken as 0.5. For values of ν_s' in the range 0.1 to 0.3, E_u varies between $1.15E_s'$ and $1.36E_s'$.

The value of Young's modulus for unloading and reloading is usually much higher than that for virgin loading. For preliminary design, values between 3 and 4 times the Young's modulus for virgin loading are appropriate for the stress range below the precompression pressure.

5.6.3 Deformation Parameters for the Winkler Model

(1) *General.* The 'beam on elastic foundation' or 'subgrade reaction' approach is often used for analysing the internal forces and deformations of a cantilevered retaining wall. In this approach, the wall is represented by a vertical elastic beam and the soil mass is modelled as a Winkler medium, for which displacement, y, is proportional to contact

pressure, p. The 'stiffness' of the Winkler medium is characterised by a coefficient of horizontal subgrade reaction k_h :

$$k_h = \frac{p}{y} \quad \ldots \ldots \ldots \ldots \ldots \quad (5.4)$$

The S.I. units commonly used in the above equation are kN/m^3, kPa and m (or MN/m^3, MPa and m) for k_h, p and y respectively. In actual design, the problem is often analysed numerically and the Winkler medium is represented by a series of elasto-plastic springs with spring stiffness or 'spring constant' k_s (Figure 9).

It must be emphasized that the Winkler medium is a grossly simplified model for soil behaviour. Therefore, sensitivity analysis should be carried out using a range of k_s values. Also, monitoring of lateral deflection is recommended in order to check the assumed soil deformation parameters as well as the predicted performance.

(2) *Wall Analysis.* For a sheet wall or a wall composed of closely-spaced piles (e.g. caissons), it is convenient to consider the forces acting over a unit width of the wall in design. The following equation may be used to evaluate k_s :

$$k_s = \frac{P}{y} = \frac{p\Delta z}{y} = k_h \Delta z \quad \ldots \ldots \ldots \ldots \quad (5.5)$$

where P = reactive *force* (per unit width of the wall) in the
            spring at depth z below ground surface
      y = lateral deflection of the wall at the spring location
      Δz = length of wall over which the spring acts

The consistent S.I. units for k_s and P are in kPa (i.e. kN/m per m deflection) and kN/m respectively. The units of y, z and Δz are all in metres.

For soils having values of Young's modulus increasing linearly with depth, k_h may be assumed to have a linear distribution with depth :

$$k_h = m_h z/d \quad \ldots \ldots \ldots \ldots \ldots \quad (5.6)$$

where m_h = constant of horizontal subgrade reaction
            of the soil for *wall* analysis
      z = depth below ground surface
      d = depth of wall embedment in the soil

For stiff walls such as bored pile (or caisson) walls, Rowe (1956) recommended m_h values of 2.4 and 63 MN/m^3 for loose and dense sand respectively.

As the parameter m_h is essentially a secant modulus, its value is strain dependent. Pun & Pang (1993) cautioned that the m_h values of Rowe (1956) were derived from a particular set of observations of very small deformations. Hence they are inappropriate to large strain conditions. Based on the passive earth pressure measurements on sand by Rowe & Peaker (1965), Pun & Pang (1993) derived the following empirical expressions for the m_h :

$$\frac{m_h}{K_p \gamma} = \frac{0.64}{(y'/d) + 0.017} \quad \text{for loose sand} \quad \ldots \ldots \text{(5.7)}$$

$$\frac{m_h}{K_p \gamma} = \frac{1.09}{(y'/d) + 0.011} \quad \text{for dense sand} \quad \ldots \ldots \text{(5.8)}$$

where K_p = coefficient of passive earth pressure
y' = lateral deflection of wall at mid-depth
   of embedment in soil
γ = unit weight of soil

For realistic assessment of deformation, best-estimate value of K_p calculated using unfactored soil strength parameters should be used in conjunction with equations (5.7) and (5.8).

(3) *Pile Analysis.* For a wall composed of widely-spaced piles, the forces acting on each pile should be considered. For a single pile, it is conventional to express the coefficient of horizontal subgrade reaction k_h in terms of a modulus of horizontal subgrade reaction K_h :

$$k_h = \frac{p}{y} = \frac{P}{B\Delta zy} = K_h/B \quad \ldots \ldots \ldots \ldots \text{(5.9)}$$

where B = width of the bearing area, e.g. width of pile
P = reactive *force* (acting on the pile) in the spring
   at depth z below the ground surface
y = lateral deflection of the pile at the spring location
Δz = length of pile over which the spring acts

The units of K_h, P and B are in kPa, kN and m (or MPa, MN and m) respectively. From equation (5.9), the spring stiffness k_s (= P/y) for use in a subgrade reaction analysis is simply $K_h\Delta z$.

For soils having values of Young's modulus increasing linearly with depth, K_h and k_h may be assumed to have a linear distribution with depth :

$$K_h = n_h z \quad \ldots \ldots \ldots \ldots \ldots \ldots \text{(5.10)}$$

$$\text{and } k_h = n_h z/B \quad \ldots \ldots \ldots \ldots \ldots \text{(5.11)}$$

where n_h = constant of horizontal subgrade reaction
   of the soil for *pile* analysis
z = depth below ground surface

The units of n_h and z are in kN/m^3 and m respectively. Table 10 summarizes the values of n_h suggested by Terzaghi (1955). These values are valid for stresses up to about half the ultimate bearing capacity, and they include an allowance for long-term movement. Back analysis of field test data by various authors gives values of n_h up to many times larger than Terzaghi's values (Habibaghi & Langer, 1984). These back-calculated n_h values correspond

to initial stiffnesses for short-term load tests, and are thus not strictly comparable with Terzaghi's recommendations. For sands, Elson (1984) suggested that Terzaghi's values should be used as a lower limit, and that the following relationship should be considered for use as the upper limit :

$$n_h = 0.19D_r^{1.16} \quad \ldots \ldots \ldots \ldots \ldots \ldots \quad (5.12)$$

where D_r = relative density of the sand

The units of n_h and D_r in equation (5.12) are in MN/m^3 and % respectively. The relative density of the sand may be estimated using SPT 'N' values (Figure 6). Discussion on the assessment of n_h from correlation with various soil properties is also given by Elson (1984), who reported the following relationship to obtain the value of K_h directly from the Young's modulus in terms of effective stress E_s' :

$$K_h = 0.8E_s' \text{ to } 1.8E_s' \quad \ldots \ldots \ldots \ldots \ldots \quad (5.13)$$

Further discussion on the effects of pile width and pile group on the deformation parameters is given by Siu (1992).

Figure 9 summarizes the methods which may be used for deriving k_s values for subgrade reaction analysis. The values of K_h, m_h or n_h required may be estimated using the guidance given in this Section. Where appropriate, field plate loading tests should be carried out to obtain E_s' values of the soil, from which K_h values can be derived.

5.7 SOIL PERMEABILITY AND CONSOLIDATION PARAMETERS

5.7.1 Permeability

Assessment of the permeability of backfills and of the insitu ground has to be made when evaluating the effects of water on the retaining wall and designing drainage measures (see Chapter 8). For insitu ground, the assessment of mass permeability must take into account the hydrogeological conditions within and outside the project site, the nature of the soil (including grading, heterogeneity, layering and discontinuities) and any other relevant features (e.g. decomposed dykes acting as aquicludes (Au, 1986)).

Table 8 gives typical ranges of values of mass permeability for a range of compacted fill materials and insitu soils commonly encountered in Hong Kong. Approximate permeability values of granular filter and drainage materials, in terms of particle size distribution and fines content, are given in Figure 10.

Permeability may be measured by testing laboratory specimens or by carrying out field tests. Laboratory permeability tests (e.g the permeameter test) are performed on small soil specimens which are prepared to densities representative of those in the field. Such tests should be used only where the soil is relatively homogeneous, such as the case of a granular filter or a drainage material. Specimens should be prepared to a density likely to be achieved by compaction. For saprolite, laboratory measurement of permeability (either by means of the oedometer or the triaxial test) has been found to yield results up to two orders of

magnitude lower than those obtained from field tests. For such soils, and also for sedimentary soils that show anisotropic permeability properties, field tests which measure the average properties of a large volume of soil are recommended (e.g. the borehole permeability test, the piezometer response test and the field pumping test). Geoguide 2 : Guide to Site Investigation (GCO, 1987) gives guidance on field permeability test methods and on interpretation of test results.

Permeability tests are only required where permeability is a critical parameter in design. Even with properly conducted testing, only the order of magnitude of the permeability of a soil mass is measured. In the evaluation of test results and selection of design parameters, allowance should be made for inaccuracies in measurement (e.g. due to difficulties in preparing the specimens to a uniform density in a laboratory permeameter test, or errors due to inherent variability in the properties of the soil mass in a field test).

5.7.2 Coefficient of Consolidation

In order to assess the time for primary consolidation of a saturated compressible soil, the coefficient of consolidation of the soil, c_v, is required. This may be determined from an oedometer test or from the consolidation stage of a triaxial compression test. However, for soils which possess 'fabric' (e.g. saprolite and layered sediments), laboratory c_v values are often unrealistically low. A preferable method is to use field test results to obtain the value of c_v by means of the following equation :

$$c_v = k_f E_o' / \gamma_w \qquad \ldots \ldots \ldots \ldots (5.14)$$

where E_o' = one-dimensional modulus of the soil obtained from empirical correlation based on field performance data (see Section 5.6.2)

k_f = field permeability of the soil (see Section 5.7.1)

γ_w = unit weight of water

This method is generally more reliable for assessing consolidation rates, provided that good quality field data are available.

5.8 ROCK STRENGTH PARAMETERS

5.8.1 Strength of Intact Rock

The strength of intact rock is usually not important in the design of retaining walls, although it can sometimes be useful as an index value. Rock strength may be assessed approximately from identification tests, e.g. by using a pocket knife or geological hammer (GCO, 1988). Where necessary, it can be measured by uniaxial compression tests on rock cores or point load index tests on rock specimens of any shape (ISRM, 1985; see also GCO, 1987). Typical ranges of values of uniaxial compressive strength of rocks commonly encountered in Hong Kong are given in Table 11. It should be noted that the strength of Grade III rock is very variable and can be quite low. Therefore, the strength of Grade III rock should be selected conservatively.

5.8.2   Shear Strength of Rock Joints

The shear strength of a rock joint can be expressed in a number of ways. Barton et al (1985) presented the development of a rock joint strength model that can account for scale effect. The model is based on the residual angle of friction, the joint roughness, the joint wall compressive strength, the effective normal stress and the length of joint. The residual angle of friction can be estimated from tilt tests on jointed rock cores. It can also be measured by direct shear tests with correction for dilation due to the roughness of the tested rock joint (Hencher & Richards, 1989).

A bi-linear strength envelope is also widely used for evaluating the shear strength of rock joints. At a low effective normal stress σ_n', the shear strength can be taken as σ_n' tan $(\phi_b + i_r)$, where ϕ_b is the 'basic' friction angle of a smooth planar discontinuity in the rock and i_r is a geometrical component due to roughness of the joint surface. This simplified model is usually adequate for routine design. However, the geometrical component of the friction angle is very hard to assess, particularly at field scale. At a high normal stress, the shear strength can be taken as σ_n' tan $\phi_b + c_j$, where c_j is the apparent cohesion of the joint.

Mineral infills are often present in rock joints. Infilled joints can have very low shear strengths, particularly if slickensided. Values of residual angles of friction for selected minerals have been reported by Deere & Patton (1971). Reference may be made to Hoek & Bray (1981) for guidance on the evaluation of shear strength of infilled rock joints.

5.8.3   Shear Strength of Rock Mass

The shear strength of a rock mass sometimes needs to be assessed in design, e.g. in the evaluation of passive resistance in bored pile (or caisson) wall design.

In one approach, the rock mass is treated as homogeneous and isotropic, e.g. for intact rock or a heavily jointed rock mass. Hoek & Brown (1980) presented an empirical failure criterion for characterizing the strength of homogeneous isotropic jointed rock masses. The failure criterion was subsequently modified by Hoek et al (1992), with the introduction of new material constants and an associated rock mass classification. There are limitations in the use of the criterion (Hoek, 1983 & 1990; Hoek et al, 1992). In particular, the designer should evaluate the applicability of the criterion to the rock mass at the site, taking into account the effects of relative scale, i.e. the spacing, orientation and possible persistence of the discontinuities in relation to the size of the structure being designed, and anisotropy.

Often, the nature of a rock mass is such that the assumption of homogeneous and isotropic properties is not valid. This can occur, for example, where discontinuities are preferentially oriented or widely spaced. In such cases, a distinct element model is more appropriate for design. In practice, a simplified wedge analysis is often used for rock socket design for bored pile (or caisson) walls extended into a rock mass (see Chapter 11). More refined analytical techniques are also available, e.g. the FLAC (Fast Lagrangian Analysis of Continua) and UDEC (Universal Distinct Element Code) computer programs (Cundall, 1980).

Where the stability or cost of a retaining wall is significantly affected by the selected values of the parameters, a sensitivity analysis should be performed and detailed geological mapping should be carried out during construction to verify major design parameters.

5.9 ROCK DEFORMATION PARAMETERS

Rock mass deformation parameters are required for carrying out serviceability limit state calculations, e.g. in the design of bored pile (or caisson) walls. As with shear strength, the effects of non-homogeneity, anisotropy and relative scale should be taken into account in the evaluation of such parameters.

The load-deformation characteristics of a rock mass are usually governed by discontinuities and can be modelled in terms of the closure, shear and dilation behaviour of rock joints as described by Barton (1986). In Barton's model, the closure, shear and dilation behaviour of a rock joint is represented by semi-empirical relationships, which are characterized by the properties of the joint, as well as the normal stress and displacement of the joint. The parameters required by the model can be determined in the laboratory using tilt tests, Schmidt Hammer tests and simple rock joint profiling techniques. Computer codes based on distinct element models such as FLAC and UDEC (Cundall, 1980) are appropriate for the calculation of rock mass deformations using such parameters.

Where it is appropriate to treat the rock mass as homogeneous and isotropic, a simplified approach is to use a 'modulus of deformation', E_m, to characterize deformation behaviour. In effect, E_m is a stress- (or strain-) dependent, secant modulus which may include components of inelastic strain in the rock mass due to closure or opening of the joint apertures and dilation or contraction resulting from joint shearing. For most retaining wall designs, an estimate of the order of magnitude of the rock mass deformation is adequate. In such cases, the use of an E_m model is acceptable.

For a rock mass containing discontinuities, the insitu modulus is generally much smaller than the laboratory modulus of the intact rock, E_l. BS 8004 (BSI, 1986a) gives the following equation which relates E_m to E_l through a 'mass factor' j :

$$E_m = jE_l \qquad \ldots \ldots \ldots \ldots (5.15)$$

For reasonably tight discontinuities, BS 8004 (BSI, 1986a) suggests that j may be taken as numerically equal to the average discontinuity spacing expressed in metres, with an upper limit of unity.

Typical ranges of values of E_l, the laboratory modulus and ν_l, Poisson's ratio for rocks commonly encountered in Hong Kong are given in Table 12. Values of E_l may be estimated from the uniaxial compressive strength of the rock, q_c, using appropriate values of 'modulus ratio', $\overline{M}$:

$$E_l = \overline{M}q_c \qquad \ldots \ldots \ldots \ldots \ldots (5.16)$$

For most common Hong Kong rocks, $\overline{M}$ can be assumed to lie between 200 and 500.

It should be noted that use of the E_m model described in this Section can only give a very rough estimate of the insitu modulus of deformation of a rock mass with no soft infilling. For other conditions, reduced modulus values may be appropriate. The following relationship given by Serafim & Pereira (1983) relates E_m to the CSIR Rock Mass Rating RMR (Bieniawski, 1974) :

$$E_m = 10^{(\frac{RMR - 10}{40})} \quad \ldots \ldots \ldots \ldots (5.17)$$

The above equation is only valid for RMR values less than about 85. The unit of the resulting E_m value is in GPa.

A geophysical technique using Rayleigh waves has been used successfully in the field to measure the shear stiffness of rock masses at very small strain. The method is fast and is much cheaper than a plate loading test (Building and Civil Engineering Research Focus, 1993).

Where the rock mass is modelled as a Winkler medium in a subgrade reaction analysis, the spring constants may be derived using Method (2) illustrated in Figure 9.

5.10 ROCK PERMEABILITY

Consideration of rock mass permeability may be required in the design of drainage measures for retaining walls founded on rock. In most rock masses, permeability is a function of the spacing and other characteristics of the discontinuities. A rock mass with tight joints is virtually impermeable. Where necessary, piezometer response tests, Lugeon tests or borehole permeability tests should be carried out to obtain an estimate of the permeability of the rock mass (GCO, 1987). Disturbance of the rock mass by the process of excavation can result in an increase in permeability due to opening of discontinuities. Also, stresses imposed on the rock mass can cause joint deformation such as closure or opening of the joint aperture, which will in turn result in changes in rock mass permeability (Barton et al, 1985). If the effect of such factors in modifying the hydrogeological conditions at the site is significant, this should be taken into account in design.

5.11 WALL FRICTION AND ADHESION

5.11.1 General

'Wall friction' is the shear force, i.e. the tangential component of the resultant (active or passive) force, which is generated at a soil/wall interface under certain loading conditions. The angle of wall friction refers to the angle between the normal to the interface and the direction of the resultant force. While values of the angle of wall friction, δ, are often expressed as a function of the angle of shearing resistance of the soil, or of the angle of skin friction between the soil and wall material, δ is not a material property.

The magnitude and direction of the mobilised wall friction at a limit state depends on

the relative movement between the soil and the retaining wall at that limit state, which in turn is a function of the limiting loading and foundation conditions. However, the magnitude of wall friction is limited by the available skin friction or bond strength between the soil and the construction material used for the retaining wall (Table 13). In some situations, the 'interface' is within the retained soil (e.g. the virtual back of an L-shape retaining wall). In such cases, the magnitude of wall friction is limited by the shear strength of the retained soil, which usually is not fully mobilised at the 'interface' at limit state, even for a wall founded on an incompressible foundation.

In design it is conventional to assume a constant value of δ, despite the fact that wall friction is generally not mobilised uniformly along the height of a retaining wall (James & Bransby, 1970; Milligan & Bransby, 1976). Different values of δ are required in the calculation of active and passive earth pressure, for the reasons given in the following Sections.

For walls retaining soft cohesive soils or granular soils which will be subjected to significant vibration (e.g. walls near railway tracks or machine foundations), δ should be assumed to be zero in design.

The adhesion to the back of the retaining wall is usually neglected since its value is difficult to determine and the simplification is on the safe side.

5.11.2 Wall Friction for Active Earth Pressure Calculations

Positive wall friction will be mobilised at an active state only where the retained soil tends to move downwards relative to the soil/wall interface or the virtual back of the retaining wall. Suggested maximum values of mobilised angle of wall friction for active earth pressure calculations for different types of walls are given in Table 14.

For the calculation of active earth pressure at the 'virtual back' of reinforced concrete L- or inverted T-shaped cantilever or counterfort retaining walls, the value of δ should be taken as the slope angle of the ground behind the wall, subject to an upper limit of $\phi'/2$.

For the range of ϕ' and δ values commonly used in design, the coefficient of active earth pressure generally reduces with increasing values of δ. For cantilevered retaining walls, the use of a value of δ larger than zero will reduce the calculated horizontal active force and its associated overturning moment. For gravity and reinforced concrete retaining walls, the calculations to check against sliding and bearing failure are not too sensitive to the δ value used, whereas the base width required to prevent overturning failure (or excessive tilting) of the wall is very sensitive to δ for a given retained height.

For crib and gabion walls, the internal filling materials should be adequately compacted or placed to achieve minimum porosity. Otherwise, vertical compression of the wall material will result in negative wall friction being generated at the virtual back after construction. This will significantly increase the coefficient of active earth pressure and hence the lateral force on the retaining wall (O'Rourke, 1987).

5.11.3 Wall Friction for Passive Earth Pressure Calculations

Beneficial wall friction will be mobilised at a passive state only where the soil in the passive zone tends to move upwards relative to the soil/wall interface. Table 15 gives the suggested maximum (negative) values of mobilised angle of wall friction for passive earth pressure calculations for different types of walls.

The calculated coefficient of passive earth pressure increases rapidly with increase in the absolute value of δ (see Section 6.6). Therefore, careful consideration should be given to the anticipated relative movement between the soil and the retaining wall in the selection of δ values for design.

5.12 BASE SHEARING RESISTANCE

'Base shearing resistance' refers to the shearing resistance available at the base of a gravity or reinforced concrete retaining wall. It is required to be calculated in the ultimate limit state check against sliding failure of free-standing retaining walls.

The available base shearing resistance depends on the nature and condition of the construction material used and of the foundation. It is also dependent on the form of construction (such as the presence of a shear key), which can influence the formation of sliding failure mechanisms. In selecting shear strength parameters for calculating base shearing resistance, account should be taken of any strength reduction at the wall/foundation interface and weakening of the foundation due to stress relief or upward hydraulic gradients. In contrast to the mobilisation of passive earth pressure, usually only a small relative translational movement at the base is required to mobilise full base shearing resistance.

For retaining walls founded on granular soil, the effective stress base shearing resistance is commonly expressed in terms of a parameter δ_b, the 'angle of base shearing resistance'. δ_b is in effect an angle of skin friction between the wall material and the foundation soil, assuming a $c' = 0$ soil strength model.

For mass concrete and reinforced concrete retaining walls without a shear key, δ_b is the angle of skin friction δ_s between the rough blinding concrete and the foundation soil, which may be taken as ϕ', the angle of shearing resistance of the foundation soil, for design purposes (Table 13). In calculating δ_b, the selected value of ϕ' should be taken as ϕ_{cv}', the critical state angle of shearing resistance. The use of ϕ_{cv}' gives a conservative estimate of δ_b as ϕ_{cv}' corresponds to the lower limit of ϕ' of the soil in a loose remoulded condition. Lower bound values of ϕ_{cv}' for common Hong Kong soils are given in Section 5.5.2.

Values of base shearing resistance higher than that calculated by assuming $\delta_b = \phi_{cv}'$ are acceptable where weakening of the foundation can be shown to be unlikely. This would require the construction programme to be specified such that excavation of the foundation is carried out in the dry season, and that the foundation is protected by a blinding layer (or other means) immediately after the Engineer has inspected the ground to check that the conditions are as assumed in the design. However, even with good site control, the design base shearing resistance should still be not greater than that calculated using ϕ', where ϕ' is the angle of shearing resistance of the 'undisturbed' foundation soil based on a $c' = 0$ soil

strength model.

Where a shear key is used, the base shearing resistance may be evaluated based on the selected value of shear strength parameters of the 'undisturbed' foundation soil (see Section 9.2.3(2)).

For rock- or earth-filled retaining walls (e.g. crib walls, gabion walls and reinforced fill structures), the design base shearing resistance should not be greater than that calculated using appropriate shear strength parameters of the infill, the filter/drainage materials at the wall base, or of the foundation soil, whichever is the weakest. Where a geotextile is provided at the wall base, the use of the appropriate δ_s value for the geotextile/soil interface should be considered (see Table 13). As for mass concrete retaining walls, ϕ_{cv}' of the foundation soil should be used to calculate δ_b (or δ_s) for the wall/foundation interface, unless foundation weakening can be shown to be unlikely.

For walls founded on cohesive soils of low permeability, the design base shearing resistance should not exceed the resistance calculated based on the soil's undrained shear strength, s_u. Due account should be taken of any strength reduction at the wall/foundation interface and weakening of the foundation soil.

The base shearing resistance of retaining walls founded on weathered granitic or volcanic (tuff and rhyolite) rock at a level where Grade I, II or III material is dominant may be determined conservatively from appropriate material properties, consideration being given to the nature of the rock/wall interface. Where the wall is keyed into rock of weathering grade I, II or III, the base shearing resistance should be calculated for a sliding mechanism involving rock joints.

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6. EARTH PRESSURE

6.1 GENERAL

The lateral pressure which acts on a retaining wall is a function of the materials and surcharges that the wall must support, the groundwater and foundation conditions, and the mode and magnitude of movement that the wall undergoes as a result of soil-structure-foundation interaction.

6.2 RELATIONSHIP BETWEEN MOVEMENT AND EARTH PRESSURE

6.2.1 States of Stress

The stresses acting on an element of soil within a soil mass may be represented graphically by the Mohr coordinate system in terms of the shear stress, τ, and the effective normal stress, σ_n'. In this system, the state of stress is indicated by a circle, known as the Mohr circle, constructed for the soil element (see Huntington (1957) for detailed guidance on the use of the system).

Critical combinations of τ and σ_n' on the Mohr diagram represent the shear strength of a soil. The locus of such combinations is known as the Mohr envelope of failure. In general, the failure envelope is curvilinear, but in order to minimise computational effort it is often approximated by a straight line on the Mohr diagram (Figure 11). If the Mohr circle lies within the failure envelope, the shear stresses in all directions are less than the shear strength of the soil. If the circle touches the failure envelope, the shear strength is fully mobilised along a plane within the soil element and a state of plastic equilibrium is considered to have been reached.

A special condition of equilibrium is the 'at-rest' state, at which the soil has not undergone any lateral strain, either because it has not been disturbed or because it has been prevented from expanding or contracting laterally with changes in vertical stresses. Any lateral strain in the soil will alter its original stress conditions. Depending on the magnitude of the strain involved, the final stress state in the soil mass can lie anywhere between two failure conditions, known as the active (K_a) and passive (K_p) states of plastic equilibrium.

The terms 'active' and 'passive' are also commonly used to describe the limiting conditions of earth pressure against a retaining wall. Active earth pressure is the lateral pressure exerted by the soil on the back of a retaining wall when the wall moves sufficiently outwards for the pressure to reach a minimum value. Passive earth pressure is the lateral pressure exerted on the wall when it moves sufficiently towards the soil for the pressure to reach a maximum value. It should be noted that throughout the literature, these terms are applied even when the method used for evaluating the lateral earth pressure does not assume the existence of an active or a passive state throughout the soil mass (e.g. Coulomb theory, which is based on limiting equilibrium analysis of a wedge of soil). For convenience, this convention has been retained in this Geoguide.

6.2.2 Modes of Movement

The movement required to mobilise the active or passive state depends primarily on the nature and stress state of the retained soil. Figure 12 gives the approximate required movements at the top of a rigid retaining wall for soil with different densities, under small at-rest pressure. The movements required to reach the active state are very small. For a wall which undergoes uniform translation, it may be assumed that the magnitude of the required movement is similar to that at the wall top in the rotational mode. Large movements are required to reach the passive state. For example, to mobilise passive earth pressure (i.e. the full passive resistance), translational movements of 2 to 6% of the wall height are required for dense sand and they can be well over 10% for loose sand (Rowe & Peaker, 1965).

The mode of movement has a considerable influence on both the magnitude and distribution of the lateral earth pressure (Figure 13). For a rigid retaining wall which is free to translate or rotate above its base, the active or passive condition may be assumed if it can be shown that sufficient movement can take place. In such cases, the earth pressure distribution can be regarded as triangular for both level ground (Figure 13(a)) and uniformly sloping ground.

In some situations, the movement of the retaining wall may be limited by an adjacent stiff foundation or other restraints (e.g. a piled bridge abutment or a screen wall framed in with the building structure). Structural deformations in such cases are usually insufficient to permit the development of the active condition, and at-rest pressure should conservatively be adopted for stability checks (Figure 13(b)).

For a rigid retaining wall rotating about its top, the soil near the top of the wall is restrained from expanding. This induces arching action in the soil and results in a non-triangular earth pressure distribution (see, for example, Harr (1966) and Fang & Ishibashi (1986)), with a corresponding resultant force acting slightly higher than that in the active condition (Figure 13(c)).

Compaction of backfill can cause relatively large earth pressures near the top of a retaining wall (Figure 13(d)). Depending on the size of the compaction plant used, the locked-in earth pressures can be very high. While methods exist for evaluating the effects of compaction (see Section 6.8), it is good practice to indicate on construction drawings the magnitude of compaction stresses allowed for in the design.

Flexible retaining structures behave differently from rigid ones. Deformation of flexible structures arising from soil-structure interaction will result in earth pressure distributions which cannot be predicted by classical earth pressure theories (Figure 13(e)). A review of some design methods used for flexible earth retaining structures is given in GCO Publication No. 1/90 : Review of Design Methods for Excavations (GCO, 1990).

Some methods of construction can give rise to restraints to retaining wall movement, thereby resulting in earth pressure distributions that are different from those assumed in classical theories (Jones, 1979). An example of this is the use of props to support the wall stem during compaction behind a reinforced concrete cantilever retaining wall. This can give rise to earth pressures which are different from those for a wall which has been backfilled

without propping. The earth pressures mobilised in these cases will be related to the 'effective' wall movement, i.e. that fraction of the total horizontal wall movement which occurs at a given point on the wall after that point has been buried by the backfilling operations. In most cases, the construction technique to be used is not known at the design stage. Hence these effects are usually ignored and are deemed to be covered by the factors of safety adopted in design.

The initial stresses in the ground can significantly affect the performance of walls retaining insitu material. While the initial stresses depend on the stress history and weathering history of the soil, certain methods of wall installation can modify these stresses (and sometimes the soil shear strength) and give rise to new 'at-rest' conditions (see Section 6.4). Depending on the stress states of the soil prior to bulk excavation in front of the wall, the movements required for the active or passive state to develop may be different from those given in Figure 12. For example, driving of a sheet pile into a dense soil displaces the soil and causes an increase in lateral earth pressures. In such cases, subsequent bulk excavation in front of the wall is likely to mobilise passive earth pressure after a relatively small movement. The situation is more complex for a loose soil in that the soil in the vicinity of the sheet pile wall will be densified. However, any improvement in the soil shear strength due to pile driving is usually ignored in design. A second example is excavation for a bored pile (or caisson) wall, which causes stress relief in the ground. Total stresses which equal the fluid pressures of the wet concrete used in the wall are then imposed, causing earth pressures to increase to values which may be higher than the overburden pressure of the soil (i.e. an apparent at-rest earth pressure coefficient greater than unity).

Bulk excavation in front of a retaining wall will relieve the overburden pressure on the soil in the passive zone, which will consequently behave as a precompressed material. The stress paths of soil elements in a precompressed material are not the same as those for a soil without precompression (Figure 14), and, depending on the initial stress state of the ground, movements larger or smaller than those given in Figure 12 are necessary to mobilise the active or passive condition respectively.

Thermal movement may impose additional stresses on a retaining wall, e.g. thermal expansion of a bridge may force the abutment retaining wall against the retained soil, thereby producing large earth pressures (Broms & Ingleson, 1971).

6.3 EARTH PRESSURES FOR DIFFERENT LIMIT STATES

The design earth pressure distribution for a retaining wall should be compatible with the limit state being considered, with due allowance given to the magnitude and direction of the anticipated movement of the wall. In general, the earth pressure distributions at different limiting conditions are not the same, and may also differ from those that exist under working conditions. However, because of the number of factors that can affect the actual earth pressure distribution, simplified distributions are often assumed in design. Guidance on design earth pressures for different limit states is given in Chapters 9, 10 and 11, which cover the design of the various types of retaining walls.

6.4 EARTH PRESSURE AT REST

The earth pressure at rest is a function of the shear strength of the soil, its stress strain history and weathering history. It is only applicable to those design situations where the retaining wall is effectively prevented from straining laterally. The magnitude of the earth pressure at rest is also of importance in assessing the amount of deformation required for the soil mass to reach the active or passive state.

For a horizontal ground surface the coefficient of earth pressure at rest is defined as the ratio of horizontal to vertical effective stress in the soil under conditions of zero lateral strain. For a normally-consolidated soil mass which has not been subjected to removal of overburden, nor to activities that have resulted in lateral straining of the ground, the coefficient of earth pressure at rest is given by K_o, which may be calculated using the following approximate expression (Jaky, 1944) :

$$K_o = 1 - \sin \phi' \qquad (6.1)$$

where ϕ' = angle of shearing resistance of the soil in terms of effective stress

This expression is applicable to the design condition of normally-consolidated sands and clays, and also to fill materials which fall within the requirements given in Table 4, with the ϕ' value calculated by assuming a $c' = 0$ soil strength model. It may also be used to obtain a conservative estimate of K_o in Hong Kong residual soils and saprolites (GCO, 1990). It should be noted that compaction of the backfill will result in earth pressures in the top layers which are higher than those calculated by equation (6.1) (see Section 6.8).

For a vertical wall retaining sloping ground, the at-rest earth pressure coefficient, $K_{o\beta}$, can be calculated from the following equation (Danish Geotechnical Institute, 1985) :

$$K_{o\beta} = (1 - \sin \phi')(1 + \sin \beta) \qquad (6.2)$$

where β = slope angle, with a positive value when the backfill slopes upwards

The earth pressure at rest, $p_{o\beta}'$, which acts in a direction parallel to the ground surface, may be taken to increase linearly with depth :

$$p_{o\beta}' = K_{o\beta}\gamma z/\cos \beta \qquad (6.3)$$

where γ = soil unit weight
z = depth below ground surface

For a downward-sloping backfill, the at-rest earth pressure should be calculated using equations (6.2) and (6.3) with the value of β set to zero.

The earth pressure at rest for the design condition of an overconsolidated soil is greater than that for a normally-consolidated soil. For level ground composed of lightly overconsolidated clay, the coefficient of earth pressure at rest, $K_{o,oc}$ may be estimated from the following equation (Canadian Geotechnical Society, 1985) :

$$K_{o,oc} = (1 - \sin \phi')OCR^{0.5} \quad \ldots \ldots \ldots \quad (6.4)$$

where OCR = overconsolidation ratio of the soil

For soils which have undergone complex stress histories, the distribution of the earth pressure at rest with depth should be carefully assessed (Burland et al, 1979). At shallow depths in a heavily overconsolidated clay, $K_{o,oc}$ may approach the passive earth pressure coefficient, K_p.

It should be noted that for walls retaining insitu ground, the initial stress conditions in the ground can be modified by the processes of wall installation and bulk excavation in front of the wall (see Section 6.2.2).

6.5 ACTIVE EARTH PRESSURES

6.5.1 Rankine Earth Pressure Theory

For a soil modelled as a c' = 0 (i.e. cohesionless) material, Rankine theory gives the complete state of stress in the soil mass, which is assumed to have expanded or compressed to a state of plastic equilibrium. The pore water pressure is assumed to be zero everywhere within the soil mass.

The Rankine active earth pressure is most conveniently calculated on a vertical plane, which is often referred to as the 'virtual back' of the retaining wall. The stress conditions require that the earth pressure on the vertical plane acts in a direction parallel to the ground surface and that it is directly proportional to the depth below the surface, i.e. the pressure distribution is triangular. Rankine's equation for active earth pressure is given in Figure 15(a).

For a retaining wall with an inclined rear face, the active force P_a and its angle of inclination i_a may be obtained by Mohr circle construction, or alternatively may simply be evaluated by the resolution of forces, as illustrated in Figure 15(b).

Rankine theory assumes sufficient movement can take place for the soil to reach a state of plastic equilibrium. However, there are some modes of movement which will invalidate the Rankine stress conditions. For example, for a retaining wall restrained at its top, the triangular Rankine earth pressure cannot develop (see Figure 13(c)).

Depending on the wall friction mobilised as a result of wall movement, the actual line of thrust acting on a retaining wall at failure may not coincide with the direction of the active force given by Rankine theory. However, because the Rankine stress field is such that all soil elements within the soil mass satisfy equilibrium and that the Mohr circles of stress are all limiting (i.e. the yield condition is not violated anywhere), the retaining wall will be 'safe' provided that it can support Rankine active force and any associated shear force acting on the back of the wall.

Rankine theory should not be applied where i_a, the angle which the Rankine active force makes with the normal to the back of the retaining wall, exceeds the angle of wall

friction δ selected for design. An example of such a case is the calculation of active earth pressures at the 'virtual back' of a retaining wall designed to retain a slope steeper than $\phi'/2$, for which i_a equals β and δ cannot exceed $\phi'/2$.

6.5.2 Rankine-Bell Equation

For a soil modelled as a c' - ϕ' material, the Rankine-Bell equation may be used to calculate the active earth pressure, p_a, at depth z for the case of a vertical wall retaining horizontal ground :

$$p_a = K_a \gamma z - 2c' \sqrt{K_a} \qquad \ldots \ldots \ldots \ldots \qquad (6.5)$$

where K_a = $(1 - \sin \phi')/(1 + \sin \phi')$
    z = depth below ground
    γ = soil unit weight
    c', ϕ' = apparent cohesion and angle of shearing resistance of the
            soil in terms of effective stress

Where there is a uniform surcharge and a horizontal water table below the ground surface, the term γz in equation (6.5) should be replaced by σ_v', the vertical effective stress, as given below :

$$\sigma_v' = q + \gamma z \qquad \text{for } z \leq h_w$$

$$= q + \gamma z - \gamma_w(z - h_w) \quad \text{for } z > h_w \quad \ldots \quad (6.6)$$

where h_w = depth of water table below ground surface
    q = uniform surcharge
    γ_w = unit weight of water

The application of the Rankine-Bell equation for a layered soil mass is illustrated in Figure 16. It should be noted that use of this equation assumes that zero wall friction is mobilised at the back of the vertical wall at the active state. Also, at the interface between two soil layers where the vertical effective stress is common, the calculated active earth pressures will be different if different values of c' and ϕ' are used for the soil layers.

6.5.3 Effect of Tension Cracks

It can be shown using equation (6.5) that the earth pressure is theoretically negative up to a depth z_0 given by the following equation :

$$z_0 = \frac{1}{\gamma} \left(\frac{2c'}{\sqrt{K_a}} - q \right) \qquad \ldots \ldots \ldots \ldots \qquad (6.7)$$

Most soils cannot sustain tensile stresses over long periods. Therefore, it is prudent to assume zero earth pressure in the theoretical tension zone. Cracks formed in the tension zone may be filled with water, e.g. during heavy rainstorms. If the soil is relatively

impermeable, this water may not drain quickly and can exert a hydrostatic pressure over the depth of the tension crack. Unless the ground conditions (e.g. surface cover, ground profile and permeability) are such that no surface water can accumulate in tension cracks, the effect of this additional pressure should be taken into account in design (see, for example, Figure 16).

For stress conditions between K_o and full passive earth pressure, tension cracks need not be considered.

6.5.4   Coulomb Earth Pressure Theory

In Coulomb theory, the force acting on the retaining wall is determined by considering the limiting equilibrium of a wedge of soil bounded by the rear face of the wall, the ground surface and a planar failure surface. Shearing resistance is assumed to have been mobilised both on the back of the retaining wall and on the failure surface. Coulomb's equation for active earth pressure for the design case of sloping ground in soil modelled as a c' = 0 material is given in Figure 17. This equation is only applicable to the case where the pore water pressure is zero within the soil mass.

In general, the Coulomb equation is only an approximate solution for the limiting earth pressure. This is because static equilibrium of the soil wedge is usually not fully satisfied, i.e. the active force P_a, the resultant force on the failure plane R, and the weight of the soil wedge W, are not concurrent and hence there is no moment equilibrium. The departure from the 'exact' solution is usually very small for the active case.

In contrast to Rankine theory, the angle of wall friction δ mobilised at limit state has to be assumed in using the Coulomb equation. Although the assumed value of δ does not significantly affect the calculated value of the active earth pressure coefficient K_a, it can have a major influence on the required base width of gravity and reinforced concrete retaining walls, due to its effect on the orientation of the active thrust line.

6.5.5   Caquot & Kerisel Charts

Caquot & Kerisel (1948) have presented tables of active and passive earth pressure coefficients derived from a method which directly integrates the equilibrium equations along combined planar and logarithmic spiral failure surfaces. These tables are applicable to soils modelled as c' = 0 materials. The pore water pressure is assumed to be zero everywhere within the soil mass. Curves based on the Caquot & Kerisel coefficients are shown in Figures 18, 19 and 20. Although these figures cover relatively simple ground geometries, they provide a very quick means of obtaining the required coefficients.

As in the Coulomb equation, the value of the mobilised angle of wall friction has to be assumed in the use of the Caquot & Kerisel charts. The guidance given in Section 6.5.4 is also applicable.

It should be noted that, for the case of a vertical retaining wall with a horizontal ground surface, Rankine theory always yields the maximum coefficient of active earth

pressure (Morgenstern & Eisenstein, 1970). Coulomb theory gives fairly close values to those given in the Caquot & Kerisel charts.

6.5.6 Trial Wedge Method

In some situations the equations of Rankine and Coulomb, and the Caquot & Kerisel charts, cannot be used for the evaluation of earth pressure without making simplified assumptions, for example where :

(a) the ground surface is irregular,

(b) the retained earth comprises several soil layers with non-horizontal interfaces,

(c) groundwater seepage exists, and

(d) there are concentrated loads and surcharges.

For the above situations, the trial wedge method (also known as the Coulomb wedge method) may be used. This is a graphical method which makes the same assumption of planar failure surface as in Coulomb theory. It is more powerful than Coulomb theory in that solutions to most active earth pressure problems are possible and that active earth pressures due to soils modelled as c' - φ' materials can be derived. Being a graphical method, it also has the advantage that the significance of the various factors involved can be readily appreciated in the calculation process. However, the method has certain limitations for the evaluation of passive earth pressures (see Section 6.6). The guidance given in Section 6.5.4 regarding the effect of δ value on design is also applicable.

The trial wedge method is illustrated in Figures 21, 22 and 23 for different situations. In all cases, the first step involves division of the ground into a series of wedges by selecting potential failure planes through the heel of the retaining wall. A force polygon is then constructed for each of the wedges in order to obtain the magnitude of the resultant force. The direction of the forces which act on the rear face of the retaining wall and on the trial failure plane are obtained by considering the direction of the relative movement at these surfaces. All the force polygons are then combined to obtain the limiting resultant force. For the active case, the maximum value of the resultant force is obtained by interpolating between the various force polygon values (e.g. see Figure 21(c)). Similarly, for the passive case the minimum value is determined. It should be noted that the forces acting on the critical wedge are in general not in moment equilibrium.

For soils modelled as c' - φ' materials, the effect of tension cracks should be considered in determining the lateral earth pressure (see Section 6.5.3). One of the tension cracks formed in the tension zone will extend downwards to reach the critical failure surface. This will reduce the length over which shearing resistance acts, but, in view of the smaller wedge weight, this has more or less the same effect as neglecting the reduction in earth pressure provided by the tension zone when applying the Rankine-Bell equation. Figure 22 shows the wedge analysis for this case.

The trial wedge method is rather laborious when the retained earth consists of different soil layers. The procedure for evaluating the active force for more complex ground conditions is illustrated in Figure 23.

For an irregular ground surface, the earth pressure distribution against the retaining wall is not triangular. However, if the ground does not depart significantly from a planar surface, a linear pressure distribution can be assumed, and the construction illustrated in Figure 24 can be used to determine the point of application of the active force. A more accurate method is given in Figure 25. This should be applied in cases where there are abrupt changes in the ground surface, non-uniform surcharges, or soil layers with very different shear strength properties.

6.6 PASSIVE EARTH PRESSURES

Rankine theory usually results in underestimation of passive earth pressure because in most cases (and particularly for the case of upward sloping ground), the direction of the line of thrust at passive failure is incorrectly assumed in the theory.

Coulomb theory can give rise to significant overestimation of the passive earth pressure : the error due to the assumption of a planar failure surface increases rapidly with increasing values of angle of wall friction δ (Morgenstern & Eisenstein, 1970). It is therefore recommended that Coulomb theory and the trial wedge method (see Section 6.5.6) should only be used when δ is less than $\phi'/3$ (Terzaghi, 1943). Also, care should be taken in the selection of the direction and magnitude of δ for design (see Section 5.11). The calculated passive earth pressure is sensitive to the value of δ, hence δ should not be overestimated as the error is on the unsafe side.

Methods which use curved failure surfaces should be applied when evaluating passive earth pressure for values of δ greater than $\phi'/3$. The values given by Caquot & Kerisel (Figures 19 and 20) may be used for simple ground geometries. For more complex ground profiles, the passive earth pressure may be calculated using the circular arc method illustrated in Figure 26. This method is quite laborious for even relatively simple conditions.

Alternatively, the generalised procedure of slices outlined by Janbu (1957) may be used (see Fredlund & Krahn (1977) for a simpler formulation of the analytical solution). Computer programs used for slope stability analyses can be readily modified for the computation of passive earth pressure (Lam, 1991). In the modification, proper account should be taken of the direction of the shearing resistance of the soil in the passive mode, which is opposite to that in a slope stability analysis. Also, in applying such computer programs, sufficient slip surfaces must be tried in order to obtain accurate results. This is important because the method gives an upper bound solution and the design factors of safety may not be adequate to cover errors due to the poor choice of critical failure mechanism. The advantages of a computerised generalised procedure of slices are that it can handle complex ground profiles and soil layering, and that appropriate piezometric pressures can be easily accounted for. Also, the computation is more efficient than the circular arc method carried out by hand.

Research has shown that the stresses and strains induced in a soil mass subjected to

a passive force are very complex and have a non-uniform distribution throughout the mass, as opposed to the constant shear strength parameters assumed in classical theories. Close to passive failure in dense sand, the average mobilised angle of shearing resistance can be well below peak strength because of progressive failure in the sand mass (Rowe & Peaker, 1965; Rowe, 1969). In loose sand, passive failure occurs at large strains and no peak value of passive earth pressure is observed at failure, in contrast to dense sand. With the current state of knowledge, the evaluation of passive earth pressure can only be based on a semi-empirical approach. In sands the use of the triaxial compression angle of shearing resistance ϕ_t', coupled with conservatively assessed values of mobilised angle of wall friction δ, can yield passive earth pressure coefficients which are satisfactory for design purposes.

6.7   INFLUENCE OF LIMITED BACKFILL

The extent of backfilling may be limited for certain retaining walls, e.g. a wall constructed immediately in front of a steep existing slope. The strength of the insitu material is often different from that of the compacted backfill. Where the ground mass is composed of soil derived from insitu rock weathering, mass characteristics such as relict structures and discontinuities should be taken into account in the design (see Sections 5.5.4 and 5.8.3).

The trial wedge method for a layered soil mass illustrated in Figure 23 may be applied for determining the active force for the case of limited backfill. Appropriate strengths should be used along different parts of trial failure surfaces which pass through different materials. The pressure distribution against the retaining wall may be determined using the procedure illustrated in Figure 25.

Depending on the extent of the materials and their strengths, the earth pressure against the retaining wall may be different from that calculated assuming uniform compacted backfill. Where the design is sensitive to the assumption on the extent of backfilling, the limits of excavation and backfilling should be clearly shown on the drawings.

6.8   EARTH PRESSURE DUE TO COMPACTION

For the construction of backfilled retaining walls, compaction of the backfill to a minimum dry density is routinely specified to ensure that it has adequate shear strength and stiffness. While compaction is important, the use of heavy compaction plant near a retaining wall can sometimes cause distress. This is because compaction induces large horizontal earth pressures which are subsequently locked into the soil mass. Such pressures can vary considerably both in magnitude and distribution and are often much greater than the earth pressures predicted by classical theories (Broms, 1971; Ingold, 1979a & b).

A simplified method has been proposed by Ingold (1979a & b) to estimate compaction-induced earth pressures against retaining walls. The design diagram for the horizontal earth pressure based on this method is shown in Figure 27. Appropriate values should be used for the earth pressure coefficient K, depending on the anticipated movement of the structure. Guidance on the types of limit states for which compaction-induced pressures should be considered is given in the chapters covering the design of the various types of retaining walls.

The compaction-induced earth pressure can sometimes be a significant part of the total pressure against a retaining wall. For reasons of economy, it is often worth limiting the loading due to compaction plant within a certain distance behind the wall. This is particularly important for vulnerable structures, e.g. reinforced concrete retaining walls with a thin cantilevered stem. For example, where the effective line load Q_l is limited to 10 kN/m, the compaction-induced pressure is generally less than about 11 kPa. The construction drawings should clearly indicate the compaction loading assumed in the design. Guidance on how to evaluate Q_l for different types of compaction plant is given in Figure 27.

6.9 EARTH PRESSURE BASED ON SOIL-STRUCTURE INTERACTION ANALYSIS

The use of classical theories for determining earth pressure is adequate for the design of most types of retaining walls covered by this Geoguide. However, for cantilevered retaining walls such as bored pile (or caisson) walls socketed in rock, soil-structure interaction analyses should be carried out for structural design and for checking serviceability.

While only very approximate, the 'beam on elastic foundation' or 'subgrade reaction' approach is commonly used for soil-structure interaction analyses because of the ease with which it can be applied. In this approach, the soil mass is commonly modelled as a series of elasto-plastic springs in which the reactive pressure generated in each spring is assumed to be proportional to its deflection (Winkler's model). Guidance on the evaluation of spring constants for use with the Winkler model is given in Section 5.6.3.

Various techniques exist for carrying out the subgrade reaction analysis. A review of such methods is given in GCO Publication No. 1/90 : Review of Design Methods for Excavations (GCO, 1990). With the common availability of computing facilities in design offices, established programs for linear elastic analysis of structural frames that can incorporate springs can be used directly for such purposes. However, it is recommended that these should be checked against closed form solutions before use. In applying such computer programs, the appropriate pressures on the 'active' side of the retaining wall and the 'initial' locked-in horizontal stresses on the 'passive' side should be input as loads. Responses such as the reactive forces in the passive springs, and the internal forces and displacements of the wall are obtained as the results of the analysis. The following checks should be made on the results :

(a) The magnitude of the resulting displacements should be compatible with the input earth pressure loading, e.g. active earth pressure is applicable only when there is sufficient movement to mobilise the active condition.

(b) The 'initial' horizontal stresses locked into the soil (assessed with due account taken of the at-rest earth pressure and the stress changes resulting from wall installation and bulk excavation in front of the wall, see Section 6.2.2) plus the reactive pressures (which are derived by dividing the reactive forces in the passive springs by the frontal area over which the springs act) should not

exceed the passive earth pressure of the soil.

Where the passive earth pressure is found to be exceeded at a spring level, the spring should be removed and the analysis repeated by applying a force equal to that due to the limiting passive resistance. In most cases, only a few iterations are required to obtain convergence.

While flexibility of the wall and soil stiffness are to some extent accounted for in a subgrade reaction analysis, the following limitations of this approach should be noted :

    (a) There is inherent theoretical difficulty in relating the spring constants to fundamental soil properties.

    (b) The subgrade reaction approach cannot directly simulate unusual initial stresses in the soil and development of wall friction.

    (c) Surface movements behind the retaining wall cannot be obtained directly from such an analysis.

More powerful techniques such as the boundary element and the finite element methods are available for solving general soil-structure interaction problems, but these are seldom necessary or justified for routine design of retaining walls. If there is a need to use these methods, due consideration should be given to the appropriateness and limitations of the soil model, as well as the initial stresses that exist in the ground and the boundary conditions. More extensive ground investigation and laboratory testing will also be required to characterise the soil for the purpose of accurate analysis. Detailed guidance on the use of these advanced techniques is beyond the scope of this Geoguide. Further discussion of some of these methods is given in GCO Publication No. 1/90 (GCO, 1990).

7.   EFFECTS OF SURCHARGE AND SEISMIC LOAD

7.1   TYPES OF SURCHARGE

Surcharges behind a retaining wall can be either permanent (e.g. loads due to shallow foundations of an adjacent building), or temporary (e.g. loads due to construction plant or storage of construction materials).

Loadings due to surcharge can be classified into two main types :

(a) uniformly-distributed loads : these are essentially continuous loads which act on the surface or the body of the retained ground, e.g. loadings from goods stacked uniformly on a platform or from traffic on roads, which may be treated as uniformly distributed actions, and

(b) concentrated loads, which include the following :

(i) line loads, e.g. loadings from strip footings,

(ii) point loads, e.g. loadings from square or circular footings, and

(iii) area loads, e.g. loadings from footings which, by virtue of their size in relation to the height of the retaining wall, have to be treated as area loads in order for their effects to be properly assessed.

7.2   DESIGN SURCHARGE LOADINGS

7.2.1   General

In Hong Kong, public highway and railway structures are generally designed for the loads given in the Civil Engineering Manual Volume V (EDD, 1983), which makes reference to BS 5400 : Part 2 (BSI, 1978) for highway loading. Retaining walls and bridge abutments which form part of a highway, as well as retaining walls for railways, should in general be designed for 45 units of type HB loading. Highway structures spanning less than 15 m and situated along rural roads other than trunk or main roads may be designed for type HA loading only. Where special conditions indicate that a smaller load would be appropriate, the agreement of the relevant authorities should first be obtained.

Where a structure (e.g. building) is located close to and behind the proposed retaining wall, the foundation details and the load distribution on the individual foundation elements should be ascertained and their effects on the retaining wall should be considered in design. While foundation loads can be idealised as a uniformly-distributed surcharge in some cases, the designer should critically consider the validity of this assumption in each situation. Sometimes it is necessary to make a direct assessment of foundation loads by considering the

dead and live loads, wind loads and other types of loading which may act on the structure (e.g. where footings are not uniformly spaced or have vastly different loading intensities). Wherever possible, the retaining wall should be located in such a way that large surcharge loadings do not exist near its back. A minimum surcharge of 10 kPa with an appropriate load factor should be allowed for around the periphery of all retaining walls to cover incidental loading during construction (e.g. construction plant, stacked materials and movement of traffic), except in cases where the layout of the site makes this clearly unnecessary.

The distribution of stresses on a retaining wall due to loads from foundations should be carefully assessed in design. A shallow foundation is very likely to exert much of the foundation load onto the retaining wall. Deep foundations which transmit loads to depth will have less effect on the retaining wall. Loads carried by individual footings will have localised effects which may affect local stability and influence the structural design. Laterally-loaded piles may exert lateral earth pressures far greater than the active earth pressures at shallow depths; such piles are sometimes sleeved to prevent lateral earth pressures from being transferred to the retaining wall. Heavy lifting equipment and machine foundations exert extreme loads which can have significant effects. Loads from such sources and their effects should be considered specifically in the design.

General guidance on the selection of loadings for design and on load combinations is given in Section 4.3.3. In some load combinations it may be necessary to consider different load cases due to possible differences in the locations of the surcharge loads. For example, in the case of an L-shaped cantilever retaining wall, two basic load cases for a uniformly-distributed surcharge should be considered in the design, as illustrated in Figure 28. The designer should include these load cases in the relevant load combination for the limit state checks.

7.2.2   Nominal Surcharge Loads

When the effect of surcharge compared with that due to earth and water pressures is small, the nominal surcharge loads given in Table 16 (expressed in terms of equivalent uniformly-distributed loads) may be used.

7.3   ASSESSMENT OF EFFECTS OF SURCHARGE

7.3.1   General

For a given surcharge, the earth pressures which act on a retaining wall depend on the load-spreading properties of the retained earth and the stiffness of both the wall and any support that may be present. Two approaches are commonly used for assessing the magnitude and distribution of lateral pressures due to surcharge, as discussed below.

The first approach assumes that the retained earth is in an active state. Rankine earth pressure theory (see Section 6.5.1) is sometimes used in the case of a uniformly-distributed surcharge, although at-rest pressures should be used for structural design if the wall is relatively stiff and cannot move forward to reduce these pressures (e.g. by virtue of its mass

or the presence of lateral supports). The trial wedge method (see Section 6.5.6) which includes the surcharge loads in the construction of the force polygon may also be applied. Because of the assumption of plane strain conditions, this method is applicable only when the surcharge is in the form of a uniformly-distributed load, line load or strip load. The method cannot be applied for a point load or an area load of limited extent. The earth pressures derived using this approach should not be used for structural design unless the deformation of the retaining wall (for the structural limit state being considered) can be shown to be sufficiently large for the assumption of active state of stress in the soil to be valid.

The second approach is based on experiments carried out on surcharge effects which have shown that the intensity and distribution of earth pressures due to concentrated surcharge are similar to those derived from elasticity theory. Discussion of the experimental evidence and the background to the modifications to the elasticity theory are given in Spangler & Handy (1984) and Terzaghi (1943, 1953). The earth pressures due to surcharge should be directly superimposed onto the active earth pressures and water pressure. It should be noted that this approach is not strictly correct for the checking of ultimate limit states, because the experiments were carried out at 'working' conditions when the retaining wall was not loaded to the point of incipient failure.

7.3.2 Uniformly-Distributed Loads

A uniformly-distributed load is commonly treated as an equivalent height of earth having the same density as the retained material. The earth pressure resulting from this load may be calculated for the increased retained height. The procedures for evaluating the magnitude and distribution of earth pressures given in Chapter 6 should be followed. In the calculations for soils modelled as c' - ϕ' materials, the depth of the tension zone should be taken from the top of the equivalent retained material (see Section 6.5.3).

7.3.3 Line Loads

The trial wedge method described in Section 6.5.6 can be used to determine the lateral earth pressures due to a line load which runs parallel to the retaining wall. In this method, the intensity per unit length of the line load should be added to the weight of the particular trial wedge to which the load is applied. A step will appear in the active force locus in the force polygon as the weight of the trial wedge suddenly increases when the line load is included. This method of calculation gives the maximum active thrust behind the wall but does not give the point of application of the resultant force. The point of application of the resultant and the earth pressure distribution with depth have to be determined by the procedure illustrated in Figures 24 and 25.

Alternatively, the magnitude and distribution of earth pressures acting on a vertical retaining wall due to a line load may be estimated by means of the formulae given in Figure 29. These have been modified from the Boussinesq solution for distribution of stresses in an isotropic semi-infinite elastic medium (Boussinesq, 1885). The modifications are based on experimental evidence and assume the presence of a rigid and non-yielding retaining wall (Spangler & Handy, 1984; Terzaghi, 1953).

7.3.4 Point Loads

The magnitude and distribution of earth pressures against a vertical retaining wall caused by a point load can be estimated by means of formulae which have been modified from the Boussinesq solution (Boussinesq, 1885). The relevant formulae are given in Figure 29. As in the case of line loads, these formulae are based on experimental evidence and assume the presence of a rigid and non-yielding retaining wall.

7.3.5 Area Loads

Lateral earth pressures against a retaining wall due to surcharge loads that can be treated as strip loads can be estimated using the trial wedge method. Alternatively, the modified Boussinesq formulae can be used to estimate the magnitude and distribution of these earth pressures by carrying out integration over the extent of the area loads (e.g. rectangular loads). A detailed treatment of this approach is given in Spangler & Handy (1984). Computers can be programmed to carry out the calculations by numerical integration.

7.3.6 Horizontal Loads

Lateral earth pressures against a retaining wall due to horizontal loads applied at the surface of the retained ground have received relatively little attention to date. However, there may be situations where horizontal loads exist and induce additional pressure on the retaining wall, e.g. lateral load transferred from road traffic onto an abutment wall. Little is known about the behaviour of retaining walls subjected to concentrated horizontal loads applied at the ground surface. Figure 29 gives an approximate pressure distribution acting on the back of a vertical retaining wall for the case of a horizontal line load.

Alternatively, the formulae given in Poulos & Davis (1974) can be used to evaluate the stress distribution in an isotropic semi-infinite elastic medium due to horizontal loads. Based on Carother's (1920) principle of images, the horizontal stresses given by these formulae should be doubled to obtain the lateral earth pressures acting on the back of a vertical rigid non-yielding smooth retaining wall.

7.4 SEISMIC LOADS

Hong Kong is situated in a region of low to medium seismicity and seismic load is generally not critical for retaining wall design. However, it is recommended that seismic load be considered specifically in design in the following cases :

    (a) abutment walls of highway and railway structures,

    (b) retaining walls affecting high risk structures or major lifelines, e.g. power plants and trunk water mains, and

    (c) cantilevered walls retaining saturated materials for which positive pore water pressures can be generated and soil shear

strength degrades under seismic excitation.

Examples of materials which degrade under seismic action include loose fill and loose colluvium, as described by Wong & Pang (1992).

The seismic load to be used for the design of abutment walls is given in the Civil Engineering Manual Volume V (EDD, 1983), which specifies a horizontal static force equivalent to an acceleration of 0.07g applied at the centre of gravity of the structure.

A project specific assessment should be carried out to evaluate the seismic load for the design of retaining walls associated with high risk structures or major lifelines. Such an assessment should include an analysis of the seismic hazard, a determination of the design earthquake return period, and an estimation of the amplification effects of the ground and the wall. For the purpose of seismic hazard analyses, reference should be made to GCO Publication No. 1/91 : Review of Earthquake Data for the Hong Kong Region (GCO, 1991), which provides information on earthquakes within a distance of about 350 km from Hong Kong. An outline of a seismic hazard analysis carried out for the Hong Kong region is given by Pun & Ambraseys (1992).

The Mononobe-Okabe (M-O) method (see Dowrick (1987) for a detailed description of the method) can be used to calculate the seismic forces induced on a retaining wall. The M-O method is based on plasticity theory and is essentially an extension of the Coulomb earth pressure theory. Although it grossly simplifies the soil-structure interaction in a seismic event, the M-O method has been used successfully for seismic design of retaining walls in many parts of the world.

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8.   EFFECTS OF WATER

8.1   GENERAL

The presence of water behind a retaining wall has a marked effect on the forces applied to the wall. Many recorded retaining wall failures can be attributed to the action of water. Therefore, it is of utmost importance to provide adequate drainage behind a retaining wall and to take proper account of the appropriate water pressures in design.

Detailed guidance on the evaluation of design water pressure is given in the following Sections. However, for conventional gravity and reinforced concrete retaining walls with adequate drainage provisions, zero water pressure may be assumed for design. A detailed assessment of water pressure is always required for gravity and reinforced concrete retaining walls of unconventional design (e.g. walls retaining a substantial height of soil, such as that described by Vail & Holmes (1982), and walls retaining mine tailings) and cantilevered retaining walls.

8.2   DESIGN WATER PRESSURES

8.2.1   Design Condition

The design water pressures should be based on the *worst credible* groundwater conditions that would arise in extreme events selected for design. Examples of extreme events are severe rainfall, flooding, and bursting of water mains. The *worst credible* water level in an extreme event should be the highest that can be envisaged given the nature of the problem, and should be assessed with consideration of catchment area, water recharge potential, drainage, etc. It should not be taken as the worst possible water level out of context, e.g. groundwater level up to the top of the retaining wall. Some guidance on the determination of the *worst credible* water conditions is given in the following paragraphs.

Where the phreatic surface or groundwater table exhibits a storm response, the design water level should be based on a storm rise corresponding to a 1 000-year return period rainstorm added to a typical groundwater level occurring before the storm. The storm rise should be that which corresponds to a rainstorm of the most critical duration for the site.

For a seasonally responding groundwater level, a seasonal rise corresponding to a 1 000-year return period wet season should be added to a typical groundwater level occurring at the beginning of the wet season.

Some groundwater levels exhibit both storm and seasonal response. In this case, either a 1 000-year return period storm rise should be added to a typical wet season water level or a typical storm rise should be added to a 1 000-year return period seasonal rise, whichever gives a higher water level.

It should be noted that the 1 000-year return period given above is merely used to indicate the low level of probability of exceedance which a designer should aim at in selecting extreme events for design. This recommendation should not be taken as a

requirement to achieve accurate prediction of water levels. As the difficulties of predicting water levels associated with rare rainfall events are well known, designers are advised to exercise judgement and to select design water levels conservatively. The intensity of a 1 in 1 000 year rainfall has a probability of exceedance of about 11% in 120 years. This low probability level may be used as a guide for selecting other extreme events for design.

Water-bearing services such as storm water drains, sewers and water mains often leak. As a minimum precaution, if at all possible, no service drains, ducts and other pipes should be placed within the 'active' zone behind a retaining wall. In some cases, more remote services could still affect the stability of the retaining wall. Therefore, each case should be considered on its own merits.

In cases where the proposed development layout cannot be modified to permit the siting of service ducts outside the zone which may influence the retaining wall, the design should take account of the effects of possible water leakage. One solution is to house services within a sealed trench, ducting system or sleeve connected to a suitable discharge point at a surface drain or natural stream. Discharge from the system should be monitored at appropriate intervals in order to detect any major leaks.

For retaining walls in low-lying areas such as reclamations, the risk of burst water mains, which can result in a local rise in water level up to or even above the ground surface, should be allowed for in the design. In areas where the groundwater level is low, the routing of water-bearing services away from the retaining wall as discussed above, if possible, is a more practical solution.

8.2.2 Evaluation of Water Pressures

Many factors influence the water flow pattern and hence the water pressures which act behind a retaining wall, e.g. nature of the ground, the groundwater conditions, the topography and surface cover, the land use and natural drainage at the site, as well as the layout of drainage provisions.

In designing a retaining wall, the groundwater conditions at the site should be established. This involves establishing a hydrological model based on the geology of the site. Observation of groundwater in standpipes, piezometers, boreholes, trial pits, as well as seepages on exposed surfaces, provides useful information. Reference should be made to Chapter 20 of Geoguide 2 : Guide to Site Investigation (GCO, 1987) for guidance. It is important to know the order of permeability of the various geological units and of any fill material to be placed behind the retaining wall, as these can have a significant influence on the water flow pattern. For example, for a compacted backfill with a permeability of 10^{-8} m/s, a foundation composed of completely decomposed granite with a permeability of 10^{-5} m/s will act as a drainage layer for the backfill.

The design water pressures can be evaluated from a flow-net based on the assessed worst credible groundwater conditions. Flow-nets may either be constructed manually or they may be obtained using numerical seepage analysis based on the finite element or finite difference method. Cedergren (1989) provides guidance on flow-net construction and the theory of groundwater flow. In order to obtain a realistic water flow pattern the seepage

analysis should be performed using unfactored permeability values selected for design. The sensitivity of the results to variations in permeability values should be assessed. Where a commercial computer program is used to carry out the seepage analysis, the assumptions made in the program and its limitations should be recognized. It is recommended that the results of computer analyses should be checked by carrying out a quick sketching of flow-nets by hand. It should be noted that flow-net construction is unnecessary for gravity and reinforced concrete retaining walls for which a proper inclined drain system is provided (see Section 8.3.2).

Infiltration into the ground can cause a rise in groundwater level and induce an increase in pore water pressure in the soil. Steady state seepage will occur when there is sufficient water infiltrating into the retained soil. The steady state condition is most likely to be achieved in a soil with a permeability of around 10^{-5} m/s, as rainfall of moderate intensity can supply enough water to sustain the seepage flow. The water pressures under such a condition may be evaluated from a flow-net.

Where the retained soil is of high permeability, say 10^{-4} m/s or more, infiltration will descend vertically without reaching a steady state flow. If the groundwater level does not rise above the retaining wall base, positive pore water pressures will not develop behind the wall. In such a case, water pressures induced by infiltration need not be considered in the design.

For compacted fill of low permeability, say 10^{-6} m/s or less, the infiltration rate is limited by the permeability of the fill and the time required for steady state flow to develop is much longer than the duration of any significant rainfall. In such a case, there is no need to consider infiltration-induced water pressures provided that the site drainage is designed to remove any possibility of ponding.

Theoretically, positive piezometric pressures will not arise as a result of infiltration except for backfills with a permeability of around 10^{-5} m/s. However, the source and hence the nature of the backfill is often not known at the design stage. For such situations, it is prudent to take account of the effect of infiltration in design.

8.3 EFFECTS OF WATER ON EARTH PRESSURES

8.3.1 General

The water flow pattern around a retaining wall can affect the earth pressure on both the 'active' and 'passive' sides of the wall. Limit equilibrium methods (e.g. the trial wedge method, see Section 6.5.6) are often used to evaluate earth pressures. In using such methods, account should be taken of the piezometric pressures on the potential failure surface and at the soil/wall interface, i.e. effective stresses should be used (Figure 30).

8.3.2 Gravity and R.C. L- or Inverted T-shaped Retaining Walls

Typical flow-nets around gravity and R.C. L- or inverted T-shaped retaining walls under steady-state groundwater seepage are shown in Figure 31. It can be seen that with an

inclined drain system, no positive pore water pressure will be developed at the soil/wall interface and along potential failure surfaces. Thus water pressures can be assumed to be zero in the design. On the other hand, with a vertical wall-back drain positive pore water pressures can exist under certain conditions (see Section 8.2.2), and the appropriate water pressures should be allowed for in the design (see Section 8.3.1).

For situations where the groundwater level will not rise above the retaining wall base, there will not be any uplift pressure. However, if the groundwater level can rise above the wall base, uplift pressure will exist even if an inclined drain system is provided. In such cases, the uplift pressure should be determined from a flow-net.

For permeable walls such as gabion walls, no positive water pressure can develop at the rear face of the wall nor can there be any uplift pressure unless the wall is submerged. For submerged permeable walls, the appropriate uplift water pressure at the base should be taken into account in design.

With regard to infiltration into the backfill, typical flow nets are shown in Figure 32. Again water pressure may be assumed to be zero in the design where an inclined drain system or a horizontal drainage layer is provided. For walls with a vertical wall-back drain, the appropriate water pressure should be allowed for in the design. Figure 30(c) gives a curve which may be used to estimate the total water forces acting on potential failure surfaces behind such walls.

8.3.3 Cantilevered Retaining Walls

A cantilevered retaining wall is often impermeable either by nature of the structural material used, or as a result of the provision of a groundwater cut-off by means of grouting. An impermeable wall can 'dam' the groundwater flow across the site and result in a rise in groundwater level. Where appropriate, this 'damming' effect should be assessed and allowed for in design. After establishing the groundwater levels on the 'active' and 'passive' sides, the design water pressures should be assessed by a seepage analysis.

For an impermeable wall in a homogeneous isotropic soil under steady-state groundwater seepage, the simplified water pressure distribution shown in Figure 33 is appropriate for design purposes. This simplified distribution assumes that the hydraulic head varies linearly down the back and up the front of the wall.

For sites with variations in soil hydraulic properties, the resultant water pressures can exceed those in the homogeneous isotropic soil condition (Figure 34). The presence of pervious silt or sand partings within a clay stratum may also convey water at hydrostatic pressure to the toe of the wall (Padfield & Mair, 1984). In such cases, the simplified water pressure distribution shown in Figure 33 may not be sufficiently accurate. Hence, a proper flow-net analysis should be carried out and the effect of the water flow pattern on earth pressures should be evaluated (see Sections 8.2.2 and 8.3.1).

Where a relatively impermeable soil layer (e.g. a marine clay stratum) or a rock mass of low permeability is present immediately below the toe of a retaining wall, a hydrostatic water pressure may act on the wall.

For a wall composed of vertical structural elements spaced horizontally apart (e.g. hand-dug caissons), some through-flow of groundwater can occur. Figure 35(a) shows a typical flow-net around a caisson wall under steady-state groundwater seepage. Depending on the spacing of the vertical structural elements, different degrees of 'damming' of the groundwater can occur. The rise in groundwater level due to the 'damming' effect should be properly assessed and allowed for in design. Pope & Ho (1982) present a simplified two-dimensional approach for assessing this effect.

Drainage may be provided behind an impermeable wall to lower the groundwater level. The effects of any drainage provision on the water flow pattern should be properly assessed. The effect of infiltration should also be considered in the design (see Figure 35(b), and Sections 8.2.2 and 8.3.1).

8.4 DRAINAGE PROVISIONS

8.4.1 General

Except for retaining walls which are designed to resist water pressures, e.g. basement retaining walls, it is good practice to provide adequate drainage behind a retaining wall. Typical drainage arrangements are given in Chapters 9 to 11.

The drainage system to be provided should be designed for the anticipated water flow without backing up or blocking. To prevent blockage, the drain or drainage material should be protected by a suitable filter material. In some cases, the filter material may have an adequate permeability to provide the necessary drainage capacity. Guidance on granular and geotextile filter design is given in Section 8.5.

For walls retaining a granular soil of high permeability, the provision of additional drainage materials is not required. However, some means of draining the water which has percolated through the backfill should be provided, particularly where the wall is founded on an impervious foundation.

For a retaining wall with level backfill, the top 1.5 m layer of fill should be a suitable material of relatively low permeability (see Section 3.7) and the ground surface should be formed with an adequate gradient towards a surface drainage channel in order to avoid ponding which can result in continuous infiltration into the backfill.

Where the retaining wall is impermeable, drainage holes (weepholes) should be provided through the wall to prevent the build-up of a hydrostatic pressure behind it. Weepholes are normally 75 mm in diameter and at a spacing of not more than 1.5 m horizontally and 1.0 m vertically; rows should be staggered. The lowest row should be about 300 mm above finished ground level. Weepholes should be provided to a level above which groundwater flow is unlikely to occur. It is also good practice to provide weepholes over the face of a retaining wall where significant infiltration is anticipated. In the case of basement walls that are designed to be watertight, weepholes should obviously be omitted but suitable waterproofing measures should be provided.

It is important that the drainage system is provided with sufficient discharge points.

These should be connected to suitable outlets. Weepholes should not be relied upon as the sole means of discharge. Drainage pipes should be provided for removing water. These should be adequately protected to ensure that they are not damaged during construction or in service. Roding eyes or manholes should also be provided at suitable locations to facilitate the clearing of pipe blockages later.

8.4.2 Design Aspects

The rate of seepage flow into the drainage system of a retaining wall is governed by the design water level and the permeability of the soil (or rock) being drained. Its value can be estimated from a seepage analysis. For such an analysis, the selected permeability values should be used directly in the estimation of seepage flow rate without factoring (Table 7).

As a general guide, the permeability of the selected drainage material should be at least 100 times that of the soil (or rock) to be drained. If this is achieved, the drainage material may be considered as a free drainage boundary relative to the retained soil mass.

The required cross-sectional area of the drainage material, A, can be calculated from the design seepage flow rate using Darcy's law :

$$A = \frac{q_d}{ki} \qquad \ldots \ldots \ldots \ldots \ldots \ldots (8.1)$$

where q_d = design seepage flow rate
k = permeability of the drainage material
i = hydraulic gradient of the flow within the drainage material

For an inclined drain behind a gravity or R.C. L- or inverted T-shaped retaining wall, i may be assumed to be equal to sin β_d, where β_d is the angle of inclination of the drain to the horizontal. For a vertical wall-back drain, depending on the drainage capacity of the outlet, a static water head may build up behind the retaining wall. In such cases, the resulting hydrostatic water pressure should be taken into account in design.

A partial material factor of 10 should be applied to the permeability of granular filter and drainage materials in drainage design (Table 7). Typical permeability values of such materials and guidance on methods of determining soil permeability are given in Section 5.7.1.

The thickness of granular drainage layers is often governed by construction considerations rather than by the drainage capacity criterion. For walls retaining compacted backfill, the provision of a drainage layer with a nominal thickness of 300 mm is usually adequate. Cedergren (1989) provides guidance on the sizing of inclined drains.

Prefabricated drainage composites made up of a synthetic core wrapped with a geotextile filter envelope have become increasingly popular. Where such materials are to be used, the design should ensure that the transmittivity of the composite drain is adequate when the drain is subjected to the design earth pressure. A partial material factor of 10 should be

applied to the transmittivity of the drain. The geotextile filter envelope should comply with the filter design criteria given in Section 8.5.3.

Close control is required in the construction of drainage provisions and an adequate level of supervision should be provided (see Chapter 12). In particular, care should be exercised to ensure that granular filter materials are handled and placed properly to prevent contamination and segregation. This is particularly important where the design has to rely on the adequacy of the drainage measures provided to control water pressure. Where areas of concentrated flow exist in the retained ground, local drainage measures may need to be incorporated to remove water safely.

8.5 GRANULAR AND GEOTEXTILE FILTER DESIGN

8.5.1 Performance Criteria

All drainage provisions should be adequately protected by properly designed filters. The following two main performance criteria for filters should be met in design :

- (a) The *retention criterion*, i.e. there must be no excessive loss of particles from the base soil initially, and no further loss of soil after the initial period of soil 'piping'.

- (b) The *permeability criterion*, i.e. the system permeability must reach an equilibrium level which must be sufficiently high so that water flow is not impeded, and which must not reduce with time.

Other design criteria and considerations specific to each type of filter are given in the relevant Sections below.

8.5.2 Design Criteria for Granular Filters

The design criteria for granular filters are set out in Table 17. These criteria are briefly described below.

The retention or stability criteria (Rules 1 and 2) are aimed at preventing the significant loss of fine particles from the filter and base soil system. The purpose of Rule 1 is to limit the size of pore channels within the filter so as to restrict the movement of fine particles from the base soil into the filter. The aim of Rule 2 is to prevent the loss of fine particles from within the filter itself.

The permeability criteria (Rules 3 and 4) are intended to facilitate groundwater flow from the base soil through the filter. The purpose of Rule 3 is to ensure that the filter is much more permeable than the base soil which it protects, while Rule 4 is intended to set a lower limit to the filter permeability by restricting the minimum grain size of the filter material.

The purpose of the segregation criteria (Rules 5 and 6) is to prevent the separation of coarse and fine particles within the filter. Rule 5 is intended to ensure a reasonably uniform grading of the filter, while Rule 6 restricts the maximum grain size of the filter.

Other factors which may be applicable to filter design and construction are covered in the notes in Table 17.

With regard to Note (2), it is known that the coarse particles of widely-graded base soils, which are commonly encountered in Hong Kong, have little effect on the filtration process. Therefore, for those base soils containing a significant amount of both gravel and fines, the coarse part should be ignored, and a revised base soil grading curve consisting of the particles smaller than 5 mm only should be used in the design.

The minimum thicknesses of filter layers recommended in Note (3) are intended to ensure integrity and consistency in performance appropriate to the chosen method of construction.

Note (4) draws attention to the point that Rule 5 should be applied to individual batches of the filter. Even if an individual batch grading curve complies fully with the design limits of the filter grading envelope, it does not necessarily ensure that the filter will satisfy Rule 5.

It should be noted that Rules 2, 4, 5 and 6 are concerned with the internal requirements of a filter. If these are satisfied, the material is a good filter, but it is also necessary to check that it is compatible with the particular base soil, in accordance with Rules 1 and 3.

Guidance on specification and construction aspects relating to granular filters is given in Section 3.3.1.

8.5.3   Design Criteria for Geotextile Filters

The geotextile filter criteria given in Tables 18 and 19 are appropriate for Hong Kong soils (including those derived from insitu rock weathering, i.e. residual soils and saprolites) provided that the following important conditions are met, namely :

(a) the flow through the geotextile filter is unidirectional,

(b) the hydraulic gradients close to the filter are moderate to low,
    and

(c) the effective normal stresses acting on the geotextile are static
    (i.e. little or no dynamic stress is exerted on the geotextile).

The above conditions are usually satisfied in the application of geotextile filters in retaining walls.

Until more experience is gained, Lawson's (1987) criteria (Table 18) are appropriate

for woven geotextiles and nonwoven heat-bonded geotextiles, and the French Geotextile Manual criteria (Table 19) are appropriate for nonwoven needle-punched fabrics. Tables 18 and 19 utilise O_{90} and O_f for design. Discussions on the determination of these geotextile opening sizes, and their relationship with other opening sizes, are given in GEO Publication No. 1/93 (GEO, 1993).

Grading tests should be carried out to obtain parameters which are representative of the soil, and such tests should be carried out without dispersants. The D_{85} size for use in the retention criterion should be selected conservatively to allow for soil variability. For a widely-graded base soil with $D_{90} > 2$ mm and $D_{10} < 0.06$ mm, the percentage of particles greater than 5 mm should be ignored and the finer fraction of the grading curve should be used for filter design.

Where the groundwater is continually being removed (as opposed to situations where flow occurs only in severe rainstorms), consideration should be given to performing filtration tests as part of the design. The tests described by Greenway et al (1986a) or Scott (1980) are suitable, and they should be carried out using the soils on site or the proposed fill materials, as relevant. Filtration tests should also be performed for designs involving gap-graded or dispersive soils. The time required to carry out filtration tests should be allowed for in the project programme.

Where the risk to life is high, as indicated in Table 5.2 of the Geotechnical Manual for Slopes (GCO, 1984), a detailed investigation of the site conditions should be carried out to assess the likely risk of clogging of the geotextile caused by chemical deposits and organic residues. This should include an examination of the performance of drainage systems in the vicinity of the site. Particular care should be taken where the site is adjacent to factories or waste disposal facilities, or where a high level of soil biological activity is anticipated, e.g. within a former industrial area as indicated by a high organic matter content in the soil.

Guidance on specification and construction aspects relating to geotextiles is given in Section 3.3.2.

8.6 DESIGN AGAINST HYDRAULIC FAILURE

The groundwater level behind a retaining wall may need to be controlled to prevent hydraulic failure. If the groundwater level behind the wall can rise, flow will occur under the wall and upward towards the formation level if the base of the wall does not penetrate into an impermeable layer. Instability or gross loosening in front of the wall, involving piping or heave, can take place if the vertical seepage exit gradient is equal to about unity in dense sands or when the uplift force at the toe of the wall exceeds the weight of the overlying soil column in loose sands.

The usual method of preventing hydraulic failure of an impermeable sheet retaining wall is to design the wall so that it penetrates a sufficient depth below the formation level. Design charts for estimating the depth of penetration required to prevent heave or piping in isotropic sands and in layered subsoils are given in Figures 36 and 37 respectively. The design curves given in Figure 36 are based on the work of Marsland (1953), and the factors of safety shown are defined in terms of the exit gradient for dense sands and the uplift force

for loose sands.   In order to cater for uncertainties in the ground conditions, as well as uncertainties in the applicability of the charts to the particular site conditions, a minimum factor of safety of 2.0 should be provided.   The net hydrostatic head should be carefully assessed in the design.

In the case of gravity retaining walls, hydraulic failure can normally be prevented by incorporating adequate drainage measures in the design, e.g. by providing an inclined drain behind the wall.   Nevertheless, the possibility of piping or heave during construction should be considered if the groundwater level can rise above foundation level.   Guidance on groundwater control for excavation and foundation works is given in Section 3.6.3.

9. GRAVITY RETAINING WALLS

9.1 GENERAL

This Chapter gives general guidance on design methods for gravity retaining walls and deals with selected design and construction aspects of various wall types, viz. mass concrete retaining walls, crib walls and gabion walls. While the general guidance in Section 9.2 also applies to reinforced fill structures, specific advice on the design and construction of such structures is given in Geospec 2 : Model Specification for Reinforced Fill Structures (GCO, 1989a).

9.2 METHODS OF DESIGN

9.2.1 General Principles

The limit states which should be considered and calculation models which can be used in the design of gravity retaining walls are outlined in the following Sections. The general approach for verifying the safety and serviceability of retaining wall designs described in Chapter 4 should be followed. Guidance on evaluation of geotechnical parameters and determination of selected values is given in Chapter 5. Where appropriate, earth pressures (and resistances) and the effects of surcharge (and seismic load) and water should be evaluated using the methods given in Chapters 6, 7 and 8 respectively.

9.2.2 Limit States

A list of limit states to be considered in design should be compiled in accordance with the principles given in Section 4.1. As a minimum, the following ultimate limit states of external instability should be considered (Figure 38) :

(a) loss of overall stability,

(b) sliding failure,

(c) overturning failure, and

(d) bearing capacity failure.

Where appropriate, the possibility of hydraulic failure should be checked. Also, serviceability limit states, which include excessive settlement, translation and rotation of the retaining wall, should be guarded against in the design.

Guidance on the partial factors of safety to be used in design against ultimate and serviceability limit states is given in Section 4.3.4.

In addition to the above, limit states related to structural design should be considered. Some general guidance on structural design is given in the relevant Sections of this Chapter

which cover the various wall types.

The various elements of design for gravity retaining walls are summarised in Table 20.

9.2.3   Design Against Ultimate Limit States

(1) *Loss of Overall Stability*. The construction of a new retaining wall will bring about stress changes in the ground mass containing the wall, which could result in 'overall' instability. Loss of overall stability is likely to occur in an area which in itself is of marginal stability (e.g. a steeply sloping site or a slope with a high groundwater level) or where a weak subsoil (e.g. a very soft clay layer) is present underneath the wall.

Limit equilibrium methods such as those developed by Janbu (1973) and Morgenstern & Price (1965) may be used to check overall stability. Detailed guidance on the use of such methods is given in the Geotechnical Manual for Slopes (GCO, 1984). The stability of slopes above and below the retaining wall should also be considered. The latter is particularly important where the loss of slope stability can lead to instability of the retaining wall.

The limitations of limit equilibrium analyses using the method of slices should be recognized. Experience has shown that for effective stress analyses of problems involving abrupt changes in the ground profile, as well as problems involving passive resistance, the results of analyses can be very sensitive to assumptions with regard to the line of action of the interslice forces, and their inclinations to the horizontal. In such cases, numerous runs need to be carried out to obtain reasonably accurate results. It is recommended that the interslice force inclinations in the passive zone of the trial failure surfaces should be chosen conservatively.

(2) *Sliding, Overturning and Bearing Capacity Failure*. Sliding failure involves outward translation of the retaining wall due to shearing failure along its base or along a soil surface below the base. Overturning failure involves rotation of the wall about its toe. Bearing capacity failure occurs when the ground bearing pressure (i.e. the intensity of loading imposed on the ground by the wall) exceeds the load-carrying capacity of the foundation.

The calculation models outlined in Figure 39 can be used for checking against these types of failures. These models assume that the gravity retaining wall acts as a monolith at limit state. Where discontinuities exist within the body of the wall (e.g. presence of a planar horizontal construction joint in a mass concrete wall), the possibility of sliding and overturning should be checked at the locations of such discontinuities.

For free-standing gravity retaining walls with compacted backfill, active earth pressures can be used in ultimate limit state calculations. It is not necessary to allow for compaction-induced lateral pressures in the calculations as movements which may occur close to ultimate limit states will reduce the earth pressures to those of the active condition.

Guidance on the determination of selected values of δ and δ_b to be used in design is

given in Sections 5.11 and 5.12 respectively.

For retaining walls with a stepped cross-section, active earth pressures should be calculated at the locations shown in Figure 40 and stability checks should be carried out at each change in section.

In the calculations, the lateral earth pressure and water pressure which act on the wall should be calculated to the bottom of the wall or of the blinding layer (if any), or in the case of a wall designed with a shear key, to a point below the bottom of the key where the assumed failure mechanism extends to that point.

Before taking the passive resistance of the soil in front of a wall into account in the calculations, consideration should be given as to whether the soil in the passive zone is likely to be eroded or removed by human activities in the future, e.g. excavation to lay services (see Section 3.8).

Mass concrete retaining walls are usually designed with a level base for ease of construction. However, crib walls and gabion walls sometimes have a tilted base to improve stability. For such cases, the alternative sliding failure mechanisms shown in Figure 41 should be considered in design. The sensitivity of the design to variations in the base tilt angle should be evaluated, taking into account the tolerances likely to be achieved in construction.

A small amount of back batter, e.g. 1 in 6 (10°), at the back face of the retaining wall (as indicated in Figure 41) can significantly reduce the earth pressure acting on the wall and even out ground bearing pressures. However, care must be taken to ensure that the wall itself will not be unstable before backfilling. Also, the back batter provided should not cause any hindrance to the compaction of backfill. For this reason, the batter is very rarely greater than 1 in 4.

Gravity retaining walls are sometimes designed with a shear key to improve sliding resistance, thus reducing the required base width of the wall. For such walls, the alternative sliding failure mechanisms shown in Figure 42 should be considered in design.

In selecting the location of the shear key, the object of maximizing the sliding resistance should be weighed against the risk of undermining the stability of the temporary excavation, which may occur if the key is positioned too close to the heel of the retaining wall.

The forces acting on a wall with a shear key are dependent on the stiffness of the wall and its key, as well as the stiffness of the foundation soil. In view of constructional difficulties and the likely large deformations, deep keys (i.e. keys with a depth greater than half the wall's base width) should not be used. In all cases, however, the shear key should be taken to a depth below the zone of disturbed soil in the foundation. Where a shear key is provided, a minimum depth of 0.5 m is recommended.

(3) *Hydraulic Failure*. Hydraulic failures of gravity retaining walls are rare. Reference should be made to Section 8.6 for further guidance.

9.2.4 Design Against Serviceability Limit States

The bulk of the movements of gravity retaining walls usually take place during construction. Such movements are dependent on the method and sequence of construction. Any additional movements subsequent to the completion of backfilling are usually due to changes in groundwater conditions, additional surcharge loadings or temperature effects.

For a gravity retaining wall founded on soil, in order to limit tilting the wall should be proportioned in such a way as to ensure the resultant force acts within the middle third of the wall base under the worst loading combination. As an alternative to the middle-third rule, appropriate deformation calculations should be carried out to demonstrate that movements in service will be within the serviceability limits specified for the design. In the latter case, it is good practice to limit the resultant force within the middle half of the wall base, even for relatively incompressible foundations. Where the middle-third or the middle-half rule is complied with, overturning failure does not need to be considered further. In applying the middle-third or the middle-half rule, the equations given in Figure 39 may be used to calculate the overturning and resisting moments, based on unfactored loadings and material parameters. Where the eccentricity e (Figure 39) is negative, i.e. the resultant load acts at a distance greater than B/2 from the toe of the foundation, the middle-third rule need not be applied.

For free-standing gravity retaining walls, active earth pressures may be assumed in applying the middle-third or middle-half rule. It is not necessary to consider compaction-induced lateral pressures.

Deformation calculations are rarely necessary for gravity retaining walls. Nevertheless, serviceability limit states should be specifically considered and appropriate checks (e.g. on tilting and settlement of the wall) should be carried out where the consequence of a serviceability limit state being reached is severe, or where movements could lead to distress in the wall. For example, walls which are designed to support sensitive services and walls which are subject to heavy surcharge deserve special consideration. The effects of consolidation settlement due to a compressible layer in the foundation also warrant attention.

If deformation calculations are required, at-rest earth pressure and compaction-induced lateral pressures should be considered unless movement of the wall is sufficient to mobilize active earth pressure.

The possibility of differential settlement along the length of a retaining wall should be considered where the foundation material is likely to be variable (e.g. filled ground) or where compressible spots exist.

9.3 MASS CONCRETE RETAINING WALLS

9.3.1 General

Mass concrete retaining walls are one of the simplest forms of retaining wall and are particularly suitable for retained heights of less than 3 m. While they can be designed for

greater heights, other types of retaining wall are usually more economical as the height increases.

The cross-sectional shape of a mass concrete wall can be affected by factors other than stability, such as construction methods, considerations of appearance and the use of space in front of the wall.

Economy in the use of materials can sometimes be achieved by stepping the back of the wall. However, the overall cost of construction should be considered in choosing an economical section for the wall. The use of simplified formwork and construction methods may effect a greater saving in time and cost than would economising materials in the wall's cross-section.

For walls with a nominally vertical front face, a back batter of approximately 1 in 50 should be provided at the wall face to avoid the optical illusion of the wall tilting forward. This batter may need to be increased for narrow walls or for walls built on relatively compressible foundations, where the tilt due to backfilling and post-construction loads can sometimes be greater.

9.3.2 Design

The guidance on design given in Section 9.2 is applicable to mass concrete retaining walls.

Structural design is seldom a major consideration. Reference should be made to Section 4.3.5 for general guidance.

9.3.3 Materials

(1) *Concrete.* Concrete of adequate durability should be specified. Where sulphates are present in soil and groundwater, reference may be made to BS 8110 : Part 1 (BSI, 1985) for guidance on the selection of cement type and mix proportions to ensure durability. In situations where the soil or groundwater is found to be particularly aggressive to concrete, adequate protection to the back of the wall should be provided. Examples of such protection include installation of a suitable form of lining (e.g. polyethylene sheeting) and application of a surface coating (e.g. using asphalt, chlorinated rubber, epoxy or polyurethane materials).

(2) *Backfill, Filter and Drainage Materials.* Guidance on the selection of these materials is given in Sections 3.2 and 3.3.

9.3.4 Construction Aspects

Vertical joints should be provided at intervals along the length of the retaining wall. The spacing of such joints depends on the expected temperature range and the shape of the wall. Suitable locations for joints are changes in level of the foundation or of the top of the wall. Joints should also be provided where the nature of the foundation changes, e.g. from

fill to insitu ground.

BS 8007 (BSI, 1987a) provides guidance on details of expansion, contraction and other movement joints, as well as their spacings. As a guide, expansion and contraction joints should be spaced not more than 22.5 m and 7.5 m apart respectively. Joints in decorative facings should coincide with the locations of movement joints.

The number of construction joints should be kept small and should be consistent with reasonable precautions against shrinkage. In a stepped-profile wall, horizontal construction joints should be provided at positions slightly above the steps. It is good practice to form a longitudinal groove at such joints to provide resistance against the shear force at the joint. Vertical construction joints should be provided at approximately 7.5 m centres and should coincide with contraction or expansion joints.

Where a shear key is provided to the base of the wall, the key should be cast blind against the side of the excavation, which should be protected as soon as it is exposed to avoid deterioration due to the effects of weather.

9.3.5   Drainage Provisions

The general guidance on drainage provisions given in Section 8.4 is applicable to mass concrete retaining walls. Figure 43 shows typical drainage schemes for this type of wall. Wherever possible, the inclined filter/drainage layer shown in Figure 43(a) should be provided. A less preferred alternative is to provide only a vertical or near-vertical filter/drainage layer immediately behind the wall. This alternative will need to be adopted where an inclined drain cannot be provided because of space constraints. In such cases, water pressures due to infiltration may have to be taken into account in design (see Section 8.3.2).

9.3.6   Aesthetics

The guidance on aesthetics given in Section 2.6 is applicable to mass concrete retaining walls.

The aesthetics of a mass concrete retaining wall may be improved by giving attention to the form and surface finish of the exposed face of the wall. Formwork with a moulded surface may be used to produce a ribbed, fluted or boxed surface, which should improve the appearance of the wall by breaking the monotony and reducing the impact of blemishes and discolouration. Treatment of the finished surface by bush-hammering or other exposed aggregate treatment should also be considered.

Careful attention should also be given to detailing. For example, the provision of a back batter of 1 in 50 or more to the front face of a retaining wall will help to avoid the optical illusion that vertical walls lean forward. Drainage holes (weepholes) should be properly positioned and detailed to minimize the visual impact of staining of the front face. For example, weepholes should be located at rebates where a ribbed feature finish is adopted.

Where appearance is important, a masonry facing or specialist units incorporating decorative art work should be considered.

9.4 CRIB WALLS

9.4.1 General

Crib walls are built up of individual prefabricated units assembled to form a series of crib-like structures containing suitable free-draining granular infill. The crib units together with the infill are designed to act together as a gravity retaining wall. Figure 44 shows cross-sections of typical crib walls.

In Hong Kong, precast reinforced concrete units are often used for building crib walls. Details of typical reinforced concrete crib wall systems are shown in Figure 45. Where units made of treated timber are used in permanent crib walls, the long-term durability of such units should be critically considered. Plain concrete units are not suitable for use in crib walls because of the ease of cracking.

The front face of a typical crib wall consists of a grid of units spaced at close intervals so that the infill does not spill through the units. Horizontal members of the grid are known as 'stretchers'. These are connected by transverse members known as 'headers' to a similar grid of stretchers parallel to the face, forming the back face of the crib wall. Where required, spacers may be used between stretchers at the front or back grids to provide additional support, and these are known as 'false headers' or 'pillow blocks'. Headers are generally perpendicular to the wall face, although some systems have variations to this.

The width of a crib wall is dictated by the length of standard precast concrete header units available. Except for walls used for landscaping, the minimum width of a crib wall should be 1.2 m. Low walls may be built vertical. Walls higher than 2 m are usually built to a back batter and with a tilted foundation to improve stability and even out ground bearing pressures. Common values of the batter are 1 in 10 (6°), 1 in 6 (10°) and 1 in 4 (14°). The height to which a single-cell wall can be easily constructed is about 5 m. For greater wall heights interlocking twin-cell crib walls can be used.

Crib walls built of precast concrete units are very sensitive to differential settlements and problems may arise for walls which are higher than about 7 m. They are generally not suitable to be used on ground which is liable to settle, nor should they be used for supporting heavy surcharge.

Crib walls are normally built in straight lengths. With the use of special units, curved walls can be constructed to a minimum radius of about 25 m on plan. Very careful planning is required to set out curved battered walls. Also, special joints and methods are necessary for bonding two crib walls together at corners in battered wall construction.

9.4.2 Design

(1) *Design Against Ultimate and Serviceability Limit States.* Crib walls should be

considered as gravity retaining walls for the purpose of design. The guidance given in Section 9.2 is applicable to crib walls.

The cross-section of a crib wall should be taken as the area enclosed by the back and front faces of the crib. The weight of the wall should be taken as the weight of the material comprising the crib, together with the weight of infill contained between the front and back faces.

Where the crib wall has a stepped cross-section, the stability of the wall should be checked at each change in section.

In order to limit deformation, crib walls should be proportioned in such a way that the resultant force acts within the middle third of the wall's cross-section.

Where the crib wall is founded on a relatively incompressible foundation, the mobilised angle of wall friction δ may be assumed to be equal to $\phi'/2$, where ϕ' is the angle of shearing resistance of the compacted backfill. Due to confinement, part of the weight of the infill is transferred onto the crib units through shear stresses. This arching effect results in large bearing stresses under the crib units at the base of the wall. If further settlement of the infill can occur after construction, it will cause arching of additional vertical stresses. Therefore, for the assumption of $\delta = \phi'/2$ to hold, it is important to adequately compact the infill material. Also, for soil foundations, a cast insitu reinforced concrete slab should be provided under the base of the crib wall to spread the bearing stresses.

(2) *Design of Crib Units*. The stretchers should be designed to resist bending caused by the horizontal earth pressure due to the weight of the infill and compaction. Header units should be designed as beams over their unsupported length, to carry a load equal to the weight of the superimposed filling. They should also be capable of transferring the tension induced by the horizontal earth pressure acting on the front stretchers to the back stretchers. It is usual to design the units for the maximum loading condition and to make all the units standard for use throughout the wall.

Earth pressure theories developed for grain silos or bins may be used to estimate the horizontal earth pressure due to the weight of the infill. Reference should be made to Schuster et al (1975) for details. Due to the limitations of theory, the crib units should be designed to resist forces due to at least twice the earth pressure computed using bin pressure theories. Compaction stresses may be estimated using the guidance given in Section 6.8. The New Zealand Ministry of Works and Development Specification CD 209 (New Zealand Ministry of Works and Development, 1980) specifies minimum bending, tensile and shear strengths for stretchers and headers. These strengths can serve as a useful guide to practical design. Reference should be made to Section 4.3.5 for general guidance on structural design.

Careful reinforcement detailing is required at the junctions between the crib units in order to ensure satisfactory transfer of forces. The bearing area of one unit resting on another should be sufficiently large to prevent crushing failure. For proper assembly, the units should be designed to interlock using recesses or dowels to give positive location during construction.

Crib walls with a convex front face in plan are much more susceptible to damage by

transverse deformations than are concave walls. Special care is required in designing and detailing the crib units for convex walls.

9.4.3 Materials

(1) *Concrete*. Precast concrete should comply with relevant specifications in Hong Kong.

(2) *Timber*. Timber should be of structural grade, properly treated, and should comply with relevant specifications on the use of structural timber in Hong Kong.

(3) *Infill Material*. The infill material should be a well-graded free-draining granular material, e.g. clean sand or crushed rock, with a maximum particle size of 75 mm. It should not contain more than 5% by weight of particles finer than 600 microns.

(4) *Backfill, Filter and Drainage Materials*. Guidance on the selection of these materials is given in Sections 3.2 and 3.3.

9.4.4 Construction Aspects

The New Zealand Ministry of Works and Development Specification CD 209 (New Zealand Ministry of Works and Development, 1980) gives useful specification clauses which may be used as model clauses for preparing a Particular Specification on crib walls.

Use of defective crib units can give rise to problems during construction, e.g. lack of fit and cracking and spalling of concrete. Therefore, dimensional tolerances and strength testing of the crib units should be included in the contract. Also, allowance should be made for the Engineer's staff to inspect the supplier's quality control during manufacture of the units, e.g. quality control testing of concrete strength and procedures for checking of concrete cover and steel reinforcements.

Where a reinforced concrete slab is provided at the base of a crib wall, its thickness should be at least 150 mm. The slab should extend 300 mm beyond the front and back faces of the crib. On sloping ground, the foundation should be stepped to follow the slope, the steps occurring at distances to suit the unit module lengths.

The crib should be filled to the top of each course of stretchers as the erection of the wall proceeds. Both the infill and backfill material should be properly compacted to prevent the development of voids and to avoid disturbing the alignment of the crib.

9.4.5 Drainage Provisions

The guidance on drainage provisions given in Section 9.3.5 is generally applicable to crib walls. A geotextile filter should be provided behind the rear face of the crib wall unless the infill material can serve as a filter to prevent migration of fines from the backfill.

9.4.6 Aesthetics

The guidance on aesthetics given in Section 2.6 is generally applicable to crib walls.

Economy in the use of crib units can be achieved by the adoption of open-faced walling (Figure 45(a)), in which the headers are simply supported on top of the stretcher course. The interspace can be planted with rock-garden-type vegetation which helps to blend the crib wall into the environment. In order to provide sufficient anchorage for vegetation, it is recommended that after construction of a crib wall, a suitable amount of top soil is exchanged with the infill material and rammed home so as to be keyed well in.

If closed-faced walling is adopted (Figure 45(b) and (c)), features can be built into the standard units to achieve an attractive design.

9.5 GABION WALLS

9.5.1 General

Gabion walls are made up of row upon row of orthogonal cages or baskets (gabions) which are filled with rock fragments and tied together. Their permeability and flexibility make them particularly suitable for use at sites which are liable to become saturated and where the foundation is composed of relatively compressible materials. Hence, gabion walls are widely used in river works. They are also used as retaining walls on dry land, especially in rugged terrain.

Gabion walls are relatively simple to construct. Where suitable rock is readily available, the use of gabion walls is particularly attractive for reasons of economy and speed of construction.

A variety of cage sizes can be produced using suitable materials to suit the terrain. The gabions are normally in modules of 2 m x 1 m x 1 m.

The basic shape of a gabion wall is trapezoidal, but the front and rear faces may be straight or stepped (Figure 46). A back batter (Figure 46(a)), commonly 1 in 10, 1 in 6 or 1 in 4, should be provided for walls higher than about 3 m to improve stability and even out ground bearing pressures.

For large walls with a cross-section wider than 4 m, economy can be achieved by adopting a cellular form of construction. This involves tying the front and rear faces of gabions by bulkheads, and filling the interior cells with rock. The size and shape of the cells should be proportioned to achieve internal stability. Counterforts or buttresses may be incorporated in the construction.

9.5.2 Design

(1) *Design Against Ultimate and Serviceability Limit States.* Gabion walls should be considered as gravity retaining walls for the purpose of design. The guidance given in

Section 9.2 is applicable to gabion walls.

The unit weight of the gabion wall material, γ_g, depends on the nature and porosity of the rock infill. The weight of the cage is small compared with that of the infill and is usually ignored in design. γ_g should be calculated using the following equation :

$$\gamma_g = (1 - n_r)G_s\gamma_w \quad \ldots \ldots \ldots \ldots \ldots \ldots (9.1)$$

where n_r = porosity of the rock infill
G_s = specific gravity of the rock
γ_w = unit weight of water

For preliminary design, G_s may be assumed to be 2.6 for fresh and slightly decomposed (i.e. grades I and II) rocks in Hong Kong. The porosity of the infill generally varies from 0.30 to 0.40, depending on the grading and angularity of the rock, as well as the method used to fill the gabions. Where the stability of the wall is sensitive to the value of γ_g, the value assumed in design should be stated on the drawings and be verified during construction.

Limit state checks should be carried out at selected planes through the gabion wall, ignoring the resistance contributed by the cage material and the connections between the cages. For stepped walls, stability checks should be carried out at each major change in section shape.

In order to limit deformation, gabion walls should be proportioned in such a way that the resultant force acts within the middle third of the wall's cross-section.

The mobilised angle of wall friction, δ, used in design should not exceed $\phi'/2$, where ϕ' is the angle of shearing resistance of the compacted backfill. In order for the assumption of $\delta = \phi'/2$ to hold, the gabion infill must be placed in such manner to achieve a dense mass which will not settle relative to the backfill after construction. Otherwise δ should be assumed to be zero. For a wall to be founded on relatively compressible materials, δ should also be assumed to be zero.

(2) *Design of the Gabion Units.* There is currently no universally accepted method for designing gabion units. For the purpose of estimating the forces induced in a gabion, it is suggested that active earth pressure is assumed to apply over the depth of the gabion. Such pressure may be evaluated using an approach similar to that adopted for reinforced fill structures (GCO, 1989a). The resistance to active earth pressure may then be assumed to be provided by the lid and base of the gabion unit.

It should be noted that the calculation approach described above should only be used as a rough guide to obtaining the required capacity of the gabion material. Where steel wire-mesh is used, an appropriate sacrificial thickness allowance should be provided to cater for possible corrosion. For gabions made of polymer grids, the effect of ultraviolet attack, the risk of fire and vandalism should be considered (see Section 9.5.3.(1)).

9.5.3 Materials

(1) *Gabion Materials.* Gabion baskets can be made from a range of materials. Nylon, polypropylene and polyethylene grids have been used. They have the advantage of being light weight and corrosion resistant. However, these materials are susceptible to attack by fire and ultraviolet light. There are instances of fire damage to gabion walls constructed from flammable polymer materials. Therefore, the risk of fire should be assessed in design. Where a polymer grid is adopted, the design detail should include a cover to the exposed grids with a non-flammable material, e.g. a fine soil which can retain moisture or sprayed concrete. Where this cannot be done, then an alternative non-flammable material should be used.

A material widely used in the commercial production of gabions is steel wire-mesh, of which there are two types, hexagonal woven and square welded.

Hexagonal woven wire-mesh is mechanically woven in a continuous sheet. The wires should be twisted together in pairs through three half turns, i.e. 'double-twisted', to form the mesh. The edges of the mesh should be selvaged with wires of a diameter of about 1.5 times that of the wire-mesh to prevent unravelling. The gabion base, top and sides should be formed from a single piece of mesh (Figure 47). The ends and diaphragms can be attached to this mesh by helical wires or other methods. The mesh can stretch or contract in two directions in its own plane and thus a rectangular wire-mesh basket filled with rock fragments can deform in any direction.

Welded wire-mesh is manufactured from steel wire electrically welded at each intersection. It does not have the ability to stretch and contract so that the baskets are less flexible than comparable woven wire-mesh gabions.

The wires used for the wire-mesh should be mild steel wire to BS 1052 (BSI, 1986b), with a minimum tensile strength of 350 N/mm^2. For permanent applications, the wires should be at least 2.7 mm in diameter and galvanized. For hexagonal wire-mesh the wires should be galvanized to BS 443 (BSI, 1990b) before weaving. For welded mesh, the mesh panels should be hot dip galvanized to BS 729 (BSI, 1986c) after welding. The making of panels with galvanized wires welded together is not recommended as the welds are left unprotected.

The soil and water with which the gabion wall is to be in contact should be assessed. Guidance on soil and water properties that are aggressive to galvanized steel is given in Geospec 2 : Model Specification for Reinforced Fill Structures (GCO, 1989a). If the soil and water conditions are aggressive, PVC (polyvinylchloride) coating should be provided to the wires. The PVC coating should be at least 0.5 mm thick and should meet the requirements of BS 4102 (BSI, 1990c). For hexagonal woven mesh, the PVC coating may be applied by hot dipping or by extrusion onto the galvanized wire before weaving. For welded mesh, it is usually applied electrostatically onto the panels. The PVC should be bonded sufficiently to the wire core to prevent capillary flow of water between the wire and the PVC, which can produce corrosion.

Binding wire, welded rings and internal bracing wires used for gabion construction should be of the same quality as the wires used for the wire-mesh. They should be at least

2.4 mm in diameter, galvanized, and where necessary PVC-coated.

Factory prefabrication will achieve better quality gabion units than fabrication on site from rolls or sheets of mesh.

Meshes made of chain-link, expanded metal and pig netting should not be used in permanent works. These materials have one or more of the following disadvantages :

    (a) tendency to unravel if one wire is broken,

    (b) difficulty in forming true rectangular shapes because of the absence
        of selvage wire, and

    (c) low resistance to corrosion.

In river works where the gabions may be subjected to abrasion by heavy water-borne material, suitable protective measures (e.g. rip-rap) should be incorporated in the design.

(2) *Infill Material.* Rock used for filling gabions should be sound, clean and well-graded. The maximum size of the rock should not exceed two-thirds the depth of the gabion to be filled or 300 mm, whichever is less. The preferred size is 150 mm to 300 mm. The smallest dimension of the rock should at least be twice the largest dimension of the mesh aperture.

(3) *Backfill, Filter and Drainage Materials.* Guidance on the selection of these materials is given in Sections 3.2 and 3.3. For a partially-submerged gabion wall, a free-draining granular backfill should be provided so that water pressure will not build up behind the wall when the water level in front of the wall is lowered.

9.5.4 Construction Aspects

For hexagonal woven mesh, each gabion is assembled by folding up the sides and lacing together all vertical edges with suitable binding wires. For welded mesh, panels of the mesh are cut to suit the dimensions of the sides, top and base of the baskets in separate pieces, which are then joined together with binding wires or welded rings.

The sides of hexagonal woven mesh gabions should be stretched taut before the gabions are filled. After filling the gabion boxes at one end, it is more convenient to lace as many empty boxes as possible before further filling. Horizontal internal bracing wires should be fitted between the outer and inner faces at about 300 mm centres in woven mesh gabions which are deeper than 500 mm.

Under-water gabions, by their nature, are usually pre-filled and placed into position by crane. In cases of difficult access, under-water gabions can be placed and filled by hand.

The gabions should be tightly filled to ensure minimum voids. Some overfilling is recommended to allow for subsequent settlement of the rock infill. This is particularly important for gabions made of welded mesh with a wire diameter of 3 mm or greater. Such

mesh is sufficiently rigid to support the gabion in the course above with little deflection and span across voids in the top of the course below. The rock in the successive courses may thus be prevented from interlocking so that a sliding failure may occur.

Sharp particles in crushed rocks may cause damage to the wire-mesh of gabions or its protective coating. Therefore, strict supervision is required to ensure that the technique used in filling is acceptable, so that no unforeseen durability problems will arise.

The lids of the gabions should meet the top edges of the sides and ends when closed, without leaving any gaps. The mesh of the lids should be tied down to the tops of any diaphragms provided, as well as to the tops of the sides and ends. Whenever possible, the vertical joints between the units should be staggered in adjacent courses, to prevent the formation of weak vertical shear planes through the gabion wall and to give a better, more integral appearance.

Curves and angles in the face of the gabion wall may be formed by cutting and folding the wire-mesh to make units of special shapes.

The above guidance is also generally applicable to the construction of gabions using polymer grids. Polymer braids are usually used to form the gabions and to connect adjacent units. Special care is required to ensure that the grids are not damaged during construction.

9.5.5 Drainage Provisions

The guidance on drainage provisions given in Section 9.3.5 is generally applicable to gabion walls.

A geotextile filter should be provided behind the rear face of the gabion wall to prevent migration of fines from the backfill into the coarse rock infill. Drainage layers at the rear face are normally not warranted. However, a drainage layer of adequate permeability should be provided at the base of the wall to guard against erosion of the foundation material.

The high permeability of the gabion units will permit direct infiltration through the body of the wall at times of heavy rainfall. In order to minimize the possibility of saturation and erosion of the foundation material under a non-submerged gabion wall, it is good practice to provide a blinding layer with adequate drainage provisions at the level of the foundation.

For submerged gabion walls, appropriate measures should be incorporated to prevent scouring and erosion of the foundation.

9.5.6 Aesthetics

Gabion walls can easily blend in with the natural landscape, especially where suitable rock is available locally for use as infill.

10. REINFORCED CONCRETE RETAINING WALLS

10.1 GENERAL

This Chapter gives general guidance on the design of reinforced concrete retaining walls and deals with selected design and construction aspects of such walls.

A reinforced concrete retaining wall resists bending due to earth pressures from the backfill, which provides part of the stabilizing weight by resting on the base slab and thereby acts together with the wall as a semi-gravity structure.

The following are the main types of wall (Figure 48) :

(a) L- or inverted T-shaped cantilever retaining walls, which have a vertical or inclined slab monolithic with a base slab,

(b) counterfort retaining walls, which have a vertical or inclined slab supported by counterforts monolithic with the back of the wall slab and base slab, and

(c) buttressed retaining walls, which have a vertical or inclined slab supported by buttresses monolithic with the front of the wall slab and base slab.

A shear key is sometimes provided below the base slab of the retaining wall to improve sliding resistance.

A cantilever wall is generally economical for retained heights of up to about 8 m. For greater heights, the thickness of the stem of the cantilever wall becomes excessive, and a counterfort retaining wall is more appropriate. The buttressed retaining wall is seldom used.

With all types of wall, the illusion of the retaining wall tilting forward can be avoided by providing a batter of approximately 1 in 50 to the front face.

10.2 DESIGN

The guidance on design given in Section 9.2 for gravity retaining walls is applicable to reinforced concrete retaining walls.

The various elements of design for reinforced concrete retaining walls are summarised in Table 20.

For a free-standing retaining wall, the wall together with the backfill up to a vertical plane above its heel (i.e. the virtual back) can be treated as a monolithic block for the purpose of checking against sliding, overturning and bearing capacity failures, and for applying the middle-third rule. Active earth pressures may be assumed in the calculations.

The wall stem and base slab of a reinforced concrete retaining wall should be designed to resist the bending moments and shear forces due to pressures acting on the wall. In the structural design, at-rest and compaction-induced earth pressures should be assumed, unless deformation calculations show that there will be sufficient movement to reduce these pressures. In order to prevent large lateral pressures from being exerted on the wall stem, it is worthwhile to limit the compaction loading behind the wall (see Section 6.8).

For L- and inverted T-shaped retaining walls, the wall stem should be designed as a cantilever. A tapered wall stem is sometimes provided to reduce the volume of concrete required.

For counterfort walls, the counterforts should be designed as cantilevers of a T-shaped cross-section and the wall stem should be designed as a continuous slab. The design should transfer the earth pressure and water pressure which act on the wall slabs to the counterforts.

Buttressed walls should be designed in a manner similar to counterfort walls.

Where a long length of wall of the same cross section is required, consideration should be given to precasting standard wall units for the project. In such cases, the foundation requirements peculiar to each section should be taken into account by designing appropriate cast insitu bases with holding down bolts or piles, if necessary.

For restrained rigid retaining walls, e.g. screen (basement) walls, abutment walls of a portal structure and R.C. counterfort or buttressed retaining walls on rock or piled foundations, the restraints themselves generally provide adequate stability for the walls, which then only need to be designed to have adequate structural strength. At-rest and compaction-induced earth pressures should be assumed in the calculations. The construction sequence of the support structures and of the wall should be taken into account in the design. If backfilling is permitted before the restraints are in place, the wall should be designed for the free-standing state. The assumptions made in the design should be clearly shown on the drawings.

10.3 MATERIALS

10.3.1 Reinforced Concrete

Concrete of adequate durability should be specified and the guidance given in Section 9.3.3(1) is also applicable to reinforced concrete retaining walls.

Steel reinforcement should comply with relevant material standards set in Hong Kong. Adequate cover should be provided to the reinforcement, especially those behind surfaces which are in contact with soil.

10.3.2 Backfill, Filter and Drainage Materials

Guidance on the selection of these materials is given in Sections 3.2 and 3.3.

10.4 CONSTRUCTION ASPECTS

The guidance on joints and shear keys for mass concrete retaining walls given in Section 9.3.4 is generally applicable to reinforced concrete retaining walls.

The positions of construction joints should be specified at the design stage. Generally, to facilitate construction there should be a joint between the base (or splay) and the wall stem. Additional horizontal joints should be provided in the wall stem to suit the lifts of the formwork. Where appropriate, vertical joints should be positioned at points of minimum shear stress in the concrete. Reinforcement should pass through the construction joints.

10.5 DRAINAGE PROVISIONS

The general guidance on drainage provisions given in Section 8.4 is applicable to reinforced concrete retaining walls. Figure 49 shows typical drainage schemes for this type of wall. Wherever possible, the inclined filter/drainage layer shown in Figure 49(a) should be provided. A less preferred alternative is to provide only a vertical or near-vertical filter/drainage layer behind the wall stem. This will need to be adopted where an inclined drain cannot be provided because of space constraints. In such cases, water pressures due to infiltration may have to be taken into account in design (see Section 8.3.2)

10.6 AESTHETICS

The guidance given in Section 9.3.6 is also applicable to reinforced concrete retaining walls.

Where non-structural facing panels are used to improve appearance, particular care should be taken in the design and detailing of connections so that differential stresses are not induced by the connections between the facing and the wall structure.

Careful detailing is required to avoid unsightly blemishes on the exposed concrete surface due to rain water and water-borne detritus. Particular attention should be given to areas where water may accumulate, e.g. weepholes. Appropriate 'drips' should be provided at the coping of the wall.

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11. CANTILEVERED RETAINING WALLS

11.1 GENERAL

The main types of cantilevered retaining walls are sheet pile walls, soldier pile walls, grouted pile walls, diaphragm walls and bored pile walls. In Hong Kong, piles for cantilevered retaining walls are often hand excavated and formed with cast insitu reinforced concrete. These are known as 'caissons'. Apart from bored pile (or caisson) walls, the other wall types are usually used to provide temporary support to excavations. Diaphragm walls and walls formed by closely-spaced reinforced concrete piles are often incorporated into the permanent works subsequently.

This Chapter gives general guidance on the design of cantilevered retaining walls. Selected design and construction aspects of caisson walls are also covered. For other wall types, useful information is given in GCO Publication No. 1/90 : Review of Design Methods for Excavations (GCO, 1990).

11.2 METHODS OF DESIGN

11.2.1 General Principles

A cantilevered retaining wall derives its support from the passive resistance developed in the soil or rock in front of the wall. The guidance on general principles of design given in Section 9.2.1 is applicable to cantilevered retaining walls.

11.2.2 Limit States

A list of limit states to be considered in design should be compiled in accordance with the principles given in Section 4.1. As a minimum, the following ultimate limit states should be considered :

    (a) loss of overall stability, and

    (b) overturning failure.

Where appropriate, the possibility of hydraulic failure should be checked. Also, serviceability limit states should be considered to ensure that movement of the cantilevered retaining wall will not be excessive.

Guidance on the partial factors of safety to be used in design against ultimate and serviceability limit states is given in Section 4.3.4.

In addition to the above, limit states related to structural design should be considered. Some general guidance on structural design is given in Section 11.2.5, as well as in Section 11.3.2, which deals specifically with caisson wall design.

11.2.3 Design Against Ultimate Limit States

(1) *Loss of Overall Stability*. The guidance given in Section 9.2.3 (1) is applicable to cantilevered retaining walls.

(2) *Overturning Failure for Walls Embedded Entirely in Soil*. Overturning failure involves the collapse of the retaining wall by rotation about a point in the embedded portion of the wall (Figure 50(a)). This can be prevented by providing an adequate depth of embedment over the length of the wall.

The theoretical pressure distribution for a cantilevered retaining wall at limiting equilibrium condition is of the fixed-earth form, as shown in Figure 50(b). This assumes the mobilisation of active and full passive earth pressures. The depth of embedment corresponding to this pressure distribution may be calculated directly using the equilibrium equations. However, the solution of these equations is algebraically complex. Hence the simplified pressure distribution illustrated in Figure 50(c) is often used. By taking moments about the 'toe', the depth of embedment in the simplified model can be found. Because of the simplification, the depth of embedment obtained is less than the value obtained from the full method. To compensate for this discrepancy, the calculated depth of embedment, d_o, should be increased by 20%.

One point to note regarding the application of the simplified model concerns the assumption about water pressures. Because the design embedment depth is greater than that adopted for the limit equilibrium calculation, the seepage path around the simplified model differs from the seepage path corresponding to actual conditions. However, the error due to this discrepancy is generally small and may be ignored in design (Potts & Burland, 1983).

In the above calculations, the depth of soil which can be relied upon to provide passive resistance should be carefully assessed. The possibility of the soil in the passive zone to be eroded or removed by human activities in the future should be considered. Where excavation in front of the wall is possible, e.g. to lay services, the presence of a trench at least 1 m deep should be considered in design (see Section 3.8).

(3) *Overturning Failure for Walls Socketed in Rock*. A rock socket is that part of the wall which is embedded entirely in moderately decomposed or better rock or, for ground with a corestone-bearing weathering profile, in a partially weathered rock mass with an interlocked structure (viz. a PW 50/90 zone (GCO, 1988)) or better. For cantilevered retaining walls socketed in rock, overturning failure involves passive failure in the soil (if present) as well as failure of the rock socket. The design should provide an adequate rock socket depth over the length of the wall. The simplified model described in Section 11.2.3(2) above and the '20% rule' are inappropriate for rock-socketed walls and therefore should not be used. Instead, the ultimate bending moment and shear force at the top of the rock socket should first be calculated based on factored loadings and material properties, assuming that active and full passive pressures are mobilised. The rock socket should then be designed to take these internal forces to prevent the ultimate limit state of overturning failure from being reached.

Two possible modes of failure, viz. bearing failure of the rock mass and failure by movement along preferentially oriented discontinuities (discontinuity-controlled failure),

should be considered in the design of rock sockets. Generally, it is very difficult to establish the top of the PW 50/90 zone (GCO, 1988) at the design stage. Therefore, designs should allow for a range of depths to this zone, and the rock socket depth to be provided should be determined based on actual ground conditions encountered during construction.

The idealised pressure distributions and design equations given in Figures 51, 52 and 53 may be used to determine the minimum rock socket depth required to prevent bearing failure. In using the equations in these figures, the ultimate lateral bearing capacity of the rock q_u needs to be determined. This may be assessed from a consideration of the rock mass quality including the strength of the rock, structure in the rock mass, and nature, orientation, spacing, roughness, aperture size, infilling and seepage of discontinuities. However, an accurate assessment is often difficult to achieve in practice. In the absence of a reliable assessment, a presumed lateral bearing pressure q_a may be used in design. Greenway et al (1986b) have adopted a q_a value of 2 MPa in the design of a caisson wall socketed in slightly decomposed granite and they considered the value conservative. It should be noted that q_a can be used directly in the design equations in place of q_u without factoring.

Where the top of a rock socket is dipping steeply, consideration should be given to the use of different q_a values in front of and behind the wall. If different q_a values are used, the design equations should be modified accordingly, based on a consideration of force and moment equilibrium of the rock socket.

For the design of rock sockets against a planar discontinuity-controlled failure, the calculation model given in Figure 54(a) may be used to determine the ultimate lateral resistance F_1 ($= R \cos \delta_2$) offered by the rock mass. Passive earth pressure may be assumed to act on surfaces AE and CD, with zero 'wall friction' mobilised at the latter surface. Factored material parameters should be used in the calculation of R.

F_1 is a function of the depth of soil cover in the passive zone d_s, the slope of the top of rock socket β, the dip θ and the angle of shearing resistance ϕ' of the presumed discontinuity OC, and the distance d_1. For a given d_s, β and ϕ', the relationship between F_1, θ and d_1 can be found by using the model given in Figure 54(a). Figure 54(b) shows that for a particular value of d_1, F_1 reaches a minimum at an intermediate value of θ.

In view of the uncertainties associated with the determination of d_1 and θ, the following design approach is recommended. First, the relationship between the minimum F_1 value and d_1 should be established. Based on this relationship and the design equations given in Figure 54(d), d_1 and F_1 values required for achieving equilibrium with the applied bending moment and shear force, and the design rock socket depth d_r, can then be obtained.

In Figure 54(d), two equations for the evaluation of d_1 have been given. The first equation considers full force and moment equilibrium, but to find d_1 using this equation involves tedious iterative calculations. The second equation ignores the stabilizing moment provided by the reaction force F_2. This results in a more conservative design, but the calculations are simpler. The design rock socket depth calculated using the latter equation can be up to 30% larger than that calculated using the first equation.

In using the equations given in Figure 54(d), the value of z can be evaluated based on the relationship between the minimum F_1 values and the corresponding d_1 values for the

successively increasing depths of the presumed discontinuity OC. This process is akin to that used to evaluate an active earth pressure distribution as shown in Figure 25. Values of z/d_1 calculated by this method generally lie between 0.55 and 0.65.

Where no adverse discontinuities are encountered in the ground investigation, the provision of a rock socket depth adequate to prevent bearing failure only may be considered. Where adverse discontinuities are recorded, the rock socket depth should be the larger of the two values required to prevent bearing failure and discontinuity-controlled failure respectively. Information about the rock mass is often insufficient at the design stage to confirm whether adverse discontinuities exist on site. A recommended approach for the determination of rock socket depth is given in Section 11.3.4.

One point worth noting in the design of walls socketed in rock relates to the depth of soil cover in the passive zone, d_s. If d_s is large enough, the ultimate shear force V at the top of the rock socket will reverse in direction (Figure 52). If d_s is increased further, there will be a point where the d_1 value will be reduced to zero, irrespective of whether bearing failure or discontinuity-controlled failure is involved. This point is reached when $q_u M/V^2$ = 0.5. For a greater depth of soil cover, the simplified rectangular pressure distribution and the equation given in Figure 53 may be used to determine the design rock socket depth. In such a case, only bearing failure needs to be considered.

The use of the simplified rectangular pressure distribution in Figure 53 cannot ensure moment equilibrium. However, it can be easily shown that the restoring moment is always greater than the overturning moment, and hence there will be some reserve in the design. Computation using a more refined model will yield a rock socket depth only slightly different from that obtained using the simplified distribution. Given the uncertainties involved in the analysis, this procedure is usually not warranted.

In order to allow for the ubiquitous local variations in the upper level of moderately decomposed rock, or of the PW 50/90 zone (GCO, 1988) in a corestone-bearing weathering profile, it is recommended that a minimum rock socket depth of 1 m should be provided. However, the total depth of embedment of the retaining wall, $d_s + d_r$, need not be greater than the penetration depth required for a wall embedded entirely in soil.

(4) *Hydraulic Failure*. Reference should be made to Section 8.6 for guidance on design against hydraulic failure.

11.2.4 Design Against Serviceability Limit States

The permissible movements to be specified in a design should be based on a consideration of the tolerable movement of nearby structures, services and land. As a guide, the outward movement at the top of a cantilevered retaining wall under the most unfavourable loading condition should generally not exceed 1% of the retained height. More stringent requirements may be necessary where structures or services are present behind the wall.

For cantilevered retaining walls embedded entirely in soil, the magnitude of the outward movement is a function of the height of soil retained, the nature of the soil and the groundwater conditions. Movement can be significant where the retained height is large, in

which case the requirement to limit movement can be a major design consideration. For walls which are socketed in rock, the magnitude of the outward movement depends also on the depth of soil cover in the passive zone. Where the amount of rock in the rock socket is close to the lower limit of 50%, the deformation of the wall could be significant. In such case, the stiffness of the ground in the region of the PW 50/90 zone (GCO, 1988) should be reduced to that of the soil matrix for the purpose of estimating wall deformation.

The possibility of the soil on the passive side of the wall being eroded or removed by human activities in the future, e.g. to lay services, should be considered (see Section 3.8).

Where appropriate, a soil-structure interaction analysis should be carried out to estimate the deformation of a cantilevered retaining wall (see Section 6.9). Figure 55 shows the Winkler model for the soil and the earth pressure distributions which can be used for such an analysis. The analysis should be based on unfactored loadings and material parameters, and for walls socketed in rock, it should be carried out for a range of depths to rock socket.

The likely stress paths of soil elements near the wall and the effects of stress changes in the soil due to wall installation and bulk excavation in front of the wall should be considered in design (see Section 6.2.2 and Figure 14). Guidance on the selection of soil and rock deformation parameters is given in Sections 5.6 and 5.9 respectively.

11.2.5   Structural Design

Reference should be made to Section 4.3.5 for general guidance on structural design.

For cantilevered retaining walls embedded entirely in soil, the simplified model shown in Figure 50(c) can be used to derive the bending moment and shear force distributions for both ultimate and serviceability structural design. It should be noted that this model ignores the additional 20% depth of embedment required for stability against overturning failure (see Section 11.2.3(2)). Therefore, for walls constructed of reinforced concrete, the reinforcements should not be curtailed at the point where the calculated bending moment is zero. They should be provided down to the bottom of the wall, and on both faces, to allow for small reverse bending moments which may occur near the toe.

For walls socketed in rock, a soil-structure interaction analysis, such as the Winkler model shown in Figure 55, should be carried out to determine the bending moment and shear force distributions (see Section 6.9). The analysis should be carried out for a range of depths to rock socket. The use of the simplified pressure distributions given in Figures 51 to 53 may yield values of maximum bending moment and shear force lower than the actual condition. The difference may be large for cases where the soil cover in the passive zone is thick. Therefore, the simplified model should not be applied in structural design for these cases. Where the soil cover is thin (less than about 3 m), the error in the maximum bending moment and shear force is not significant.

11.3 BORED PILE AND CAISSON WALLS

11.3.1 General

Cantilevered retaining walls in Hong Kong are sometimes formed by constructing a row of bored piles. In steep terrain or where access is difficult for construction plant and equipment, hand-excavated caissons are often used in place of machine bored piles. The walls formed by this method are referred to as 'caisson walls'. The popularity of caissons is now decreasing owing to health and safety concerns. The guidance on design of bored pile walls given in the remaining part of this section and in Section 11.3.2 is applicable to caisson walls.

In bored pile (or caisson) wall construction, the piles can either be contiguous, or they can be closely spaced with infilled concrete panels in between. The spacing between the piles should generally be not more than three times the pile diameter, otherwise special support and protective measures should be provided to retain the soil exposed by excavation. In some cases (e.g. walls retaining existing fill), it may be necessary to carry out precautionary works to prevent local failure even where the design pile spacing is less than three times the pile diameter.

The rotational movement of a bored pile (or caisson) wall embedded entirely in soil can be substantial. As such, bored pile (or caisson) walls with a large retained height may not be feasible, unless moderately decomposed or better rock or, for ground with a corestone-bearing weathering profile, a PW 50/90 zone (GCO, 1988) is found at a relatively shallow depth at the site to permit the wall to be designed as a rock-socketed wall. Bored pile and caisson walls are economical only for moderate wall heights, as the required size (section modulus) of the piles and the cost of rock excavation (for rock-socketed walls) increase rapidly with increase in retained height.

11.3.2 Design

The guidance on design given in Section 11.2 is generally applicable to bored pile and caisson walls. It is important that proper account is taken of the pile spacing and diameter in calculating the forces on the 'active' and 'passive' sides, as well as in calculating the bending moments and shear forces in the piles.

For ultimate limit state checks, the following approach is recommended. Where the design pile spacing S is less than or equal to 3D, where D is the pile diameter, the active and passive earth pressures can be assumed to act over the full pile spacing. Where S is greater than 3D, the active force for the retained height above excavation level should still be calculated over the full pile spacing S. However, both the active and passive earth pressures below excavation level should be assumed to act over an effective width equal to 3D only. The remaining part of the spacing, i.e. a width equal to S - 3D, should be assumed to have equal (at-rest) earth pressures on both sides.

Where the bored pile (or caisson) wall is used to retain a weathered rock mass, consideration should be given to the characteristics of the rock mass in design (e.g. relict structures and discontinuities, see Sections 5.5.4 and 5.8.3). In such cases, the applicability

of classical theories for calculating active and passive earth pressures is questionable. Where persistent adversely-oriented discontinuities are suspected, pressures which may arise due to sliding along such planes within the rock mass should be assessed by considering the limiting equilibrium of potential failure wedges (see, e.g. Whiteside (1986)). Where the rock material has weathered to a soil, the active force derived from the wedge analysis should be compared with that obtained from classical earth pressure theories, and the larger force should be used in design. For material not weathered to a soil, the sliding rock wedge will move as a rigid body. In such cases, compatibility of movement between the rock wedge and the retaining wall will need to be considered in order to assess the line of action of the active force. The possibility of concentrated loading on the wall should be avoided by careful detailing and construction. For a closely-jointed rock mass, the modified Hoek-Brown failure criterion may be used to determine the active and passive pressures if the rock mass is such that it can be treated as homogeneous and isotropic (see Section 5.8.3).

For rock-socketed bored pile (or caisson) walls, both bearing failure of the rock mass and discontinuity-controlled failure should be considered. In checking against bearing failure, the bearing pressure within the rock socket depth should be calculated by assuming no lateral stress dispersion, i.e. bearing over the diameter of the pile only. Further guidance is given in Figures 51 to 53.

Two possible modes of discontinuity-controlled failure which can occur in a rock mass are shown in Figure 56. It should be noted that for planar failure to take place, there needs to be a vertical relief joint, cutting across the rock in between the piles, along the length of the wall. In design, such a joint is often conservatively assumed to be present and the method outlined in Figure 54 can be used to calculate the ultimate lateral resistance offered by the rock mass. The 'angle of wall friction' along the presumed vertical joint can be assumed to have a value equal to the angle of shearing resistance of a typical joint in the rock.

Care is required in evaluating the values of X_p and P_p for the wedge analysis illustrated in Figure 54. For all cases, X_p should be based on passive pressure acting over a width equal to S (with zero 'wall friction'). Where $S \leq 3D$, P_p should be based on passive pressure acting over the same width. However, if $S > 3D$, P_p should be based on a combination of passive and at-rest pressures over widths of 3D and S - 3D respectively.

The wedge failure shown in Figure 56(b) affects only individual piles. To minimise the chance of this mechanism occurring, a capping beam should be provided across groups of piles to make them act integrally. For cases where wedge failures can be clearly identified, a detailed analysis should be carried out to assess the rock socket capacity. Hoek & Bray (1981) give appropriate guidance on wedge analysis.

In carrying out soil-structure interaction analyses using the Winkler model for serviceability checks and structural design (see Sections 11.2.4 and 11.2.5), the following approach is recommended. Initially, active earth pressures should be applied to the 'active' side. On the 'passive' side, the locked-in horizontal stresses, which should be evaluated by taking into account the effects of wall installation and bulk excavation, should be applied. The resulting wall movement should then be checked to see whether it is large enough to result in active earth pressures over the full height of the wall. If not, an appropriate earth pressure distribution on the active side, with at-rest pressures at the lower portion of the

wall, should be used and the analysis repeated. A few iterations may be required to achieve compatibility between wall movement and earth pressure. In each iteration, it is necessary to check that the locked-in horizontal stresses plus the reactive pressures due to the forces in the passive springs do not exceed the passive earth pressures (see Section 6.9).

Due to the difficulties in establishing the depth of the PW 50/90 zone (GCO, 1988) at the design stage, the calculation of rock socket depth, bending moments, shear forces and deformations should be carried out for a range of depths to this zone. Different rock socket depths and reinforcement details should be shown on the drawings for the different possible cases anticipated. The assumed rock mass quality at the rock socket should also be stated. The design to be adopted should be based on a consideration of the actual ground conditions encountered during construction (see Section 11.3.4).

11.3.3 Materials

(1) *Reinforced Concrete*. The guidance given in Section 10.3.1 is also applicable to bored pile and caisson walls.

(2) *Filter and Drainage Materials*. Guidance on the selection of these materials is given in Section 3.3.

11.3.4 Construction Aspects

(1) *Hand-dug Caissons*. Caissons are usually excavated in stages of about one metre depth. Each stage of excavation is lined with insitu concrete using a tapered steel shutter suitably supported and designed for ease of striking. The concrete lining should be at least 75 mm thick. The shutter should remain in place to provide support for the fresh concrete and surrounding ground while the next stage of excavation proceeds. Where necessary, submersible electric pumps should be used for dewatering at the base of the caissons.

Precautionary measures to ensure adequate safety during caisson construction are very important. The Guidance Notes on Hand-dug Caissons (HKIE, 1981) deal with the safety and technical aspects of hand-dug caissons.

Base heave failures have been experienced with hand-dug caissons in Hong Kong (Chan, 1987). Heave results from hydraulic forces and is preceded by loss of strength of residual soils and saprolites. The possibility of base heave renders the construction of deep hand-dug caissons extremely hazardous, particularly when carried out below the groundwater table. Therefore, suitable dewatering or groundwater cut-off, and/or ground strengthening measures should be carried out. Some guidance on design against base heave failures can be found in GCO Publication No. 1/90 : Review of Design Methods for Excavations (GCO, 1990).

Groundwater drawdown for caisson excavation can result in ground settlement and damage to adjacent structures. Therefore, an assessment of the effects of caisson works on nearby structures, services and land should be carried out, taking into account the method and sequence of works. Precautionary measures, such as the provision of grout curtains

behind caissons to reduce the effect of drawdown, should be carried out if unfavourable ground and groundwater conditions are expected. Shirlaw (1987) gives guidance on the choice of grouts for hand-dug caisson construction.

For caissons socketed in rock, the cost of excavating rock and concreting can be significant. The following approach for determining the rock socket depth and reinforcement requirements is recommended.

A small number of widely-spaced caissons should first be selected for construction, with the aim of evaluating the variations in rock levels and rock mass quality at the site. These caissons should be excavated progressively to a level sufficiently deep to prevent both bearing failure of the rock mass and discontinuity-controlled failure. In order to facilitate mapping of the exposures as excavation proceeds, excavation in rock should be carried out in stages of not more than about 1.5 m deep.

Excavation of the remaining caissons should initially be taken to a level which is sufficiently deep to prevent bearing failure of the rock mass, mapping the exposures as excavation proceeds. If on the basis of this mapping and the ground conditions encountered in the selected caissons it can be judged that the presence of adverse discontinuities in the rock mass is unlikely, then excavation can be stopped and the caissons concreted.

In some cases, e.g. in constructing large diameter caissons where conservative assumptions of the rock mass quality can result in significant cost implications, additional ground investigation in advance of caisson excavation should be considered. Such investigations should include high quality rotary core drilling, impression packer testing and permeability testing in rock, at the locations of the proposed caissons. Accurate information on rock levels and rock mass quality can permit more realistic designs to be achieved, with possibilities of significant cost savings.

Vertical joints should be provided at suitable intervals along the length of the capping beam and should coincide with joints provided along the concrete facing panel. The guidance on joints for mass concrete retaining walls given in Section 9.3.4 is generally applicable to caisson walls.

(2) *Large Diameter Machine Bored Piles*. Large diameter machine bored piles are commonly excavated by means of a reverse-circulation drill, a rotary auger or a rotary drilling bucket, under a bentonite slurry which provides support to the sides of the shaft. Bored piles are also excavated by grabs and chisels within a steel casing, which is advanced progressively to refusal with the use of an oscillator, an electrical or hydraulic vibrator, or a pneumatically-powered swinghead. Where excavation is to be carried out beyond the casing, the excavation should be supported by an excess water head or bentonite slurry.

Tight control should the exercised on the properties of the bentonite slurry. Reference should be made to Table 8.2 of the General Specification for Civil Engineering Works (Hong Kong Government, 1992) for typical specifications for permissible limits on the properties of bentonite slurry.

Local collapse of the side wall of a shaft, which results in overbreak, is not uncommon in bored pile construction. Major instability problems such as global collapse of

the shaft and base heave have also been experienced in Hong Kong. In order to ensure stability of the shaft, the hydrostatic pressure of the bentonite slurry should be controlled so that it is always greater than the sum of the water and earth pressures combined, with due allowance given for arching effects. Alternatively, a steel casing may be advanced to beyond the excavation level to provide support.

For machine bored pile walls socketed in rock, the same approach for determining the rock socket depth and reinforcement requirements as for caisson walls is recommended. In order to evaluate the rock mass quality at the site, the rock sockets in selected bored piles should be closely examined. Adequate precautionary measures to ensure safety of inspection personnel descending the piles are essential. Reference should be made to the Guidance Notes on Hand-Dug Caissons (HKIE, 1981) for the safety aspects.

Where dewatering is carried out, the effect of groundwater drawdown on nearby structures, services and land should be assessed. If necessary, precautionary measures, such as the provision of grout curtains behind the bored pile wall to reduce the effect of drawdown, should be carried out.

11.3.5 Drainage Provisions

The general guidance on drainage provisions given in Section 8.4 is applicable to bored pile and caisson walls. Figure 57 shows a number of drainage arrangements. Other variations are also possible.

Type I and II drainage arrangements are suitable for widely-spaced piles while type III, IV and V are appropriate for closely-spaced piles. Type I suffers from the drawback that temporary support is required for the installation of the filter. A minimum filter thickness of 450 mm should be provided in order to facilitate compaction of the filter material. In the type II arrangement, a reasonably smooth soil surface should be formed for proper installation of the prefabricated drainage composite. Careful design and detailing is required for type III, IV and V arrangements.

Due to construction difficulties, it is hard to ensure that the drainage arrangement behind a bored pile (or caisson) wall can remain effective and provide the full design drainage capacity under continuous seepage conditions. Therefore, the possibility of 'damming' of the groundwater should be considered and the corresponding rise in groundwater level carefully assessed (see Section 8.3.3).

In cases where a bored pile (or caisson) wall is required to be designed for steady state seepage pressures arising from infiltration, the drainage provision can be assumed to be able to remain effective throughout the design life of the wall, provided that continuous ponding behind the wall is effectively prevented to remove the possibility of uninterrupted seepage. The simple chart given in Figure 30(c) can be adopted for the evaluation of the effects of infiltration for a bored pile (or caisson) wall.

11.3.6 Aesthetics

A retaining wall composed of exposed piles often has a rough and irregular appearance. Therefore, a reinforced concrete facing is usually provided to hide the piles. The guidance given in Section 9.3.6 is applicable to bored pile and caisson walls with a concrete facing. Where appearance is important, decorative art work may be incorporated in the facing.

Care should be taken in the design and detailing of connections so that differential stresses are not induced by the connections between the facing and the wall structure.

Careful detailing is required to avoid unsightly blemishes on the concrete facing due to rain water and water-borne detritus. Particular attention should be given to areas where water may accumulate, e.g. weepholes. Appropriate 'drips' should be provided at the coping of the wall.

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12. SUPERVISION AND MONITORING

12.1 GENERAL

It is very important that the designer is adequately represented on site during construction. It is insufficient to rely wholly on the contractor's obligations under the contract to guarantee that the quality of the permanent works and safety of the temporary works meets the designer's intentions. The designer should incorporate into the contract documents sufficient control measures to ensure that during construction there will be adequate geotechnical supervision commensurate with the size and complexity of the particular project.

Experienced site staff, good project management and the implementation of quality assurance procedures are all essential to ensure quality in construction. Site staff should be familiar with the ground conditions assumed in the design and should inform the designer if the actual conditions are found to vary significantly from those assumed.

Only those aspects of construction control which are directly related to verification of the geotechnical design assumptions are covered in this Chapter. Detailed guidance on the general duties of site staff, project management and quality assurance principles are outside the scope of this Geoguide. For these aspects, reference should be made to specialist literature such as the ASCE Manual of Professional Practice : Quality in the Constructed Project (ASCE, 1988) and British Standards in the BS 5750 series (BSI, 1987b).

12.2 SUPERVISION AND ITS ROLE IN DESIGN

Proper supervision is essential to ensure the safety and quality of a retaining wall and associated works.

The most important aspects of supervision during construction are :

(a) checking to ensure that the retaining wall is constructed in accordance with the design,

(b) checking the validity of design assumptions, including identification of significant differences between the actual ground conditions and those assumed in the design analyses,

(c) undertaking inspections and compliance testing of the ground, compliance and compaction control testing of the backfill, filter and drainage materials and other construction materials, to check that the requirements of the specifications are met,

(d) assessing the safety and adequacy of the methods used in constructing the retaining wall, and of the construction sequence,

(e) assessing the safety of temporary works, and the effects of such works on the wall and the nearby ground, structures and services,

(f) monitoring the performance of the wall and its surroundings,

(g) identifying the need for remedial measures, alterations to design, construction method and sequence, etc., and implementing appropriate remedial works, and

(h) evaluating the performance of the completed wall, usually up to the time of handing over to the owner or maintenance authority.

The level and quality of supervision which can be ensured during construction should be allowed for in the selection of design parameters and partial safety factors. Design decisions which are influenced by the reliability of supervision and monitoring must be clearly identified. It should be noted that the minimum partial safety factors recommended in Tables 6 and 7 cannot cover gross errors due to improper construction.

12.3 CONSIDERATION OF SUPERVISION IN DESIGN

12.3.1 Planning

The requirements for inspection, control, field and laboratory testing during construction, as well as monitoring of construction and performance, should be planned during the design stage.

A supervision scheme should be included in the geotechnical design report (see Section 4.6), a copy of which should be kept on site. The scheme should specify the level and content of supervision. Table 21, which broadly classifies the level of supervision into four classes, gives some general guidance on the content of geotechnical supervision. In devising a supervision scheme, the following factors should be taken into account :

(a) risk to persons and property,

(b) degree of uncertainty in the design assumptions,

(c) complexity of the ground conditions,

(d) any innovative elements in the design,

(e) complexity of the method and sequence of construction of both the permanent and the temporary works,

(f) requirements for testing and monitoring,

(g) feasibility of design modifications or of implementing corrective measures during construction, and

(h) size of the project.

The supervision scheme should state the items of information required to be recorded on site and the frequency of recording. The acceptance criteria, e.g. allowable deformation, for the results obtained from any site testing and monitoring should also be stated. The content of the supervision to be carried out by the site staff must be clearly indicated. Those aspects which will require the involvement of the designer during construction, together with the need for any specialist services, should be identified and highlighted in the geotechnical design report. In all cases, the route for communicating site queries to the designer must be clear to the site staff, and should be included in the geotechnical design report.

12.3.2 Inspection and Control

Visual inspection is an essential element in the supervision of construction work. The ground conditions and the geotechnical properties of the soils and rocks assumed in the design should be clearly shown on the drawings for easy reference during construction. For gravity and reinforced concrete retaining walls, the foundation should be inspected to check that the material exposed is consistent with that assumed in the design. For cantilevered walls retaining insitu ground, the ground conditions should be examined upon excavation. Inspection of construction operations which can influence the design, e.g. compaction and installation of filter and drainage systems, should also be carried out.

Relevant compliance and compaction control tests on materials such as fill, filter and drainage materials should be specified. Field tests (e.g. GCO probing) and monitoring (e.g. of groundwater levels) may be required. Routine testing and monitoring are usually carried out by site staff and should be specified as part of the works. The effect of undertaking control testing and monitoring on the construction programme should be evaluated in design and allowance should be made in the construction contract. The designer should indicate in the geotechnical design report those results that are required to help verify the design assumptions, as well as the timing at which these should be made available. Such results should be evaluated by the designer before the decisions which are influenced by them are taken.

12.3.3 Assessment of Design

The design should be assessed during construction on the basis of the results of inspection, control testing and monitoring. This assessment should include comparison of the actual conditions with those assumed. Particular attention should be given to any significant changes in ground and groundwater conditions, loadings on the retaining wall, and the surrounding environment. Unexpected conditions may be revealed during construction. If necessary, the design should be re-evaluated and appropriate actions taken. The designer's involvement may need to extend beyond the construction period. Performance monitoring sometimes needs to be continued after completion of a retaining wall in order to verify the design (see Section 12.4).

12.4   MONITORING

12.4.1   Assessment of Need

The need for performance monitoring should be considered in the design of retaining walls, taking into account the factors listed in Section 12.3.1. In general, monitoring is warranted where :

   (a) the risk to life or property is high as indicated in Table 5.2 of the Geotechnical Manual for Slopes (GCO, 1984),

   (b) the design is sensitive to possible variations in the assumed ground conditions, or

   (c) the construction of the retaining wall will cause changes in the surrounding ground which can significantly affect the wall's performance, e.g. where the buried parts of the wall could cause damming of groundwater.

For non-critical structures, visual observations should be adequate. Where the design is based on the 'observational method' (Peck, 1969), a monitoring scheme with contingency actions should be devised (see Section 4.5). Monitoring of construction works should also be considered, for example dewatering due to caisson construction.

12.4.2   Planning

Proper planning is a prerequisite for the success of a monitoring operation. A flow chart which broadly identifies the different stages in planning is shown in Table 22.

With regard to monitoring, the geotechnical design report (Section 4.6) should state the following :

   (a) the object of each set of observations or measurements, the parts of the retaining wall to be monitored, and the locations and depths of the monitoring stations,

   (b) the frequencies of taking readings and reporting,

   (c) the way in which the results are to be presented and evaluated, particularly where evaluation is to be carried out by the site staff,

   (d) the 'alert' levels and the acceptance criteria,

   (e) the contingency action plan if the 'alert' levels or the acceptance criteria are not complied with,

   (f) the period of time for which monitoring is to continue after

completion of construction, and

(g) the parties responsible for making measurements and observations, interpreting the results and installing, monitoring and maintaining the instruments.

Monitoring to be carried out by the contractor should be clearly specified in the contract documents including drawings.

12.4.3 Measurements and Instrumentation

Where monitoring of the performance of a retaining wall is warranted, the main types of measurements required are :

(a) deformation of the ground and of nearby structures and services,

(b) groundwater levels, pore pressures, flows from drains and their relation to rainfall,

(c) settlement and lateral movement of the retaining wall, and

(d) strains in structural members and values of contact pressure between the ground and the retaining wall (usually only monitored where the consequences of failure are severe or the design contains innovative elements).

The choice of instruments should take into account the required accuracy of the measurements, the reliability of the instruments and the site conditions. The use of simple instrumentation and methods of monitoring is recommended. Some allowance needs to be made for instrument failures during construction. Guidance on the choice and installation of selected instruments is given in the Geotechnical Manual for Slopes (GCO, 1984) and Geoguide 2 : Guide to Site Investigation (GCO, 1987). Typical working accuracies of some commonly-used monitoring methods are given in Table 23. These should be taken into account in specifying acceptance criteria. More detailed guidance on geotechnical instrumentation is given in specialist literature such as Dunnicliff (1988), Hanna (1985) and ICE (1989).

12.4.4 Evaluation of Results

Monitoring results should be interpreted and evaluated by an experienced engineer who is familiar with the design requirements and who understands the object of the monitoring. Evaluation of results should be carried out at appropriate stages, particularly where the data are required to provide an indication of safety. Collection of monitoring records without proper evaluation is simply a waste of valuable resources.

The evaluation should take into account the actual conditions encountered, as well as the construction sequence, incidents which could affect performance, and environmental

conditions. Changes in values between consecutive measurements should be examined. Qualitative observations including the architectural appearance of the retaining wall should also be considered.

12.4.5 Termination of Monitoring

Monitoring during construction can usually be terminated on completion of the works. However, in cases where the design is particularly sensitive to the assumed ground conditions (e.g. with respect to groundwater levels) and the consequence of failure is severe, it is prudent to extend the period of monitoring beyond the construction period in order to verify the long-term performance of the retaining wall.

The length of the post-construction monitoring period may need to be altered as a result of observations obtained during construction. For monitoring of groundwater levels and flow from drains, generally a minimum of at least one wet season should be covered. Where errors in the design assumptions could give rise to grave consequences, the period of monitoring should be extended to include a severe rainstorm for which the design assumptions can be justified. Monitoring should be terminated only if the designer is satisfied with the observed performance. The need should be reviewed regularly to ensure that the work does not develop into perpetual surveillance.

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13. MAINTENANCE

13.1 GENERAL

The performance of retaining walls can be adversely affected by many kinds of environmental and human factors. Periodic maintenance should be carried out to ensure satisfactory long-term performance of retaining walls. Without proper maintenance, the service life of a wall may be significantly reduced, leading to the need for preventive or remedial works, or even replacement of the wall, within an unexpectedly short time. Designers should be aware of typical maintenance requirements and should consider these during the design stage.

13.2 MAINTENANCE REQUIREMENTS FOR RETAINING WALLS

Routine maintenance requirements for the retaining walls covered by this Geoguide are summarised in Table 24. Most of these items are common to all structural types, but their importance may vary depending on the type of wall and the surrounding environment. Further guidance on the maintenance of instruments (e.g. piezometers), surface protection to slopes, surface drainage systems and services is given in Chapter 11 of the Geotechnical Manual for Slopes (GCO, 1984).

Non-routine maintenance is usually only required if the retaining wall has deteriorated to the point where substantial repairs are considered necessary, or where special features are incorporated in the design.

In cases of severe distress or instability, or where a wall has to be upgraded because of environmental changes, preventive or remedial stabilisation works (as opposed to maintenance) will usually be required. Detailed discussion of such measures is beyond the scope of this Geoguide.

13.3 CONSIDERATION OF MAINTENANCE REQUIREMENTS IN DESIGN

Careful design and attention to detailing can reduce both the scope of maintenance required and the physical labour involved. The designer should take account of the likely needs for inspection, monitoring, maintenance and repairs. Efficient access should be provided for carrying out these works, and the safety of personnel must be borne in mind. In this respect, the designer should consider the provision of guard-rails, access ladders and platforms at suitable locations. Lockable housings should be provided to monitoring stations to protect the instruments against vandalism since replacement of damaged instruments is often more difficult than their installation.

On completion of the design, the designer should provide a summary sheet containing all the information relevant to the retaining wall. This should be updated at the end of construction if necessary to include any as-built modifications to the original design. Such information should form the basis for the maintenance records of the wall and should be passed to the owner or the maintenance authority. An example of a retaining wall basic

record sheet is given in Figure 58.

Where considered appropriate by the designer, a maintenance manual should also be provided. A comprehensive maintenance manual should contain, as a minimum, an inspection programme, the recommended maintenance works, any monitoring requirements for the installed instruments together with advice on 'alert' levels, and recommendations for routing of feedback to initiate necessary maintenance actions. Any items which require specialist input should be identified and brought to the attention of the owner and the authority responsible for maintaining the retaining wall.

13.4 MAINTENANCE MANAGEMENT AND INSPECTIONS

Discussion of maintenance management and the content of maintenance inspections is beyond the scope of this Geoguide. The guidance given on these aspects in the Geotechnical Manual for Slopes (GCO, 1984) is also broadly applicable to retaining walls.

In preparing a maintenance manual, the designer should include recommendations on the frequencies of maintenance inspections. The suggested frequencies of routine and engineer inspections for retaining walls are given in Table 25.

Observations made during maintenance inspections should be fully and accurately recorded, so that a complete performance history of the retaining wall is available for review. The use of checklists can help to reduce the possibility of omissions in reporting.

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TABLES

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Table 1 - Advantages and Disadvantages of Different Types of Retaining Wall

Type of Retaining Wall	Advantages / Applications	Disadvantages / Restrictions
Gravity Retaining Wall	(a) Provides stability by virtue of the weight of the wall. (b) Suitable for retention of fill, embankments, etc.	(a) Takes up a relatively large amount of space. (b) Support may need to be provided to any excavation required for construction purposes.
1. Mass Concrete Retaining Wall	(a) Relatively simple to construct and maintain. (b) Can cope with curves and can be built to different shapes for architectural purposes. (c) Features can be incorporated into the finished face.	(a) Reasonably good foundation is required. (b) Large quantities of concrete are used, and curing time is required before becoming effective. (c) Generally uneconomic for heights above 3 m.
2. Crib Wall	(a) Relatively simple to construct and maintain. (b) Maximum use of soil as structural component, and hence minimizes the need for manufactured materials. (c) Uses prefabricated elements, enabling better quality control of components.	(a) Self-draining filling material is required. (b) Cost may be high for small quantities. (c) Generally unsuitable for heights above 7 m.
3. Gabion Wall	(a) As (a) and (b) for crib wall. (b) Permits construction on weaker foundations. (c) Flexible structure, hence can tolerate higher differential settlements than mass concrete and R.C. retaining walls. (d) Relatively easy to demolish in part or in full.	(a) Self-draining filling material is required. (b) Cost may be high for small quantities.
4. Reinforced Fill Structure	(a) As (a), (b) and (c) for crib wall. (b) Can cope with tighter curves than conventional R.C. retaining wall. (c) As (b) ,(c) and (d) for gabion wall.	(a) Land-take requirement may exceed that for other gravity type retaining walls. (b) Reinforced zone requires protection. (c) Stringent requirements on selected fill to be used with steel reinforcing elements. (d) Patents aspects need consideration in contracts. (e) Costs may be high for small quantities.
Reinforced Concrete Retaining Wall	(a) Provides stability by virtue of the strength and stiffness of the reinforced concrete and the weight of the retained fill. (b) Suitable for retention of fill, embankments, etc.	(a) Construction of the spread footing, and key if used, sometimes requires large excavation with structural support. (b) Reasonably good foundation is required. (c) Curing time is required before becoming effective. (d) Thin walls are susceptible to damage by impact.
1. R.C. Cantilever Retaining Wall (L- or inverted T-shaped)	(a) Only conventional construction methods are involved. (b) Features can be incorporated into the finished face.	(a) Generally uneconomic for heights above 8 m.
2. R.C. Counterfort or Buttressed Retaining Wall	(a) Can be constructed to greater heights than R.C. cantilever retaining wall. (b) As (a) and (b) above.	(a) Formwork may be costly.
Cantilevered Retaining Wall	(a) Provides stability by virtue of the bending strength and stiffness of the cantilever. (b) Used mainly for retention of excavated ground where space is restricted or where, owing to the nature of the subsoil, the bearing pressure on the foundation must be kept low. (c) Particularly suitable when it can derive support from, and finally be incorporated into, an adjacent structure.	(a) May require substantial penetration into the ground for stability if rock or strong bearing layer is not found at shallow depth. (b) Cost and ground movement are generally much higher than gravity or R.C. retaining walls. (c) Design is very sensitive to changes in ground level. (d) Impermeable wall may cause a rise in the local groundwater level.
1. Bored Pile (or Caisson) Wall	(a) No temporary cutting is required. (b) Construction of hand-dug caissons is conventional practice in Hong Kong. (c) No heavy plant is required for caisson construction, and hence it can be constructed in areas where access is difficult. (d) Several caissons may be dug simultaneously to reduce construction time.	(a) Extra care to ensure safety during caisson construction is necessary. (b) Excavation of caissons may require dewatering, which will cause ground settlement. (c) Construction cost is high.

Table 2 - Recommended Minimum Frequencies of General Compliance
Tests on Proposed Backfill Behind Retaining Walls

Volume of Backfill in Batch[(1)](m^3)	Number of Samples per Batch
0 - 3000	Three
> 3000	One for each 1000 m^3 or part thereof

Notes : (1) A batch is any quantity of fill material which is from the same source and has similar properties.

(2) General compliance tests should be specified by the designer to check whether the proposed backfill satisfies the material specification. The tests on each sample should normally include determination of the particle size distribution, Atterberg limits, moisture content and compaction characteristics, as well as any electrical and chemical tests that are considered necessary.

(3) Shear strength testing may have to be specified to check design assumptions. The selection of test samples and the frequency of testing should take into account the variability of the materials.

(4) The recommended frequencies are appropriate for soft fill materials commonly encountered in Hong Kong (e.g. fill consisting of completely decomposed granite obtained from a single source). For crushed rock products, a lower frequency of testing may be considered.

(5) Particular requirements for reinforced fill structures are given in Geospec 2 (GCO, 1989a).

Table 3 - Recommended Minimum Frequencies of Compaction Control
Tests on Backfill Placed Behind Retaining Walls

Size of Area of Backfill in Batch[1](m^2)	Number of Tests per Batch
0 - 100	Three
100 - 500	Two for each 100 m^2 or part thereof over 100 m^2
> 500	One for each 100 m^2 or part thereof over 500 m^2

Notes : (1) A batch is any quantity of fill material which is from the same source and has similar properties, and which is deposited in a single layer behind the retaining wall.

(2) Compaction control tests should be included in performance specifications to check whether the compacted backfill conforms to the specified level of compaction. The tests should include the determination of insitu densities and moisture contents from which values of relative degree of compaction can be calculated.

(3) Particular requirements for reinforced fill structures are given in Geospec 2 (GCO, 1989a).

Table 4 - Grading and Plasticity Requirements for Retaining Wall Backfill

Requirement	Backfill Type	
	Crushed Rock Products	Soil
Maximum Size (mm)	200	75[2]
% Passing 63 microns BS Sieve Size	0	0 - 45
Coefficient of Uniformity	≥ 5	≥ 50[4]
Liquid Limit (%)	Not applicable	≤ 45[5]
Plasticity Index (%)	Not applicable	≤ 20[5]

Notes : (1) Relevant test methods for grading and plasticity of fill materials are specified in Clauses 5.1 and 5.2 of Geospec 2 (GCO, 1989a).

(2) The backfill may contain up to 5% of rock fragments not exceeding 200 mm in size, provided that these neither interfere with the compaction requirements nor cause any damage to the retaining wall.

(3) In addition to the above requirements, the maximum particle size should not exceed two-thirds of the thickness of the compacted layer of backfill in order to ensure good compaction.

(4) This applies to soils derived from insitu rock weathering only. For sands and gravels of alluvial origin, the coefficient of uniformity should be not less than 5 and the material should not be gap-graded (i.e. having two or more distinct sections of the grading curve separated by sub-horizontal portions).

(5) There is no need to check the liquid limit and plasticity index of the soil if the backfill contains less than about 30% by weight of particles less than 63 μm.

(6) The determination of the particle size distribution of the backfill should be carried out without using dispersants.

Table 5 - Some General Data on Chemical and Thermal Stability of Synthetic Polymer Fibres Commonly Used in the Manufacture of Geotextiles

Polymer Type	Resistant to		Stable between (°C)	Remarks
	Acid Conditions	Alkali Conditions		
Polypropylene	pH ≥ 2	All	-15 to 120	Attacked at elevated temperatures by hydrogen peroxide, sulphuric acid and nitric acid. Weakened by certain solvents, e.g. diesel fuel. Insignificant change in strength between 20°C and 35°C.
Polyester	pH ≥ 3	pH ≤ 10	-20 to 220	Degrades by hydrolysis under strongly alkaline conditions. Therefore, concrete must not be cast directly against it. Insignificant change in strength between 20°C and 35°C.
Polyamide (nylon 6.6)	pH ≥ 3	pH ≤ 12	-20 to 230	Degrades by hydrolysis under strongly acidic conditions. Reduces in strength by up to 30% when immersed in water or used in a saturated environment. Insignificant change in strength between 20°C and 35°C.
Polyethylene	pH ≥ 2	All	-20 to 80	Same as polypropylene, except strength at 35°C is lower than that at 20°C by about 25%.

Note :   Data based on Cooke & Rebenfeld (1988), Lawson & Curiskis (1985) and Van Zanten (1986).

Table 6 - Minimum Partial Load Factors for Use in Design
Against Ultimate Limit States

Loading	Partial Load Factor, γ_f
Dead load due to weight of the retaining wall	1.0
Dead load due to weight of soil, rock and water	1.0
Surcharge	1.5[1]
Seismic load	1.0
Water pressure	1.0
Note : (1) γ_f should be set to zero for those surcharges which produce a favourable effect.	

Table 7 - Minimum Partial Material Factors for Use in Design
Against Ultimate Limit States

Material Parameters	Partial Material Factor, γ_m
Soil : unit weight γ	1.0
base friction angle tan δ_b[1]	1.2
drained shear strength	1.2[2]
undrained shear strength s_u	2.0
permeability k	1.0
Rock : unit weight γ	1.0
shear strength of joint	1.2
compressive strength, e.g. q_c, q_u	2.0
permeability k	1.0
Water : unit weight γ_w	1.0
Granular filter and drainage materials : permeability k	10.0
Structural materials : unit weight γ	1.0
strength parameters f_m	As per relevant structural code
Notes : (1) The selected value of δ_b should be based on the unfactored shear strength of the relevant soil or the soil/wall interface (see Section 5.12). (2) For a c' - ϕ' (Mohr-Coulomb) strength model, the γ_m value should be applied to the selected values of shear strength parameters c' and tan ϕ'.	

Table 8 - Typical Ranges of Values of Geotechnical Parameters for
Selected Hong Kong Soils

Soil Type	Bulk[3] Unit Weight, γ (kN/m³)	Dry Unit Weight, γ_d (kN/m³)	Shear Strength Parameters		Mass Permeability k(m/s)
			ϕ'	c'(kPa)	
Compacted Fill[3]					
Completely decomposed granites	19 - 21	15 - 19	38° - 42°	0 - 5	10^{-6} - 10^{-7}
Completely decomposed volcanics (tuffs & rhyolites)	18 - 21	15 - 19	35° - 38°	0 - 5	10^{-6} - 10^{-8}
Crushed rock fill	18 - 21	18 - 21	45° - >50°	0	$> 10^{-2}$ - 10^{-4}
Insitu Soil					
Completely decomposed granites[5][6]	16 - 21	14 - 19	35° - 44°	5 - 15	10^{-5} - 10^{-7}
Completely decomposed volcanics (tuffs & rhyolites)[5][6]	16 - 21	14 - 19	32° - 38°	5 - 10	10^{-5} - 10^{-7}
Colluvium (matrix material)[7]	15 - 21	13 - 19	26° - 40°	0 - 10	10^{-4} - 10^{-7}

Notes : (1) The ranges of values given in this Table, which are appropriate for the stress levels normally encountered in the design of retaining walls, are for general guidance only.

(2) Soils derived from the meta-sedimentary and argillanceous rocks (see Figure 1) may have markedly different typical values of geotechnical parameters.

(3) Saturated bulk unit weights have higher values.

(4) The typical values of parameters for compacted fill are for materials which fall broadly within the grading and plasticity limits given in Table 4 and compacted to at least 95% of their maximum dry density.

(5) The typical values of parameters for completely decomposed granites and volcanics (tuffs & rhyolites) correspond to tests on Class 1 samples as defined in Geoguide 2 (GCO, 1987).

(6) The lower bound value of ϕ_{cv}' for insitu completely decomposed granites and completely decomposed volcanics (tuff & rhyolites) is about 34° and 30° respectively.

(7) The relatively wide ranges of the parameters reflect the variable composition of colluvial matrix material in Hong Kong.

(8) It is not meaningful to give typical values for existing fill, which can be extremely variable.

Table 9 - Typical Ranges of Values of Young's Modulus and Poisson's Ratio
for Various Soils Considered as Homogeneous Elastic Materials

Soil Type	Young's Modulus, $E_s{}'$ (MPa)	Poisson's Ratio, $\nu_s{}'$[4]
Loose sand	5 - 20	0.30 - 0.40
Medium dense sand	16 - 40	0.20 - 0.35
Dense sand	30 - 100	0.15 - 0.30
Soft clay	1 - 4 (2 - 6)	
Firm clay	3 - 8 (5 - 12)	0.1 - 0.3 (0.5)
Stiff clay	5 - 15 (10 - 20)	

Notes : (1) Table based on Elson (1984).
(2) It is preferable, but may not be convenient or possible, to idealise sands and soft clays as elastic materials with modulus proportional to depth (i.e. Gibson materials), rather than homogeneous materials as assumed in this Table.
(3) Values of Young's modulus E_u and Poisson's ratio ν_u corresponding to 'undrained' loading for clays are given in brackets.
(4) It should be noted that elastic theory cannot account for the effect of dilatancy upon shear, which may affect the apparent Poisson's Ratio of the soil beyond the initial elastic range.

Table 10 - Typical Ranges of Values of Constant of Horizontal
Subgrade Reaction of Sand for Pile Analysis

Relative Density	Loose	Medium Dense	Dense
SPT 'N' Value	4 - 10	10 - 30	30 - 50
n_h (dry sand) (MN/m³)	2.2	6.6	17.6
n_h (submerged sand) (MN/m³)	1.3	4.4	10.7

Note : Values of n_h are based on Terzaghi (1955).

Table 11 - Typical Ranges of Values of Uniaxial Compressive Strength
for Common Hong Kong Rocks

Rock Type	Material Decomposition Grade[3]	Uniaxial Compressive Strength, q_c(MPa)
Granites, Granodiorites and Tuffs	I	150 - 350
	II	100 - 200
	III	10 - 150
Notes : (1) Values of q_c are based on the work reported by Lumb (1983), Gamon (1984) and Irfan & Powell (1985). (2) Quoted figures are for intact rock. Failure through discontinuities gives lower values which should be discarded. (3) Based on Geoguide 3 classification (GCO, 1988).		

Table 12 - Typical Ranges of Values of Laboratory Modulus of Deformation
and Poisson's Ratio for Common Hong Kong Rocks

Rock Type	Material Decomposition Grade[3]	Laboratory Modulus of Deformation, E_l (GPa)	Poisson's Ratio ν_l
Granites and Granodiorites	I	40 - 70	)
	II	20 - 50	)) 0.2 - 0.4)
	III	5 - 30	)
Tuffs and Dyke Rocks (e.g. Dolerite)	I	60 - 150	)
	II	30 - 80	)) 0.2 - 0.5)
	III	5 - 40	)
Notes : (1) Values of E_l and ν_l are based on GEO internal data. (2) E_l values are for small imposed stress levels (<5 MPa). (3) Based on Geoguide 3 classification (GCO, 1988).			

Table 13 - Typical Ranges of Values of Angle of Skin Friction Between
Granular Soils and Various Construction Materials

Construction Material	Typical Values of Angle of Skin Friction, δ_s
Smooth concrete (cast with steel formwork),	0.8 to 0.9ϕ'
Rough concrete (cast with timber formwork),	0.9 to 1.0ϕ'
Smooth masonry blocks[2] (dressed granitic or volcanic blocks)	0.5 to 0.7ϕ'
Rough masonry blocks (irregular granitic or volcanic blocks)	0.9 to 1.0ϕ'
Smooth steel (polished)	0.5 to 0.6ϕ'
Rough steel	0.7 to 0.8ϕ'
Geotextile	0.5 to 0.9ϕ'[4]

Legend :

ϕ' Angle of shearing resistance of soil

Notes :   (1) Values of δ_s are based on Potyondy (1961), NAVFAC (1982b) and literature on geotextiles.

(2) Angle of skin friction may be measured using a direct shear test, in which an assessment of the roughness of the test specimen should be made.

(3) In selecting the value of δ_s for design, account should be taken of the large-scale roughness of the construction material in the wall and the form of construction.

(4) The value of δ_s for geotextile is product specific.

Table 14 - Suggested Maximum Values of Mobilised Angle of Wall
Friction for Active Earth Pressure Calculations

Type of Wall	Maximum Value of Mobilised Angle of Wall Friction, δ_{max}
At the back of crib walls, gabion walls and reinforced fill structures, and cantilevered retaining walls embedded entirely in soil	$+\phi'/2$
At the 'virtual back' of R.C. L- or inverted T-shaped cantilever or counterfort retaining walls	$+\phi'/2$ or the backfill slope, whichever is smaller
At the soil/wall interface of mass concrete and R.C. reversed L-shaped cantilever or buttressed retaining walls, and bored pile (caisson) walls socketed in rock	$+2\phi'/3$

Notes : (1) The direction of the anticipated relative movement between the retained soil and the retaining wall should be considered in design. By convention, a positive value of δ corresponds to the case of the retained soil moving downwards relative to the wall.

(2) The δ_{max} values given in this Table are applicable to retaining walls which will not move downwards relative to the retained soil when active earth pressure is developed, e.g. walls founded on relatively incompressible foundations. Where the wall could settle after completion of backfilling (either due to consolidation of the compressible foundation or additional loading on top of the wall causing foundation settlement), δ should be assumed to be zero in design. For walls founded on compressible ground, δ should also be assumed to be zero.

(3) The δ value to be used should ensure that vertical equilibrium of the wall can be achieved.

Table 15 - Suggested Maximum Values of Mobilised Angle of Wall
Friction for Passive Earth Pressure Calculations

Type of wall	Maximum Value of Mobilised Angle of Wall Friction, δ_{max}	
	Loose Soil	Dense Soil
Walls which move horizontally or settle only very slightly relative to the soil in the passive zone, e.g. retaining walls bearing on relatively incompressible foundations.	0	$-\delta_s/2$
Sheet retaining walls with freedom to move downwards relative to the soil in the passive zone, e.g. walls bearing on soft or loose soil which settle under vertical forces due to active earth pressure or inclined anchored loads.	$-2\delta_s/3$	$-2\delta_s/3$
Walls whose stability relies on soil in the passive zone which may settle under external loads.	0	0

Legend :

δ_s Angle of skin friction between soil and construction material used for the retaining wall (Table 13).

Notes :   (1) The direction of the anticipated relative movement between the soil in the passive zone and the retaining wall should be considered in design. A negative value of δ corresponds to the case of the soil moving upwards relative to the wall.

(2) The δ_{max} values given in this Table are derived from the results of large-scale laboratory tests in sand (Rowe & Peaker, 1965). They may be assumed to be applicable to fill material which falls within the requirements given in Table 4 and also to soils derived from insitu weathering of granites, granodiorites and tuffs. For soft cohesive soils, δ should be assumed to be zero.

(3) Loose soil and dense soil are as defined in Geoguide 3 (GCO, 1988).

Table 16 - Table of Nominal Surcharge Loads

Source of Loading	Equivalent Uniformly-Distributed Load
Buildings with shallow foundations[1]	10 kPa per storey[2]
Highway structures[3] (a) HA Loading (b) HB Loading (45 units)	10 kPa 20 kPa
Footpaths isolated from roads, cycle tracks and play areas[4]	5 kPa

Notes : (1) While the loading from buildings with shallow foundations can reasonably be idealized as a uniformly-distributed load in some cases, the designer should scrutinize this assumption in each situation (see Section 7.2.1).

(2) Where the use of the building is for the storage of heavy goods or machinery, the equivalent uniformly-distributed load of 10 kPa per storey may not be adequate. A detailed assessment for such cases is recommended.

(3) The designer should use the appropriate combinations of highway loadings for different design situations (see Section 7.2.1).

(4) Where the footpath is not isolated from roads and there is a possibility that vehicles may park on the footpath, the appropriate highway loading should be adopted.

Table 17 - Design Criteria for Granular Filters

Rule Number	Filter Design Rule[1]	Requirement
1	$D_{15}F_c < 5 \times D_{85}S_f$	Stability (i.e. the pores in the filter must be small enough to prevent infiltration of the material being drained)
2	Should not be gap-graded (i.e. having two or more distinct sections of the grading curve separated by sub-horizontal portions)	
3	$D_{15}F_f > 5 \times D_{15}S_c$	Permeability (i.e. the filter must be much more permeable than the material being drained)
4	Not more than 5% to pass 63μm sieve and that fraction to be cohesionless	
5	Uniformity Coefficient $4 < \dfrac{D_{60}F}{D_{10}F} < 20$	Segregation (i.e. the filter must not become segregated or contaminated prior to, during, and after installation)
6	Maximum size of particles should not be greater than 50 mm	

Notes : (1) In this Table, $D_{15}F$ 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, $D_{85}S$ 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.

(2) For a widely graded base soil, with original $D_{90}S > 2$ mm and $D_{10}S < 0.06$ mm, the above criteria should be applied to the 'revised' base soil grading curve consisting of the particles smaller than 5 mm only.

(3) The thickness of a filter should not be less than 300 mm for a hand-placed layer, or 450 mm for a machine-placed layer.

(4) Rule 5 should be used to check individual filter grading curves rather than to design the limits of the grading envelope.

(5) The determination of the particle size distributions of the base soil and the filter should be carried out without using dispersants.

Table 18 - Geotextile Filter Criteria for Use with Woven
and Nonwoven Heat-bonded Fabrics

Retention Criteria	Permeability Criteria
For predominantly granular soils with $D_{85} > 0.10$ mm, e.g. residual soils which are granular in nature and alluvial sandy soils, and for PFA : $$O_{90} \leq D_{85}$$ For predominantly fine-grained soils with $D_{85} < 0.10$ mm, use appropriate criterion below : (a) For non-cohesive soils, e.g. silts of alluvial or other origin, and for non-dispersive cohesive soils, $$0.08 \text{ mm} \leq O_{90} \leq 0.12 \text{ mm}$$ (b) For dispersive cohesive soils, $$0.03 \text{ mm} \leq O_{90} \leq D_{85}$$	$O_{90} \geq D_{15}$ and minimum VWFR requirement using $O_{90} = D_{15}$ (see diagram below) VWFR ≥ 35 l/m²/s

Legend :

O_{90}	Particle size at which 90% by weight of particles are retained on geotextile upon dry sieving using ballotini (glass beads)
D_n	Grain sizes of soil corresponding to n% passing in a grading test
VWFR	Volume water flow rate, defined as volume of water passing/unit area/unit time/100 mm water head

Notes :	
(1)	Criteria based on Lawson (1987).
(2)	Consideration should be given to soil and geotextile variability and the effect of geotextile compression on VWFR in design.
(3)	The determination of the particle size distribution of the base soil should be carried out without using dispersants.

Table 19 - Geotextile Filter Criteria for Use with Nonwoven Needle-punched Fabrics

Retention Criteria	Permeability Criteria
$O_f < C_g \times D_{85}$	$\psi > A_g \times k$
where $C_g = C_1 \times C_2 \times C_3 \times C_4$	where $A_g = A_1 \times A_2 \times A_3 \times A_4 \times A_5$
$C_1 =$ 1.0 for well graded continuous soil 0.8 for uniform soil ($C_u < 4$)	For high risk structures, e.g. earth dams, $A_1 =$ 100 to allow for contamination during installation and in-service
$C_2 =$ 1.25 for dense[2] and confined soil 0.8 for loose[2] or unconfined soil	$A_2 =$ 3 to allow for compression of geotextile under load
$C_3 =$ 1.0 for hydraulic gradient, i < 5 0.8 for 5 < i < 20 0.6 for 20 < i < 40 or alternating flow	$A_3 =$ 10 to allow for i up to 10
$C_4 =$ 1.0 for filtration function only 0.3 for filtration and drainage functions	$A_4 =$ 10 for allowable head loss $\Delta H = 0.1$ m $A_5 =$ 3 is a global safety factor
For 'fine' soils, i.e. soils with a $C_g \times D_{85}$ value less than 0.05 mm, O_f can be taken as 0.05 mm.	$\therefore \psi > 10^5 k$
For 'problem' soils, i.e. soils containing fines which can go easily into suspension (e.g. sand with low clay content),	For other structures, e.g. slopes, embankments and drainage trenches,
$4D_{15} < O_f < C_g \times D_{85}$	$\psi > 10^4 k$
For gap-graded soils, use grading curves of fine fraction for design.	For clean sands with fewer than 12% < 0.08 mm, $\psi > 10^3 k$

Legend :	
O_f O_f, known as the filtration diameter, is taken as the D_{95} of the soil that has passed the geotextile upon hydrodynamic sieving	k Coefficient of permeability of the soil
	k_n Coefficient of permeability of the geotextile
D_n See Table 18	MDD Maximum dry density of soil
C_u Uniformity coefficient $= D_{60}/D_{10}$	ψ Permittivity $= k_n/T_g(s^{-1})$, where T_g is thickness of the geotextile

Notes :   (1) Criteria based on the French Geotextile Manual (CFGG, 1989).
(2) Dense soil : degree of compaction $\geq$ 95% MDD or relative density $\geq$ 65%.
　　Loose soil : degree of compaction < 95% MDD or relative density < 65%.
(3) For potentially dispersive soils, long-term filtration tests are recommended (GEO, 1993).
(4) As note (3) of Table 18.

Table 20 - Elements of Design for Gravity and Reinforced Concrete Retaining Walls

Wall Type	Elements of Design	Earth Pressure in Backfill	Compaction-induced Lateral Pressure	Geotechnical Parameters
Free-standing Gravity or Reinforced Concrete Retaining Walls	Checking against ultimate limit state of sliding, overturning and bearing capacity failure	Active	No need to consider	Factored
	Checking against middle-third or middle-half rule	Active	No need to consider	Unfactored
	Serviceability calculations, e.g. movement assessment	At-rest[1]	Need to consider[1]	Unfactored
	Structural design	At-rest[1]	Need to consider[1]	Unfactored
Restrained Rigid Retaining Walls	Structural design	At-rest	Need to consider	Unfactored
Note : (1) Where wall movement is large enough to mobilize the active condition, active earth pressure can be used and compaction-induced lateral pressure can be omitted in the calculation.				

Table 21 - Classes of Geotechnical Supervision

Supervision Class	Frequency of Inspection and Personnel	Situation	Content of Geotechnical Supervision
R	Routine inspection by site staff with visits by a suitably experienced person from the design office when required	Where the design and the stability of the temporary works is relatively insensitive to errors in the geotechnical design assumptions, but where moderate geotechnical problems may still arise during construction.	Visual inspection. Determination of actual ground conditions, including detailed description of soil and rock exposed in excavations (e.g. at the formation level of retaining wall foundations) and any seepage observed. Qualitative assessment of performance of the retaining wall and temporary works. Assessment of any unforeseen ground conditions and liaison with designer for implementation of remedial works.
A	Periodic inspections by a suitably qualified and experienced engineer from the design office	Where stability analyses or ground movement calculations are sensitive to variations in the geotechnical design assumptions, which need to be checked on site as works proceed. After checking the geotechnical design assumptions, the need for modifications to the design has then to be judged by the geotechnical supervisor.	In addition to the above, quantitative assessment is required. Additional items of supervision which may be required include : (a) additional site investigation (e.g. mapping and measurement of geological features such as rock discontinuities; pumping tests), (b) determination of soil and rock properties by laboratory tests on recovered samples or insitu tests (e.g. plate loading tests), (c) determination of the properties of construction materials (e.g. index properties, strength and deformability), and (d) monitoring of groundwater levels, pore pressures, drain flows, settlement and lateral movement of the retaining wall and nearby ground, structures and services.
B	Periodic inspections by a senior suitably qualified and experienced engineer from the design office	Applies to special sites of high sensitivity.	As for class A above.
C	Full-time supervision by a suitably experienced person	Where day-to-day checks on compliance with specifications or working procedures are necessary. Applies to works which depend for their success and safety on a high standard of workmanship and materials, e.g. installation of excavation supports involving complicated working procedures, and ground anchoring.	Quantitative assessment of the actual ground conditions and the performance of the retaining wall and temporary works. Measurements should be taken during each significant stage of construction and comparisons should be made with the predicted behaviour of the retaining wall, the foundation and the retained ground. Construction effects on nearby ground, structures and services should be closely monitored.

Notes : (1) This Table is applicable for works which could pose a high risk to life or property. Less stringent requirements may be appropriate for walls in the low and negligible risk categories. The risk categories are those defined in the Geotechnical Manual for Slopes (GCO, 1984).

(2) For planning a monitoring operation, see Table 22.

(3) Combinations of the classes may be appropriate, e.g. A and C, B and C, R and B.

Table 22 - 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 Number 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
Note : Table taken from GCO (1984).		

Table 23 - Typical Working Accuracies of Monitoring Methods

Required Measurement	Monitoring Method / Instrument Used	Typical Working Accuracy
Earth movements (a) Settlement[1] (b) Lateral movements	 Routine levelling Precise levelling Steel tape or levelling staff to take offsets[2]	 $\pm$ 12 $\sqrt{\Delta_d}$ mm $\pm$ 4 $\sqrt{\Delta_d}$ mm $\pm$ 2 - 4 mm
Groundwater (a) Levels in observation wells and piezometers (b) Flow from drains	 Electrical dipmeter Flowmeter or graduated container and stopwatch	 $\pm$ 15 mm $\pm$ 0.2 litre/min
Structural deformations (a) Translations (b) Separations (c) Rotations (d) Strains	Steel tape or levelling staff to take offsets[2] Inclinometer Distometer Demec gauges or electronic digital calliper[3] Tiltmeter[3] Vibrating wire strain gauges	$\pm$ 2 - 4 mm $\pm$ 6 mm per 30 m depth $\pm$ 0.5 mm (up to 20 m) $\pm$ 2 mm $\pm$ 40" (i.e. ten 4" units) $\pm$ 10 microstrains

Legend :

Δ_d Traverse distance in kilometres

Notes : (1) A stable datum has to be established.
        (2) A clear sight line has to be established.
        (3) Measurements are very sensitive to the skill of the operator.

Table 24 - Routine Maintenance for Retaining Walls

Item	Typical Maintenance Requirements	Design Notes
Unplanned Vegetation	(a) Removal or trimming of undesirable growth on or around the wall	(a) Where possible, avoid structural details which encourage entrapment of loose soil and moisture (e.g. corners, recesses, etc.) (b) Add small impervious aprons to surface channels to reduce potential for vegetation blockage
Planned Vegetation	(a) Trimming to control growth (b) Fertilisation of root zone environment	(a) Consider planter boxes/trenches at parapet levels or landscaping of platforms incorporated in rigid structures (b) Provide suitable retention systems for purpose-planted vegetative covers on flexible structures (e.g. reinforced fill slopes)
Rigid Surface Covers (e.g. chunam and shotcrete)	(a) Remove undesirable vegetation growth on or around the cover (b) Repair minor cracks or spalling (c) Regrade and repair eroded areas	(a) Where possible, avoid rigid surfacings on permanent slopes formed in association with retaining walls : vegetative covers preferred (b) Consider tree rings if large areas of rigid surfacing unavoidable
Surface Drainage System (including channels, catch-pits, trash grills, sand traps, etc)	(a) Clearance of debris, vegetation growth and other obstructions (b) Repair cracks with cement mortar or mastic sealing compound	(a) Where possible, avoid sharp channel bends and changes in gradient (b) Incorporate catchpits at channel junctions or abrupt changes in gradient, trash grills at junctions of channels with downpipes, and sand traps at the base of the wall where large quantities of erodible material are expected
Weepholes and Drainage Pipes Through the Wall	(a) Clear obstructions to weepholes and pipe ends (b) Probe with rods or flush with clean water jet under controlled pressure for deeper obstructions	(a) Provide weepholes and pipes at specific seepage points if full-height filter not incorporated immediately behind the wall
Horizontal (Raking) Drains	(a) Clear obstructions (e.g. weeds and debris) from drain exits (b) Flush with clean water jet under controlled pressure for deeper obstructions (c) Where appropriate brush clean, replace or repair removable drain inner liners	(a) Consider the long-term maintenance requirements before adopting horizontal drains for controlling groundwater pressures behind the wall (b) Consider provision of outlet pipes and surface channels to divert permanent drain flows to safe discharge points
Water-carrying Services	(a) Inspect the wall and surroundings for signs of unusual seepage or moisture (b) Arrange investigation and repair with appropriate maintenance parties if service leakage suspected	(a) Consider carefully re-routing/replacement options for services directly affected by construction (b) Where possible, minimise effects of excavation and loading on adjacent service pipes and channels
Structural Facings and Materials	(a) Check for evidence of bulging, tilting, settlement or corrosion of reinforcement (b) Repair minor cracking or spalling of concrete surfaces and re-point deteriorated mortar joints on masonry faces ; monitor if continuing significant distress suspected (c) Clean unsightly discolourations or surface staining (e.g. below weepholes) (d) Replace missing or deteriorated joint fillers and sealants	(a) Avoid large areas of plain concrete surfacing; where appropriate incorporate suitable ribs, fluting, boxing or other decorative finishes (b) Consider specialist decorative art work for very prominent large facings (c) Detail construction and expansion joints at suitable locations
Instruments	(a) Keep covers and locks to instrument boxes or housings in good working order (b) Check instruments functioning at suitable intervals (c) Arrange investigation and repair of faulty instruments with appropriate agents	(a) Where possible, avoid instrumentation as an integral part of the design. Instrumentation is warranted in two cases : (i) to provide information to verify *critical* design assumptions, and (ii) to detect and monitor possible distress in high risk walls. (b) The following are the most commonly used types : piezometers, flow-measuring devices, telltales, tilt plates, settlement plates, extensometers and inclinometers

Notes : (1) Safe and efficient access is important for maintenance operations and should be allowed for specifically in the design where appropriate.
(2) Where signs of distress are observed, the cause should be identified. Unusual seepage or moisture should also be thoroughly investigated.

Table 25 - Suggested Frequencies of Routine and Engineer Inspections
for Retaining Walls

Type of Inspection	Inspection Frequencies for Different Risk Categories[1]	
	High to Low	Negligible
Routine inspection	Yearly, preferably before wet season	Every five years
Engineer inspection	Every five years	-

Notes : (1) The risk categories are those defined in the Geotechnical Manual for Slopes (GCO, 1984). Where the land use is changed, the adequacy of the maintenance and monitoring requirements should be reviewed.

(2) The inspection frequencies are for general guidance only. Account should be taken of the age and condition of the wall and the particular monitoring requirement. More frequent inspections should be carried out when there have been adverse environmental changes or when signs of distress have been observed. Immediate actions and follow-up actions are required in emergency situations.

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FIGURES

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LIST OF FIGURES

Figure 1 - Geological Sketch Map of Hong Kong

(a) Gravity Retaining Walls :

    (i) Mass Concrete Retaining Wall
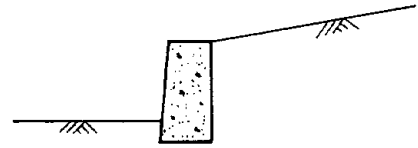
    (ii) Crib Wall
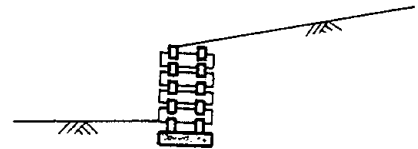
    (iii) Gabion Wall
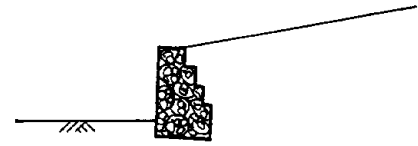
    (iv) Reinforced Fill Retaining Structure
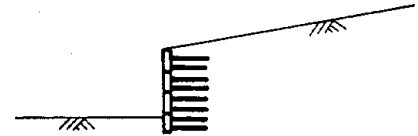
(b) Reinforced Concrete Retaining Walls :

    (i) R.C. L- or Inverted T-shaped
    Cantilever Retaining Wall
    (with or without key)
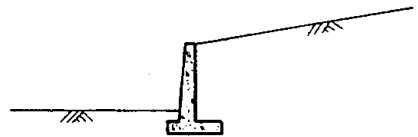
    (ii) R.C. Counterfort Retaining Wall
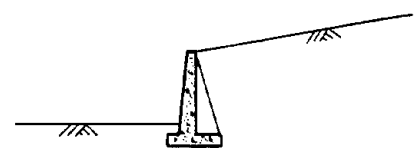
    (iii) R.C. Buttressed Retaining Wall
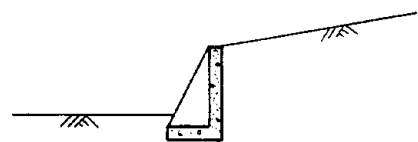
(c) Cantilevered Retaining Walls :

    (i) Bored Pile or Caisson Wall
    (may or may not be socketed in rock)
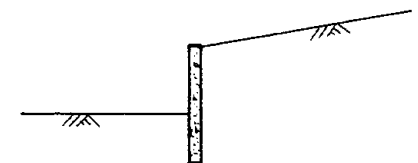
Figure 2 - Types of Retaining Walls
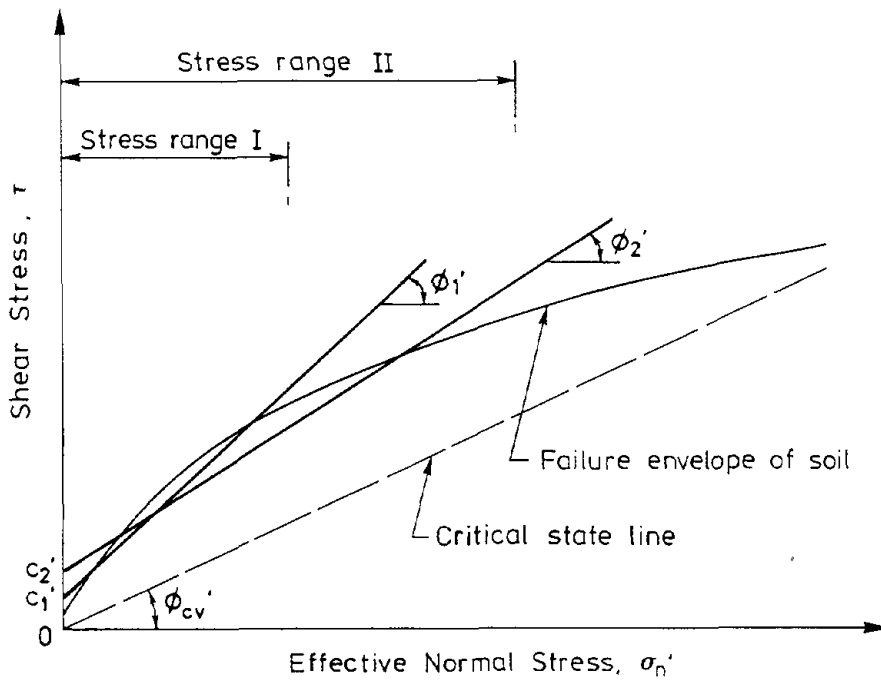
Figure 3 - Characterization of Soil Shear Strength Using a c' - ϕ' Model

(a) Limited data. Only strong samples tested - unsafe strength parameters

(b) Limited data. Only weak samples tested - conservative strength parameters

(c) Limited data. Weak samples tested at low stress and strong samples at high stress - conservative strength parameters

(d) Limited data. Strong samples tested at low stress and weak samples at high stress - unsafe strength parameters in low stress range and conservative parameters in high stress range

(e) No data in the low stress range - unsafe strength parameters in low stress range

Legend :

p_f'	$(\sigma_{1f}' + \sigma_{3f}')/2$	$\sigma_{1f}', \sigma_{3f}'$	Major and minor principal
q_f	$(\sigma_{1f}' - \sigma_{3f}')/2$		effective stresses at failure
	Limits of shear strength	σ_{nf}'	Effective normal stress at failure
	envelope of soil	τ_f	Shear stress at failure
*	Relevant stress range		

Figure 4 - Pitfalls in Using the Least Squares Method for Interpretation of Laboratory Shear Strength Test Data

τ_f Largest relevant stress range for all the modes of action

Estimated upper limit of strength envelope

Estimated lower limit of strength envelope

σ_{nf}'

τ_f Relevant stress range for a particular mode of action

Straight line for the purpose of determining c' and ϕ'

ϕ'

Upper and lower limits of strength envelope from Figure (a)

c'

σ_{nf}'
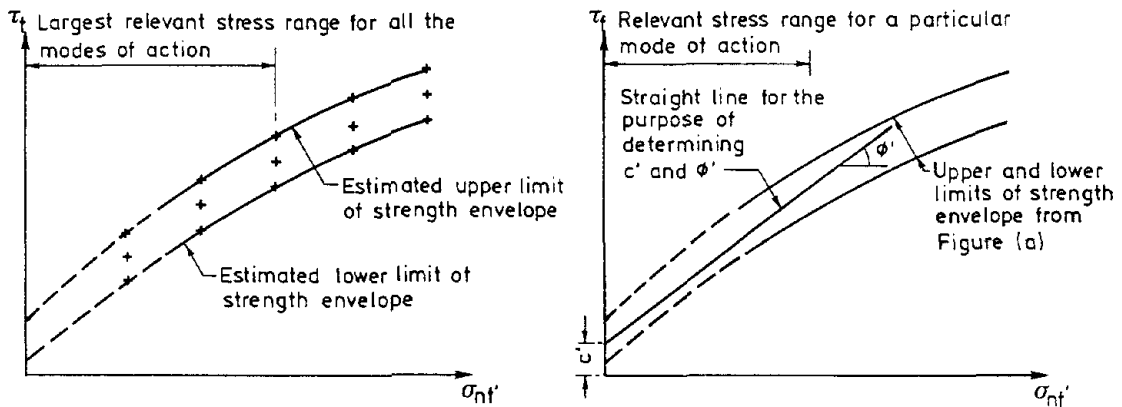
(a) Estimation of Strength Envelope over the Full Testing Range

(b) Estimation of c' and ϕ' for a Particular Mode of Action

PROCEDURE :

1. Divide the ground into domains of similar materials, based on ground investigation results and the geological model.

2. Select samples representative of each soil domain for testing. Consideration should be given to the likely variability of materials in deciding the number of tests.

3. Estimate the relevant stress ranges for different modes of action, e.g. active pressure, passive pressure, bearing capacity, etc. for each soil domain.

4. Plan the testing schedule (for each soil domain) with an aim to give data distributed over a stress range about twice the largest relevant stress range estimated in step 3.

5. Estimate the upper and lower limits of strength envelope of each soil from data, extrapolating conservatively over the low stress region with due allowance given to possible curvature of the strength envelope (see Figure (a) above).

6. Select values of c' and ϕ' for a particular mode of action based on the upper and lower limits of strength envelope estimated in step 5, within the relevant stress range estimated in step 3 (see Figure (b) above). Account should be taken of the amount of data within the relevant stress range, extrapolation in the low stress region, possibility of curvature of the strength envelope and the likely variability of the soil. The value of c', in particular, should be selected with care.

Notes :   (1)   The above procedure is intended for general guidance only. Considerable judgement and experience is required to determine selected values of relevant design shear strength parameters for different limit states (see Sections 5.3 and 5.5.2).

(2)   The above τ_f - σ_{nf}' plots are used to illustrate the interpretation of direct shear test data. For interpretation of triaxial test data, p_f' - q_f plots are drawn, sometimes with the stress paths included. The fact that the end points of triaxial tests are not known until after the tests presents difficulties in planning the testing schedule. Previous triaxial test data on similar materials, where available, can be invaluable in the specification of confining pressures to provide results which satisfy the aim of step 4 above.

Figure 5 - Suggested Procedure for Obtaining c' and ϕ' of
a Soil from Laboratory Test Data

Figure 6 - Empirical Method for Estimating Relative Density of a Sand from
Standard Penetration Test 'N' Values
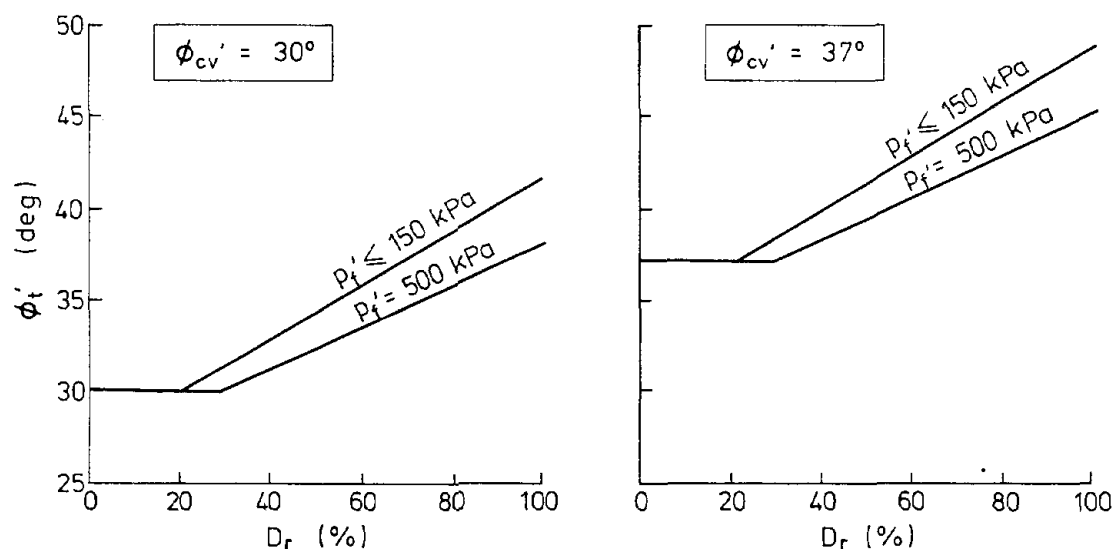
ϕ_t' = Angle of shearing resistance of the sand obtained from triaxial compression test corresponding to a stress level of p_f' at failure

$\quad = \phi_{cv}' + 3I_R$

where $\quad I_R = D_r \left\{ 5 - \ln \left(\dfrac{p_f'}{150} \right) \right\} - 1 \quad$ but $\not< 0$, for $p_f' > 150$ kPa

$\quad\quad = 5D_r - 1 \quad$ but $\not< 0$, for $p_f' \leq 150$ kPa

D_r = Relative density of the sand

p_f' = Mean effective stress at failure, may be taken as the effective normal stress at the point on the failure envelope for which ϕ_t' is calculated

ϕ_{cv}' = Constant volume or critical state angle of shearing resistance of the sand; depends on angularity and stress level. Design values may be assumed to be between 30° (for rounded grains) and 37° (for angular grains)

Notes : (1) The above empirical method is based on Bolton (1986), which contains a summary of ϕ_{cv}' values for a range of sands.

(2) The relative density of a sand may be determined from empirical correlation with SPT 'N' values, e.g. as given in Figure 6.

(3) Reference should be made to Section 5.5.6 for discussion on the selection of ϕ' values for design.

Figure 7 - Empirical Method for Estimating ϕ' of a Sand from Relative Density
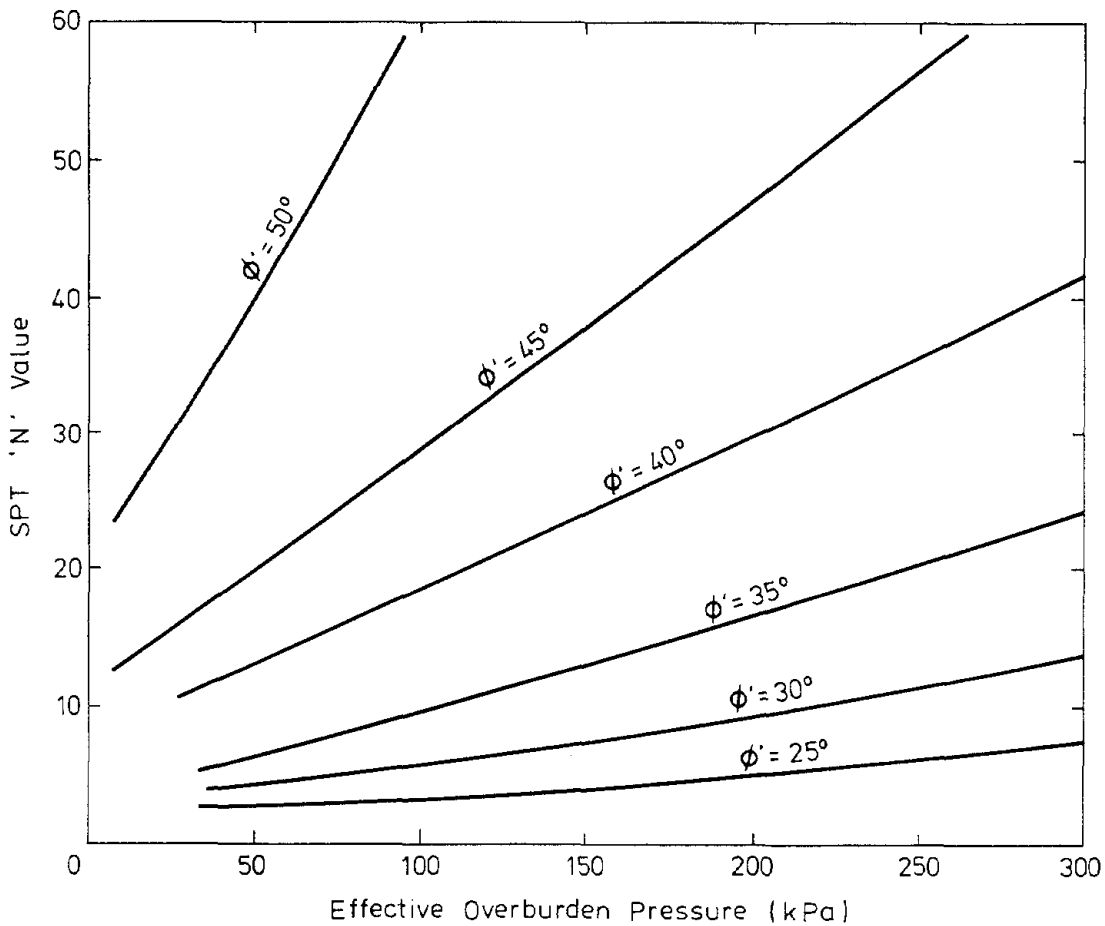
Notes : (1) Figure taken from Schmertmann (1975).

(2) In using this figure, the effective overburden pressure should correspond to the point at which the SPT was carried out. The point estimates should then be used to assess the average ϕ' value of the sand stratum.

(3) Reference should be made to Section 5.5.6 for discussion on the selection of ϕ' values for design.

Figure 8 - Empirical Method for Estimating ϕ' of a Sand from Standard Penetration Test 'N' Values

Soil layer I - N_1

Δz_1

Soil layer II - N_2

Δz_2

Soil layer III - N_3

Δz_3
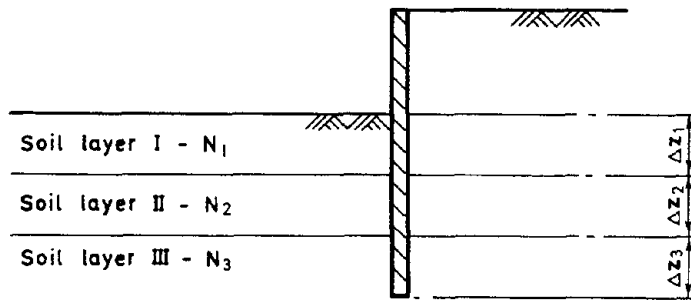
(a) Ground Profile and Properties

Retaining wall modelled
as elastic beam ⟶

Spring 1 Δz_1 k_{S1}

Spring 2 Δz_2 k_{S2}

Spring 3 Δz_3 k_{S3}

N_1 z_1

N_2 z_2

N_3 z_3

Ground modelled as elasto-plastic springs
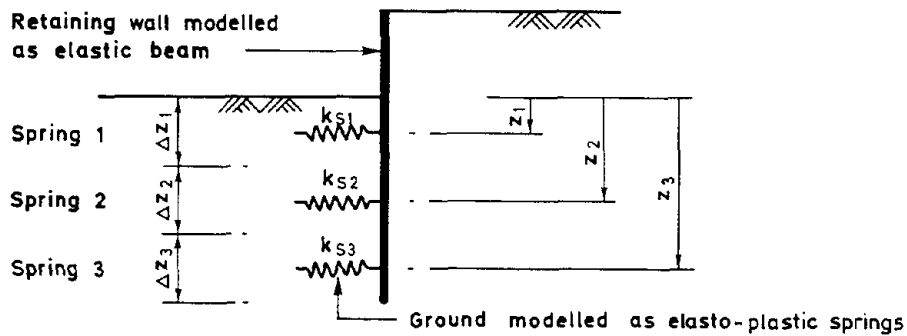
(b) Model for Subgrade Reaction Analysis

Method 1 : k_s Based on m_h or n_h

(i)   Obtain a representative m_h or n_h value for the soil layers (see Section 5.6.3)

(ii)   For 'wall' analysis[2], $k_{hj} = m_h z_j / d$
For 'pile' analysis[2], $k_{hj} = n_h z_j / B$

(iii)   For 'wall' analysis[2], $k_{sj} = k_{hj} \Delta z_j = m_h z_j \Delta z_j / d$
For 'pile' analysis[2], $k_{sj} = B k_{hj} \Delta z_j = n_h z_j \Delta z_j$

Method 2 : k_s Based on E_s'

(i)   Obtain E_{sj}' based on empirical correlations with N values (see Section 5.6.2)[5] or plate loading tests

(ii)   Obtain K_{hj} by assuming $K_{hj} = 0.8$ to $1.8 \ E_{sj}'$ [5] (see Section 5.6.3)

(iii) $k_{sj} = K_{hj} \Delta z_j$

Legend :			
m_h, n_h	Constants of horizontal subgrade reaction	N_j	SPT 'N' values
k_{sj}	Spring constants	E_s'	Young's modulus of soil
B	Width of pile	K_{hj}	Modulus of horizontal subgrade reaction
d	Thickness of soil at the passive side of the wall	Δz_j	Length of beam served by spring
		z_j	Depth of spring

Notes :

(1)   Consistent units should be used in the equations given in this figure (see Section 5.6.3).

(2)   For a sheet wall or a wall composed of closely-spaced piles, the structure should be treated as a wall in the analysis. For a wall composed of widely-spaced piles, individual piles should be considered.

(3)   If a spring is found to act in tension (e.g. k_{s3}), then this means that the resistance is from the side of the retained soil, in which case the depth of the spring (viz. z_3) should be taken from the surface of the retained ground.

(4)   Method 2 should only be used for walls composed of widely-spaced piles.

(5)   For a rock layer, the modulus of deformation E_m should be used in place of E_s'. Reference should be made to Section 5.9 for guidance on E_m values.

Figure 9 - Alternative Methods of Deriving Values of Spring Constants
for Subgrade Reaction Analysis

Sieve Aperture Size					
μm			mm		

Curve	Coefficient of Permeability, k (m/sec)
①	4×10^{-1}
②	3×10^{-1}
③	3×10^{-2}
④	7×10^{-4}
⑤	5×10^{-5}
⑥	1×10^{-2}
⑦	9×10^{-3}
⑧	4×10^{-3}
⑨	1×10^{-3}
⑩	4×10^{-4}
⑪	5×10^{-5}
⑫	2×10^{-5}
⑬	5×10^{-6}

Fine	Medium	Coarse	Fine	Medium	Coarse
SAND			GRAVEL		

(a) Coefficient of Permeability for Selected Clean Coarse-grained Drainage Materials

(b) Effect of Fines on Permeability

Notes : (1) Figure based on NAVFAC (1982a) and Hong Kong data.

(2) Curves ⑫ and ⑬ in (a) and the corresponding data in (b) are for filter materials composed of granitic quarry fines compacted to 95% of their maximum dry density.

Figure 10 - Approximate Values of Permeability of Granular Filter and Drainage Materials

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Figure 11 - Mohr Circles for Equilibrium States in Soil

Soil Type	Movement for the Development of Failure Conditions, $\frac{y}{H}$	
	Active Case	Passive Case
Dense sand	0.001	0.02 – 0.06
Loose sand	0.005	0.06 – 0.35
Stiff clay	0.01	0.02
Soft clay	0.02	0.04

Notes : (1) Figure based on data given by Canadian Geotechnical Society (1985) and Rowe & Peaker (1965).

(2) Figure only applicable for rigid walls and soil with K_o less than unity.
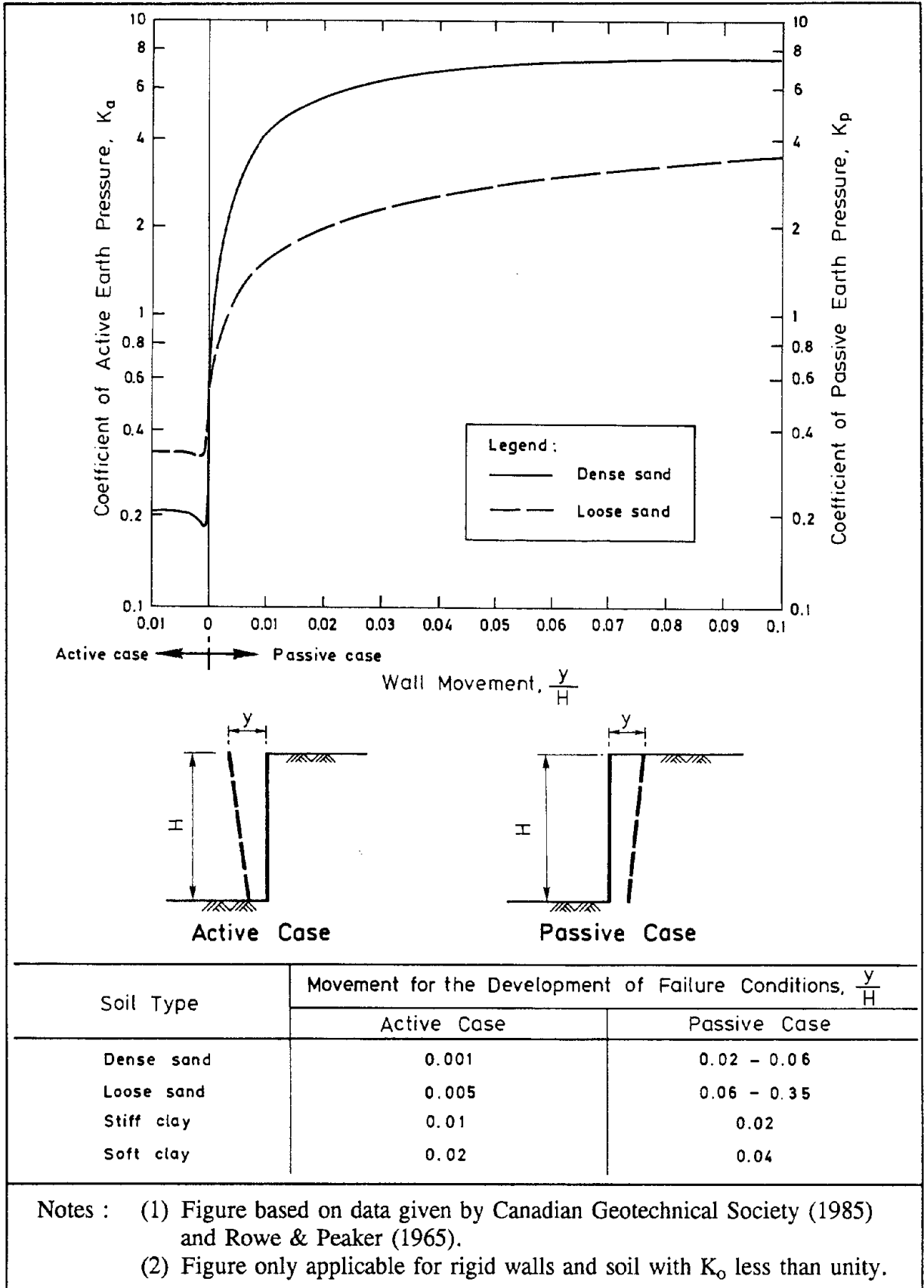
Figure 12 - Approximate Values of Retaining Wall Movements for the Development of Active and Passive Failures in Soil

(i) Active State

(ii) Passive State
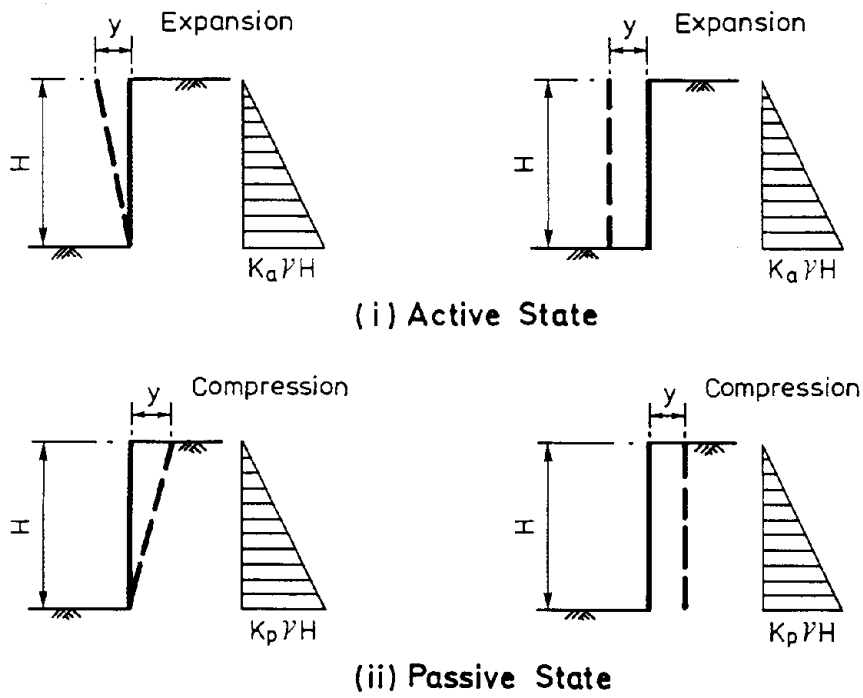
(a) Rigid Retaining Wall Free to Translate or Rotate about Its Base

(b) Restrained Rigid Wall

(c) Top of Wall Restrained

(d) Effects of Compaction

(e) Strutted Flexible Wall
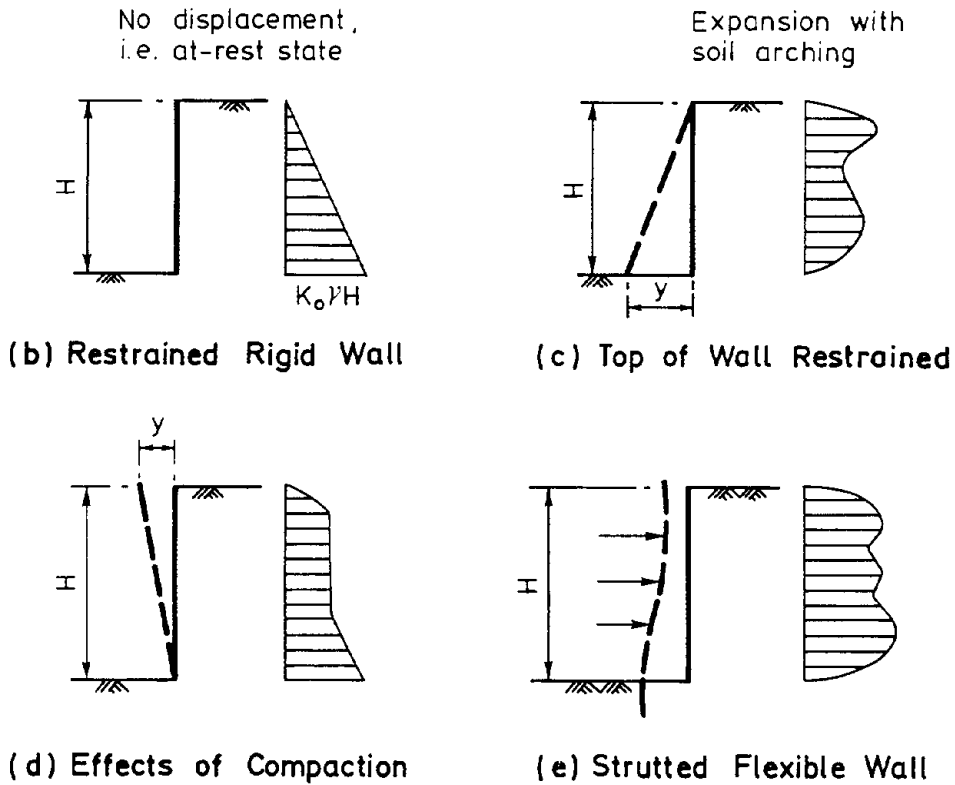
Note :   Figure based on Canadian Geotechnical Society (1985).

Figure 13 - Idealised Relationships Between Wall Movement and Earth Pressure
Distribution for Different Types of Retaining Walls
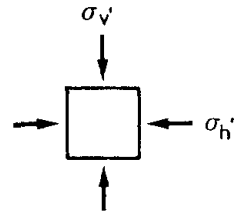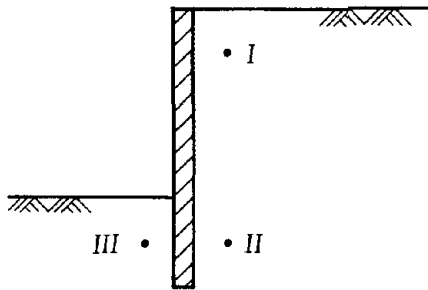
Soil Zones Near a Cantilevered Wall
Retaining Insitu Ground
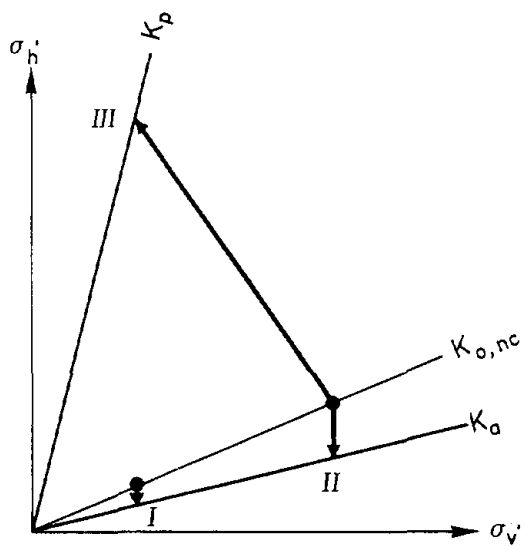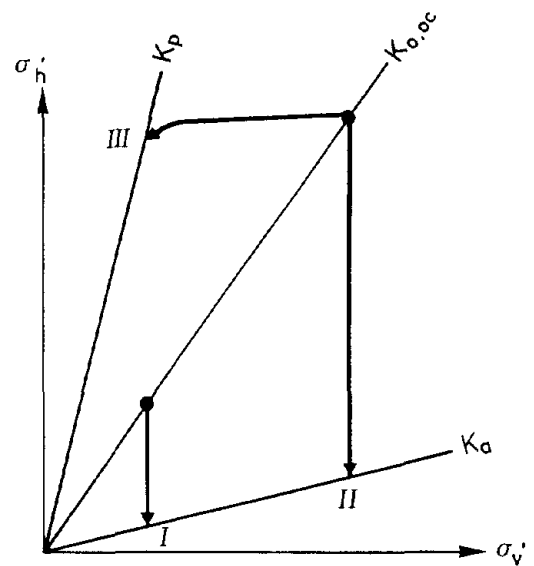
(a) Soil without Precompression   (b) Precompressed Soil

Note :   Figure adapted from Padfield & Mair (1984).

Figure 14 - Stress Paths for Soil Elements Near a Cantilevered Wall
Retaining Insitu Ground
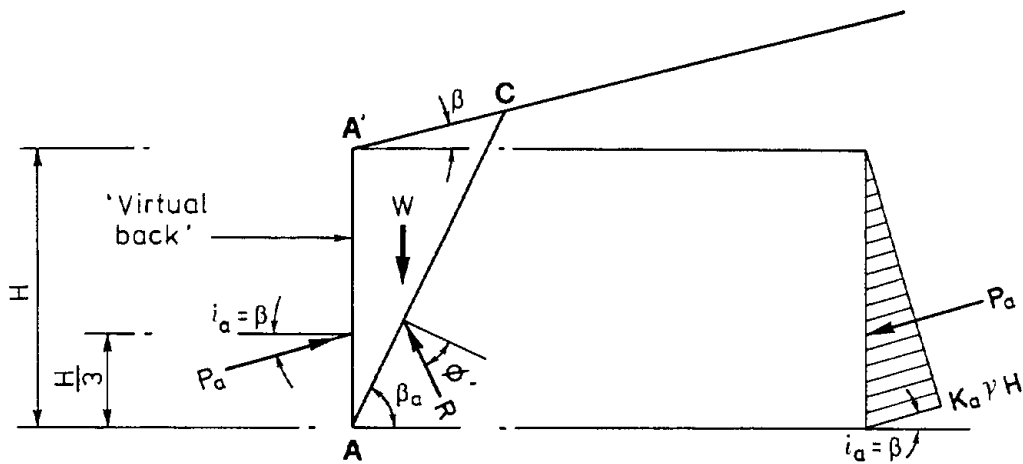
$$P_a = 0.5\,K_a\,\gamma H^2$$

$$\text{where } K_a = \cos\beta\left(\frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi'}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi'}}\right)$$

$$\beta_a = 45° + \frac{\phi'}{2} - \frac{1}{2}(\epsilon - \beta)$$

$$\sin\epsilon = \frac{\sin\beta}{\sin\phi'} \qquad (0 < \epsilon < 90°)$$

$$\text{For } \beta = 0, \quad K_a = \frac{1 - \sin\phi'}{1 + \sin\phi'} \quad \text{and} \quad \beta_a = 45° + \frac{\phi'}{2}$$

(a) Rankine Active Earth Pressure on Vertical Plane AA'
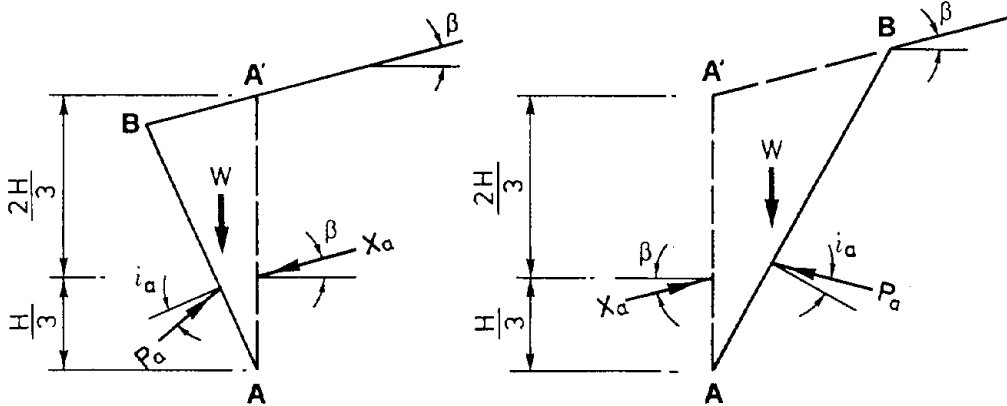
The active force P_a and its angle of inclination[1] may be obtained by the resolution of forces comprising P_a, X_a and W (the weight of the soil wedge AA'B). The value of X_a should be calculated by using the equation for P_a given in (a) above. The earth pressure distribution with depth is triangular, with P_a acting at the lower third point of BA.

(b) Rankine Active Earth Pressure on Non-vertical Planes

Note : (1) The angle of inclination i_a is determined by the condition that the line of action of X_a on plane AA' must be parallel to the ground slope β. Rankine theory should not be applied where i_a exceeds the angle of wall friction which can be mobilised at failure, after taking into consideration the anticipated movement of the wall at failure (see Section 5.11).

Figure 15 - Calculation of Active Earth Pressure for Vertical and Non-vertical Planes Using Rankine Theory

Surcharge, q

Depth of tension zone, z_o

$P_a = \bar{\gamma}_w z_o$

$P_a = K_{a1}(q + \bar{\gamma}_1 z) - 2c_1\sqrt{K_{a1}}$

$P_a = K_{a1}\left[q + \bar{\gamma}_1 h_w + \gamma_1(z - h_w) - u_1\right] - 2c_1'\sqrt{K_{a1}}$

where $u_1 = \gamma_w(z - h_w)$

$P_a = K_{a2}\left[q + \bar{\gamma}_1 h_w + \gamma_1 H_1 + \gamma_2(z - h_w - H_1) - u_2\right] - 2c_2'\sqrt{K_{a2}}$

where $u_2 = \gamma_w(z - h_w)$

$\gamma_w(H_1 + H_2)$

z

Assumed impermeable lower boundary

$\bar{\gamma}_1, c_1', \phi_1'$

$K_{a1} = \dfrac{1 - \sin\phi_1'}{1 + \sin\phi_1'}$

γ_1, c_1', ϕ_1'

γ_2, c_2', ϕ_2'

$K_{a2} = \dfrac{1 - \sin\phi_2'}{1 + \sin\phi_2'}$

(a) Hydrostatic Pressures (b) Active Earth Pressures (c) Soil Properties

Notes : (1) The depth of the tension zone z_0 may be calculated using equation (6.7). Negative earth pressures within this zone should be ignored. Water in the tension crack is assumed to exert a hydrostatic pressure.

(2) $\bar{\gamma}_1$ is the soil unit weight above the water table, and γ_1 & γ_2 are the saturated unit weights of soil below the water table.

(3) Water pressure is assumed to be hydrostatic, which implies that the retaining wall and the lower boundary are impermeable. For non-hydrostatic conditions and the presence of perched water tables, the effect of pore water pressures should be properly assessed.

Figure 16 - Calculation of Active Earth Pressure for a Vertical
Retaining Wall Using the Rankine-Bell Equation
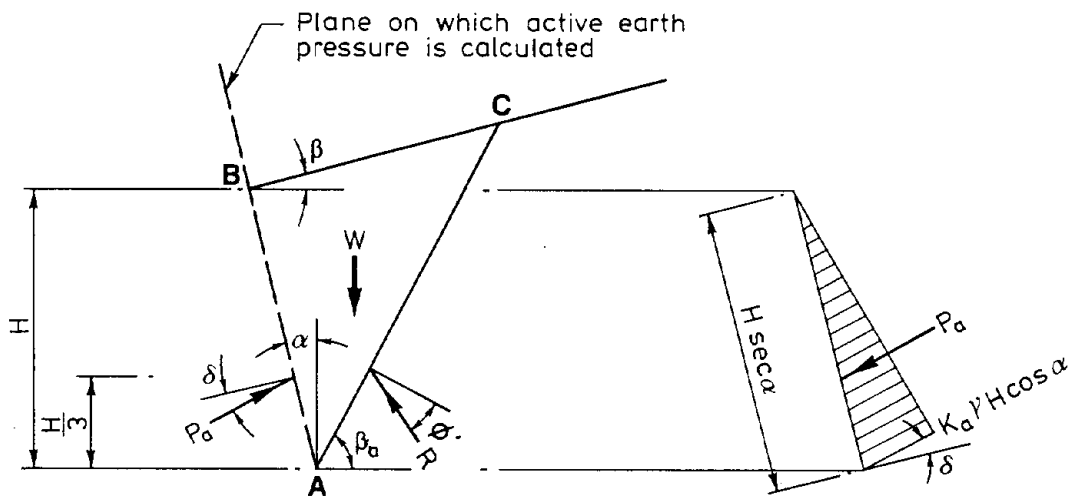
$$P_a = 0.5 K_a \gamma H^2$$

$$\text{where} \quad K_a = \frac{\cos^2(\phi' - \alpha)}{\cos^2\alpha \, \cos(\delta + \alpha)\left[1 + \sqrt{\dfrac{\sin(\phi' + \delta)\,\sin(\phi' - \beta)}{\cos(\delta + \alpha)\,\cos(\beta - \alpha)}}\,\right]^2}$$

$$\cot(\beta_a - \beta) = \sec(\phi' + \delta + \alpha - \beta)\sqrt{\frac{\sin(\phi' + \delta)\,\cos(\delta + \alpha)}{\sin(\phi' - \beta)\,\cos(\beta - \alpha)}} \; - \; \tan(\phi' + \delta + \alpha - \beta)$$

For $\alpha = 0$ and $\delta = \beta$, K_a = Rankine's value given in Figure 15 (a).

Notes :
(1) Generally, the Coulomb equation above gives only an approximate solution for the earth pressure. This is because static equilibrium is usually not fully satisfied, i.e. the forces P_a, R and W (the weight of the soil wedge), are usually not concurrent and hence there is no moment equilibrium. The departure from the 'exact' solution is usually very small for the active case but passive earth pressure determined using Coulomb theory may be dangerously overestimated (see Section 6.6).

(2) The value of mobilised angle of wall friction δ has to be assumed in the calculation. Consideration should be given to the anticipated movement of the wall at failure in selecting the value of δ for design (see Section 5.11).

Figure 17 - Calculation of Active Earth Pressure Using Coulomb Theory
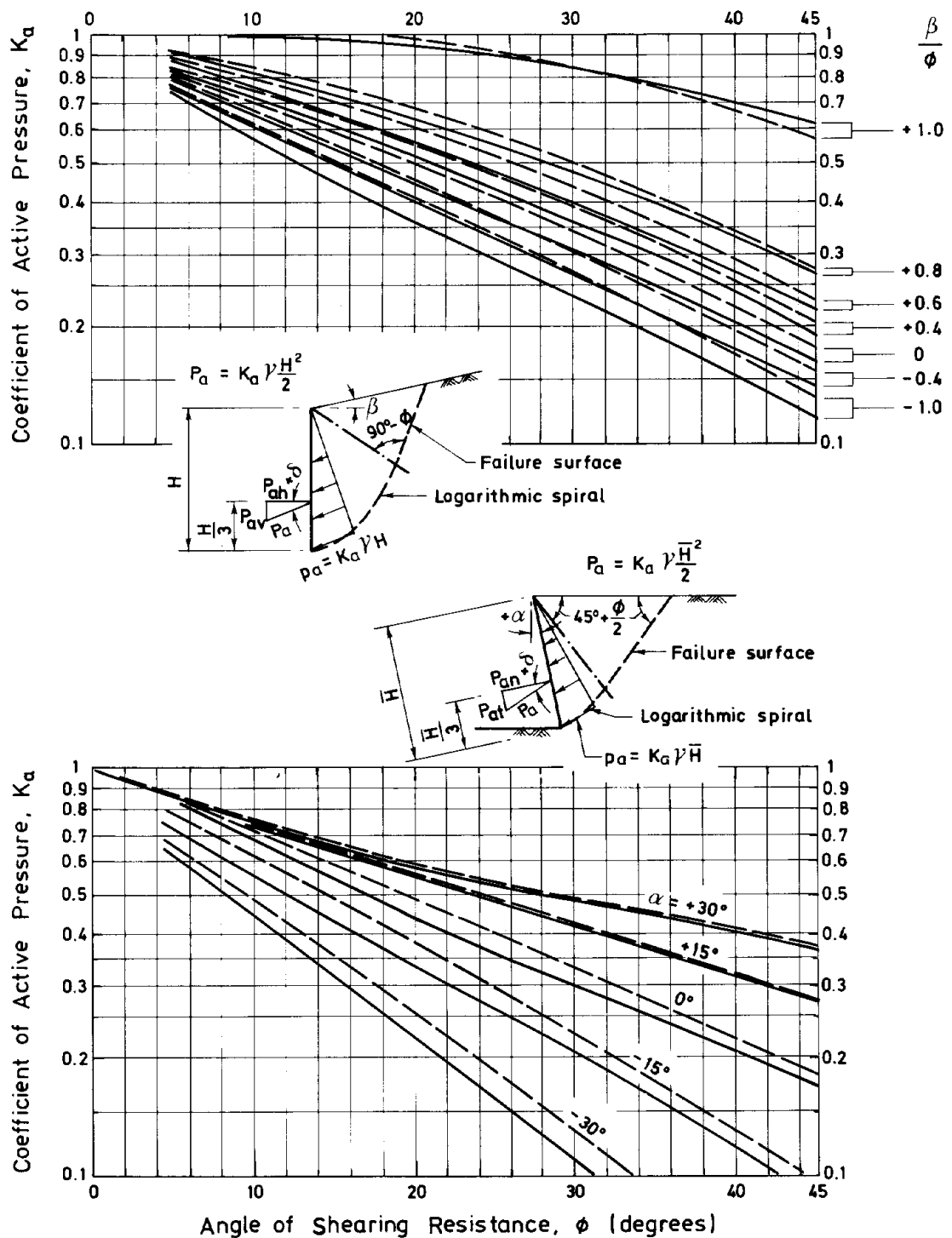
Figure 18 - Active Earth Pressure Coefficients for Different Wall
Configurations and Retained Slope Angles

Reduction Factor, R_d

0   10   20   30   40   45

$\frac{\delta}{\phi} = -0.7$

−0.6
−0.5
−0.4
−0.3
−0.2
−0.1
0

$+\beta$

90° − ϕ Failure surface

P_{pv} P_p

P_{ph}

Logarithmic spiral

$P_p = K_p \, \gamma H$

$P_p = K_p \, \gamma \frac{H^2}{2}$; $P_{ph} = P_p \cos \delta$; $P_{pv} = P_p \sin \delta$

Curves shown are for $\frac{\delta}{\phi} = -1$

Example :

$\phi = 35°$; $\frac{\beta}{\phi} = -0.2$; $\frac{\delta}{\phi} = -0.3$

K_p for $\frac{\delta}{\phi}$ of −1 = 7.4

Reduction factor $R_d = 0.54$

K_p for $\frac{\delta}{\phi}$ of −0.3 = $R_d \times K_p$ (for $\frac{\delta}{\phi}$ of −1)

                 = 0.54 × 7.4 = 4.0

Coefficient of Passive Pressure, K_p

$\frac{\beta}{\phi} = +1$ +0.8   +0.6   +0.4   +0.2   0   −0.2   −0.4   −0.6   −0.8   −0.9   −1.0

Angle of Shearing Resistance, ϕ (degrees)

Note :   Figure based on NAVFAC (1982b) and Caquot & Kerisel (1948).

Figure 19 - Passive Earth Pressure Coefficients for a Vertical Wall
Retaining Sloping Ground

Note :   Figure based on NAVFAC (1982b) and Caquot & Kerisel (1948).

Figure 20 - Passive Earth Pressure Coefficients for a Sloping
Wall Retaining Horizontal Ground

(a) Trial Wedges

(b) Force Polygon for Typical Wedge

(c) Combination of Force Polygons to Obtain Maximum P_a

PROCEDURE :

1. Divide the ground into a series of wedges by selecting potential planar failure surfaces 1, 2, 3, 4 etc.

2. Evaluate the weight W and the resultant forces due to water pressure U_1 and U_2 acting on each wedge AB1, AB2 etc.

3. Construct the force polygon for each wedge, taking into account the line of action of forces P and R ; the direction of these forces for both the active and passive cases is shown in (a) above for wedge AB2. The water forces act in a direction normal to the respective surfaces.

4. Combine all the force polygons to obtain the limiting resultant force required. For the passive case, the minimum value of P_p is obtained by interpolating between the values from the force polygons constructed.

Notes : (1) Figure based on New Zealand Ministry of Works and Development (1973).
(2) The trial wedge method makes the same assumption of planar failure surface as the Coulomb theory. There are limitations in the use of the method for the evaluation of passive earth pressures (see Section 6.6). Care should be taken in selecting the value of δ for design (see Section 6.5.4).
(3) The point of application of the resultant active force and the earth pressure distribution on plane AB may be determined using the methods illustrated in Figures 24 and 25 respectively.

Figure 21 - Illustration of the Trial Wedge Method with Hydrostatic Pressure for a Soil Modelled as a c' = 0 Material

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(a) Trial Wedges

(b) Force Polygon for Typical Wedge

(c) Combination of Force Polygons to Obtain Maximum P_a

PROCEDURE :

Same as that given in Figure 21 except that the depth of tension zone z_0 and the hydrostatic pressure P_{w1} and P_{w2} have to be determined (see Section 6.5.6).

Notes :   (1)   Notes (1) to (3) of Figure 21 are also applicable.
          (2)   Adhesion to the back of the retaining wall is ignored.

Figure 22 - Illustration of the Trial Wedge Method with Hydrostatic Pressure for a Soil Modelled as a c' - ϕ' Material

(a) Geometry, Loadings and Parameters

(b) Force Polygon for Trial Wedge I

(c) Force Polygons for Trial Wedge II

(d) Final Pressure Distribution

PROCEDURE :

1. Draw trial wedge I in layer I. Evaluate the forces W_1, q_1 and $c_1'l$ and construct the force polygon as shown in (b) above. By varying the inclination of the failure plane, obtain maximum P_{a1} using the procedure given in Figure 22.

2. Draw trial wedge II by choosing a failure plane AB in layer II (see (c) above).

3. By varying the inclination of plane BC in layer I, obtain maximum X_a using the procedure given in Figure 22. The resultant force U_1 on plane BC due to water pressure should be determined by a flow net analysis.

4. Using the maximum value of X_a obtained in step 3, construct the force polygon for the soil mass above AB as shown in (c) above. The water forces U_2 and U_3 should also be determined from the flow net analysis.

5. Repeat steps 2 to 4 using other trial failure planes such as AB' until maximum P_{a2} is obtained.

Notes : (1) Notes (1) to (3) of Figure 21 are also applicable.
(2) Where layer II is a rock mass, due account should be taken of discontinuity-controlled failure modes and water pressures.

Figure 23 - Illustration of the Trial Wedge Method with Groundwater
Seepage for a Layered Soil Mass

PROCEDURE :

1. Draw a line through the C.G. of wedge AA"CDEF parallel with the pre-determined failure plane to intersect plane AA" at point X. (For a constant ground slope, AX = 1/3 AA"). For a soil modelled as a c' = 0 material, the wedge between the failure plane and the ground surface should be used.

2. Draw a line through point X parallel to BG and a vertical line through the C.G. of wedge ABA' to intersect at point Z.

3. P_a may be assumed to act through point Z, making an angle δ to the normal to plane AB on which the active force is calculated.

Notes : (1) Figure based on New Zealand Ministry of Works and Development (1973).

(2) This method may be used for an irregular ground surface which does not depart significantly from a plane surface and which does not support non-uniform surcharges. Where this is not the case, or where the ground consists of two or more soil layers, the general method illustrated in Figure 25 should be used. The general method should also be used where water forces are present.

(3) If the active force is calculated on a vertical plane, steps 2 and 3 are unnecessary as P_a acts through point X.

(4) For the depth of tension zone z_0, see Section 6.5.3.

Figure 24 - Approximate Method for the Evaluation of the Line of Action of Active Force

Trial Wedges   Earth Pressure on AB
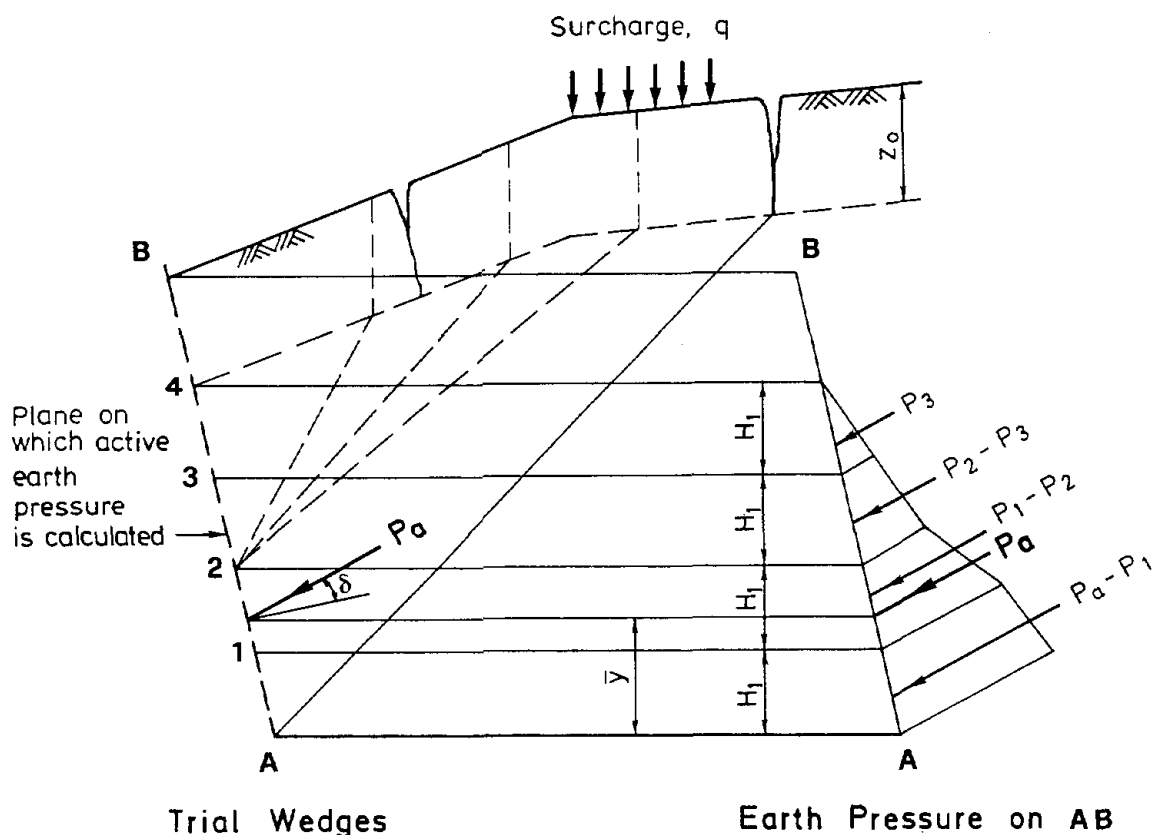
PROCEDURE :

1. Subdivide the line A4 into about four equal parts of depth h_1 from below the tension zone.

2. Compute the active forces P_1, P_2, P_3, etc. as if each of the points 1, 2, 3 etc. were the base of the retaining wall. The trial wedge method may be used for each computation.

3. Determine the pressure distribution by working down from point 4. A linear variation of pressure may be assumed between the points where pressure has been calculated.

4. Determine the elevation of the centroid of the pressure diagram, $\bar{y}$. This can be taken to be the approximate elevation of the point of application of the resultant active force P_a, which makes an angle δ to the normal to plane AB.

Notes : (1) Figure based on New Zealand Ministry of Works and Development (1973).

(2) This method should be used for a layered soil mass, where the ground surface is irregular or where there are non-uniform surcharges.

(3) Water forces must be considered separately.

(4) For the depth of tension zone z_0, see Section 6.5.3.

Figure 25 - General Method for the Evaluation of Active Earth Pressure Distribution

PROCEDURE :

(c) Force Polygon for Circular Arc Wedge

1. Determine the directions of the assumed surface of sliding BA' and the plane portion of the failure surface A'E using the following formulae :

$$\alpha_{a1} = 45° + \frac{\phi'}{2} + \frac{1}{2}(\epsilon + \beta) \qquad \alpha_{a2} = 45° + \frac{\phi'}{2} - \frac{1}{2}(\epsilon + \beta)$$

where $\sin \epsilon = \dfrac{\sin \beta}{\sin \phi'}$

In the above formulae, the value of ϕ' may be assumed to be the lesser of ϕ_1' and ϕ_2' to give a conservative estimate of the passive force P_p.

2. For a particular position of A', construct A'C perpendicular to A'E at A'. Then produce a perpendicular bisector OP cutting A'C at O, and draw arc AA' with O as centre.

3. Evaluate the weights W_1, W_2 and W_3 and the resistance due to $c_1'l_1$, $c_2'l_2$ and $c_2'(\overline{AA'})$. Determine the forces U_1, U_2 and U_3 due to water pressure from a flow net analysis. Then construct the force polygons as illustrated in (b) and (c) above, with the water forces acting normal to the respective surfaces.

4. Determine the force P_p from the force polygons. Draw the locus of P_p as shown in (a) above for different trial positions of A'.

5. Repeat steps 2 to 4 until the minimum value of P_p is found.

Notes : (1) Figure based on NAVFAC (1982b). Care should be taken in selecting the value of δ for design (see Section 6.6).

(2) The point of application of the resultant passive force and the earth pressure distribution on plane AB may be determined based on the principle illustrated in Figure 25.

(3) Where layer II is a rock mass, due account should be taken of discontinuity-controlled failure modes and water pressures.

Figure 26 - The Circular Arc Method for the Evaluation of Passive Earth Pressure
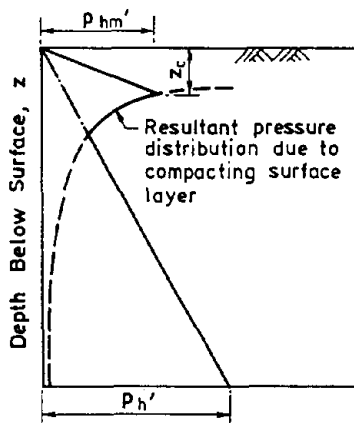
(a) Horizontal Earth Pressure Distribution in Uncompacted Fill Resulting from Compaction of Surface Layer Only
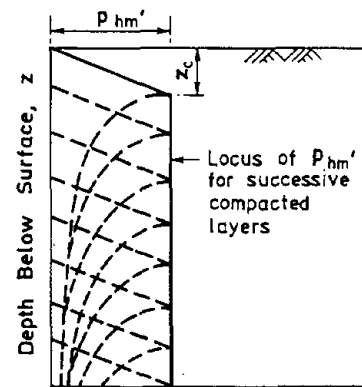
(b) Horizontal Earth Pressure Distribution Resulting from Successively Compacted Layers of Fill
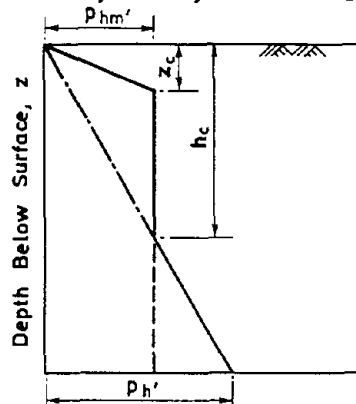
(c) Design Diagram for Horizontal Earth Pressure Induced by Compaction

$$P_{hm}' = \sqrt{\frac{2Q_1\gamma}{\pi}}$$

$$P_h' = K\gamma z$$

$$z_c = K\sqrt{\frac{2Q_1}{\pi\gamma}}$$

$$h_c = \frac{1}{K}\sqrt{\frac{2Q_1}{\pi\gamma}}$$

Legend :

K	Earth pressure coefficient[2]
Q_1	Intensity of effective line load imposed by compaction plant[3]
z_c, h_c	Critical depths as shown
γ	Soil unit weight
P_{hm}'	Maximum horizontal earth pressure induced by compaction
P_h'	Horizontal earth pressure induced by overburden stress

Notes :

(1) Figure based on Ingold (1979a & b).

(2) For retaining walls which can move forward sufficiently to mobilise active condition in the fill, $K = K_a$. For unyielding rigid structures, $K = K_o$. For walls supporting a fill slope, it may be assumed that the compaction-induced earth pressure is the same as that given by the diagram in (c) above for a horizontal final surface, except z_c should be taken as zero.

(3) For dead weight rollers, the effective line load is the weight of the roller divided by its roll width, and for vibratory rollers it should be calculated using an equivalent weight equal to the dead weight of the roller plus the centrifugal force generated by the roller's vibrating mechanism. The centrifugal force may be taken to be equal to the dead weight of the roller in the absence of trade data.

(4) The compaction-induced earth pressure assumed in the design should be clearly stated on the drawings.

Figure 27 - Simplified Method for the Evaluation of Compaction-induced Earth Pressures

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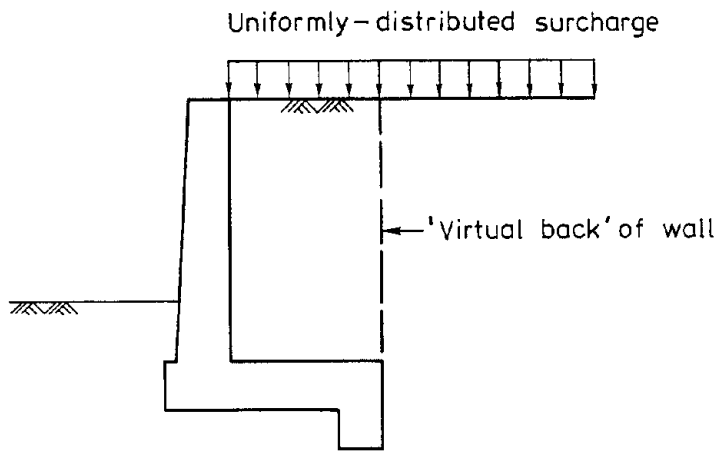
(a) Load Case 1: Critical for Foundation and Structural Design
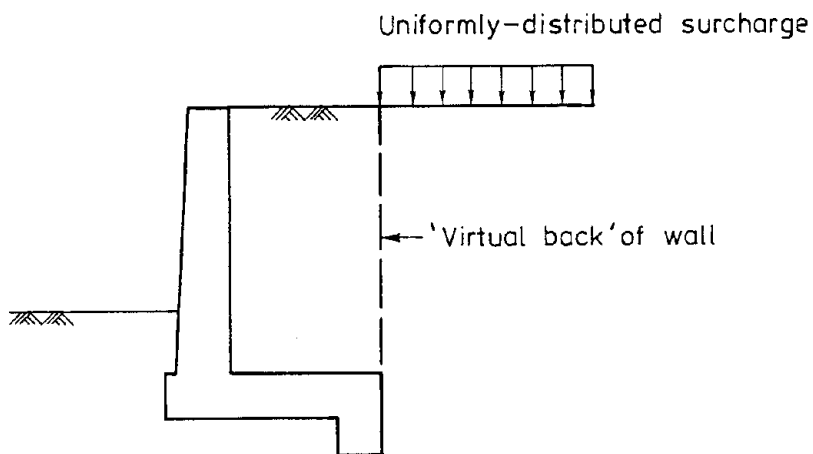
(b) Load Case 2 : Critical for Stability

Note :   Figure based on New Zealand Ministry of Works and Development (1973).

Figure 28 - Example of Load Cases for a Uniformly-Distributed Surcharge for the Design of L-shaped Cantilever Retaining Walls

Line load, Q_l

$x = mH$

$z = nH$

σ_h

P_h

y_p

Point load, Q_p

$x = mH$

$z = nH$

σ_h

P_h

y_p

SECTION A-A

Wall

σ_h

θ

$\sigma_{h\theta}$

Q_p

$x = mH$

Line load, Q_h

x

z

σ_h

P_h

H_e

For $m \leqslant 0.4$

$$\sigma_h \left(\frac{H}{Q_l}\right) = \frac{0.20n}{(0.16 + n^2)^2}$$

$$P_h = 0.55 Q_l$$

For $m > 0.4$

$$\sigma_h \left(\frac{H}{Q_l}\right) = \frac{1.28 m^2 n}{(m^2 + n^2)^2}$$

$$P_h = \frac{0.64 Q_l}{(1 + m^2)}$$

For $m \leqslant 0.4$

$$\sigma_h \left(\frac{H^2}{Q_p}\right) = \frac{0.28 n^2}{(0.16 + n^2)^3}$$

$$P_h = 0.69 Q_p$$

For $m > 0.4$

$$\sigma_h \left(\frac{H^2}{Q_p}\right) = \frac{1.77 m^2 n^2}{(m^2 + n^2)^3}$$

$$P_h = 0.48 Q_p \left[\frac{m(1 - m^2)}{(1 + m^2)^2} + \tan^{-1}\left(\frac{1}{m}\right)\right]$$

$$\sigma_{h\theta} = \sigma_h \cos^2(1.1\theta)$$

$$H_e = x \tan\left(45° + \frac{\phi'}{2}\right)$$

$$\sigma_h \left(\frac{H_e}{Q_h}\right) = 2\left(1 - \frac{z}{H_e}\right)$$

Resultant $P_h = Q_h$

(a) Lateral Pressure on
Wall Due to
Vertical Line Load, Q_l

(b) Lateral Pressure on
Wall Due to
Vertical Point Load, Q_p

(c) Lateral Pressure on
Wall Due to
Horizontal Line
Load, Q_h

Symbol	m	$\frac{P_h}{Q_l}$	$\frac{y_p}{H}$
– – –	0.4	0.55	0.61
——	0.5	0.51	0.56
······	0.6	0.47	0.52
–·–·–	0.7	0.43	0.49

Value of $n = \frac{z}{H}$

Value of $\sigma_h \left(\frac{H}{Q_l}\right)$

(d) Pressure Distribution Due to
Vertical Line Load, Q_l

Symbol	m	$\frac{P_h}{Q_p}$	$\frac{y_p}{H}$
– – –	0.4	0.69	0.58
——	0.5	0.65	0.52
······	0.6	0.59	0.47

Value of $\sigma_h \left(\frac{H^2}{Q_p}\right)$

(e) Pressure Distribution Due to
Vertical Point Load, Q_p

Note : Figure based on Terzaghi (1953).

Figure 29 - Calculation of Lateral Pressure on a Vertical Retaining
Wall Due to Vertical and Horizontal Loads
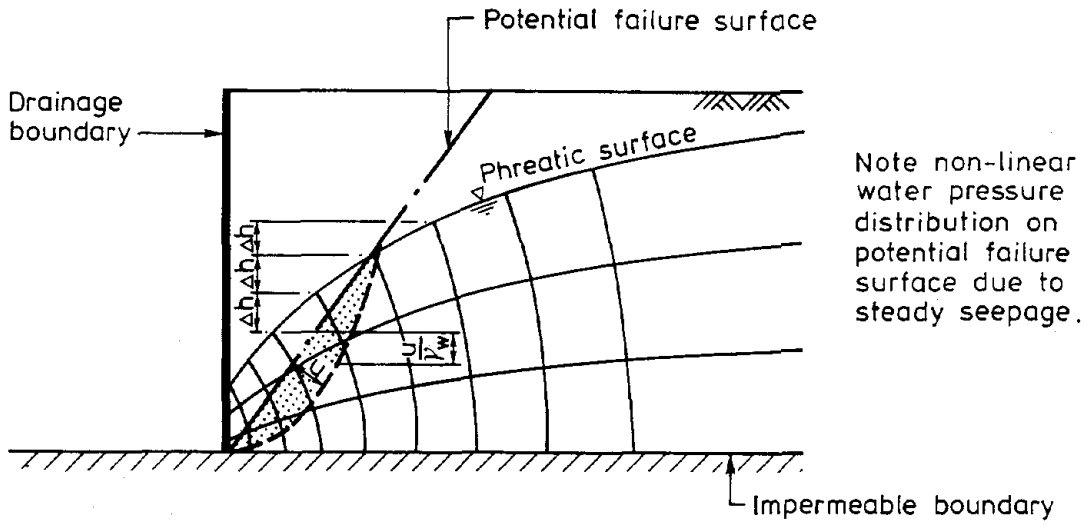
(a) Groundwater Seepage Condition
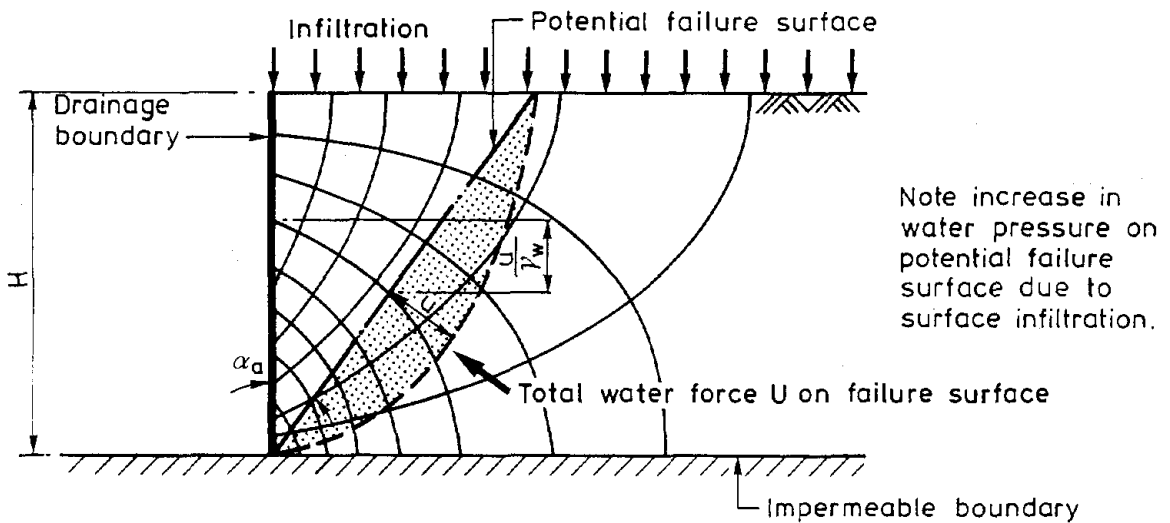
(b) Surface Infiltration
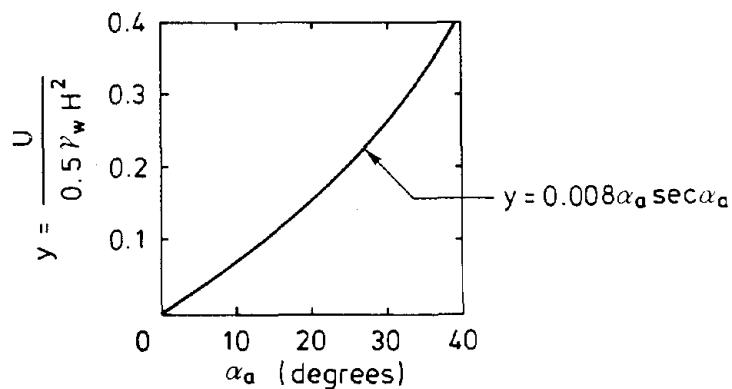
(c) Variation of U with α_a for Condition (b) Above

Note :   Figure (c) based on NAVFAC (1982b).

Figure 30 - Evaluation of Water Pressure on Potential Failure Surfaces
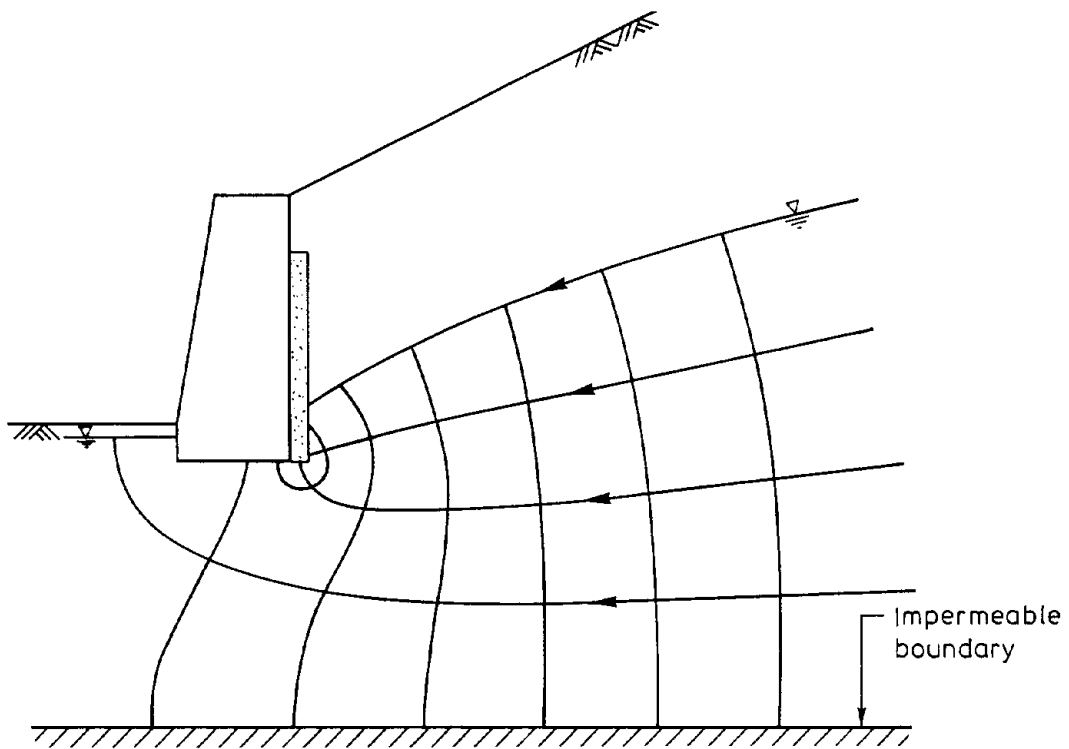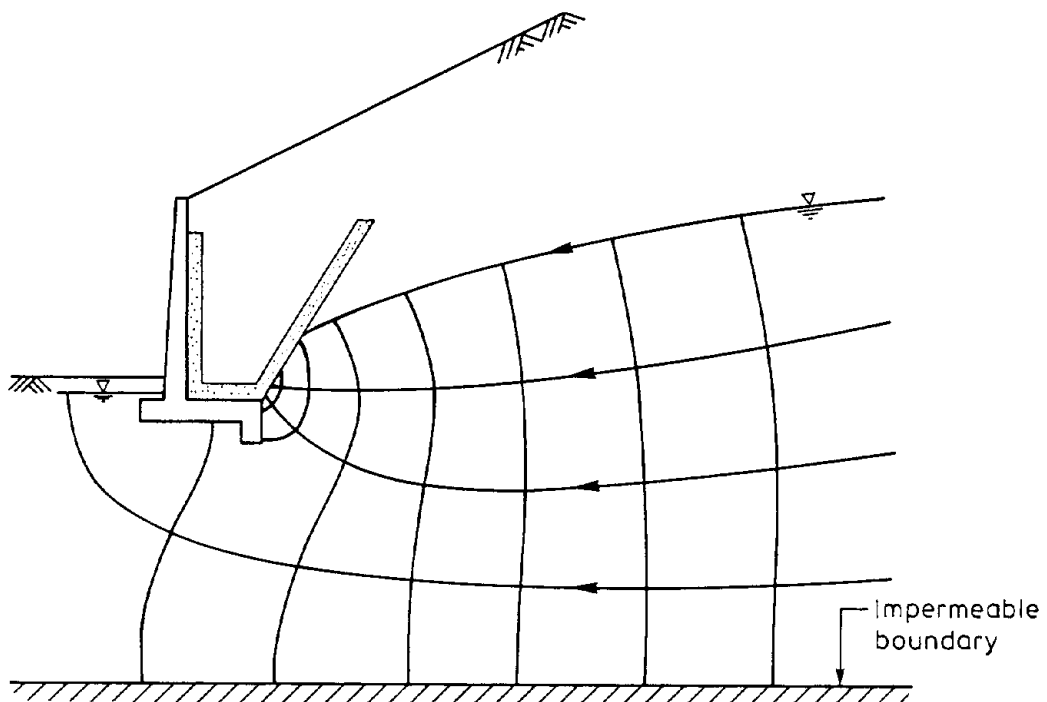
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(a) Gravity Retaining Wall

(b) Inverted T-shaped Retaining Wall with Shear Key

Note :   A homogeneous isotropic soil condition is assumed.

Figure 31 - Flow-nets Around Retaining Walls Under Steady-state
Groundwater Seepage
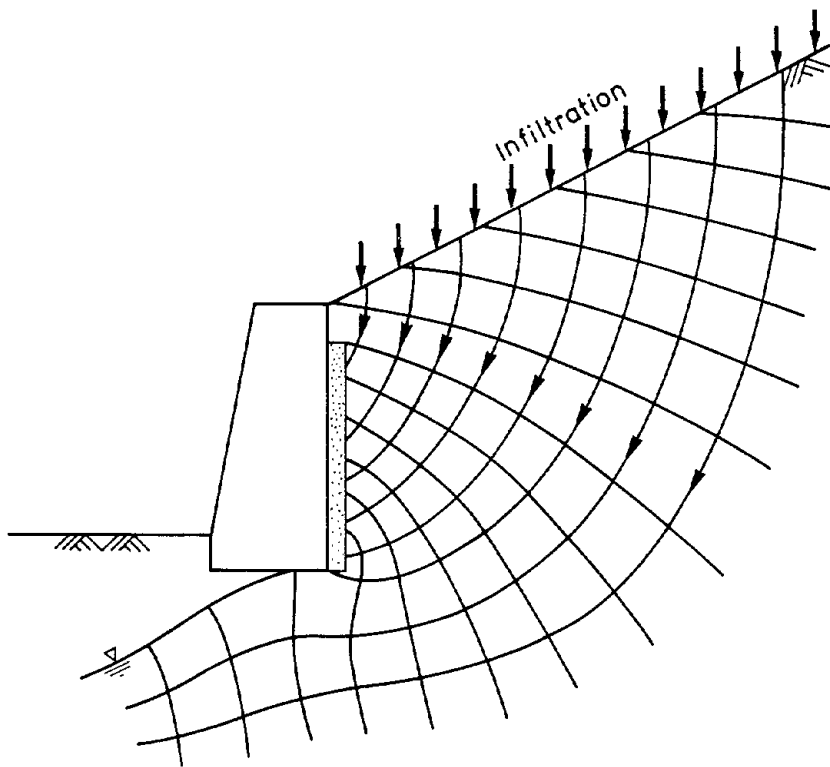
(a) Gravity Retaining Wall
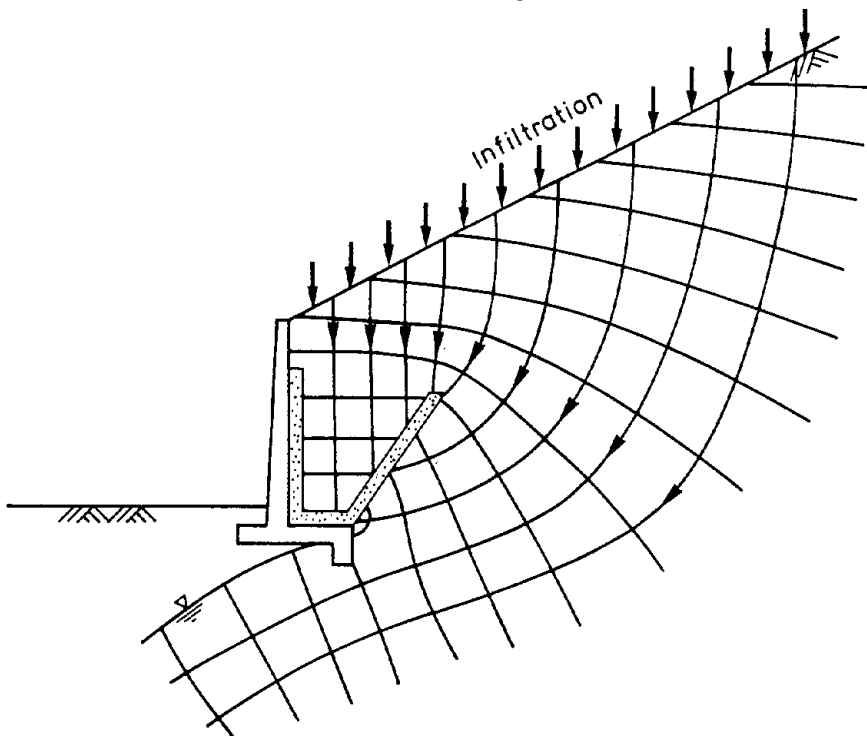
(b) Inverted T-shaped Retaining Wall with Shear Key

Note :   A homogeneous isotropic soil condition is assumed.

Figure 32 - Flow-nets Around Retaining Walls Due to Surface Infiltration

$$u_f = \frac{2(H+d-h_{w2})(d-h_{w1})}{2d+H-h_{w1}-h_{w2}} \, \gamma_w$$

$\gamma_w(d-h_{w1})$ $\gamma_w(H+d-h_{w2})$
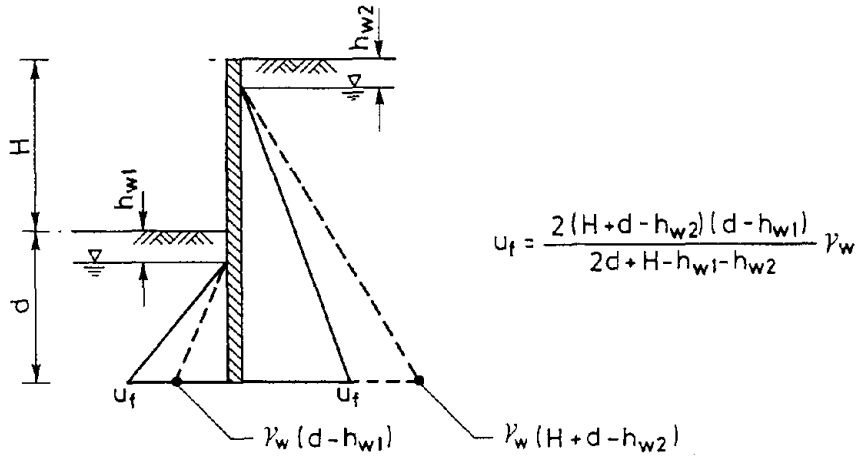
(a) Gross Water Pressure Across Wall with Seepage

$$u_c = \frac{2(H+h_{w1}-h_{w2})(d-h_{w1})}{2d+H-h_{w1}-h_{w2}} \, \gamma_w$$
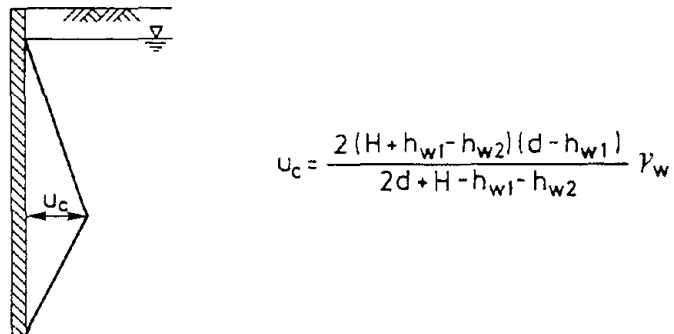
(b) Net Water Pressure Across Wall with Seepage

Notes :   (1) Figure adapted from Padfield & Mair (1984).
           (2) A homogeneous isotropic soil condition is assumed.

Figure 33 - Water Pressure Distribution Across an Impermeable Cantilevered Retaining
Wall Under Steady-state Groundwater Seepage

(a) Homogeneous and Isotropic Soil : Surface Infiltration

(b) Homogeneous and Isotropic Soil : Groundwater Seepage

(c) High Permeability over Low Permeability

(d) Anisotropy $k_{ho} > k_v$

(e) Low Permeability Lens $k_3 \ll k_1$, $k_3 \ll k_2$

(f) Discontinuous Lens of Low Permeability Material

Note :   Figure adapted from Kaiser & Hewitt (1982).
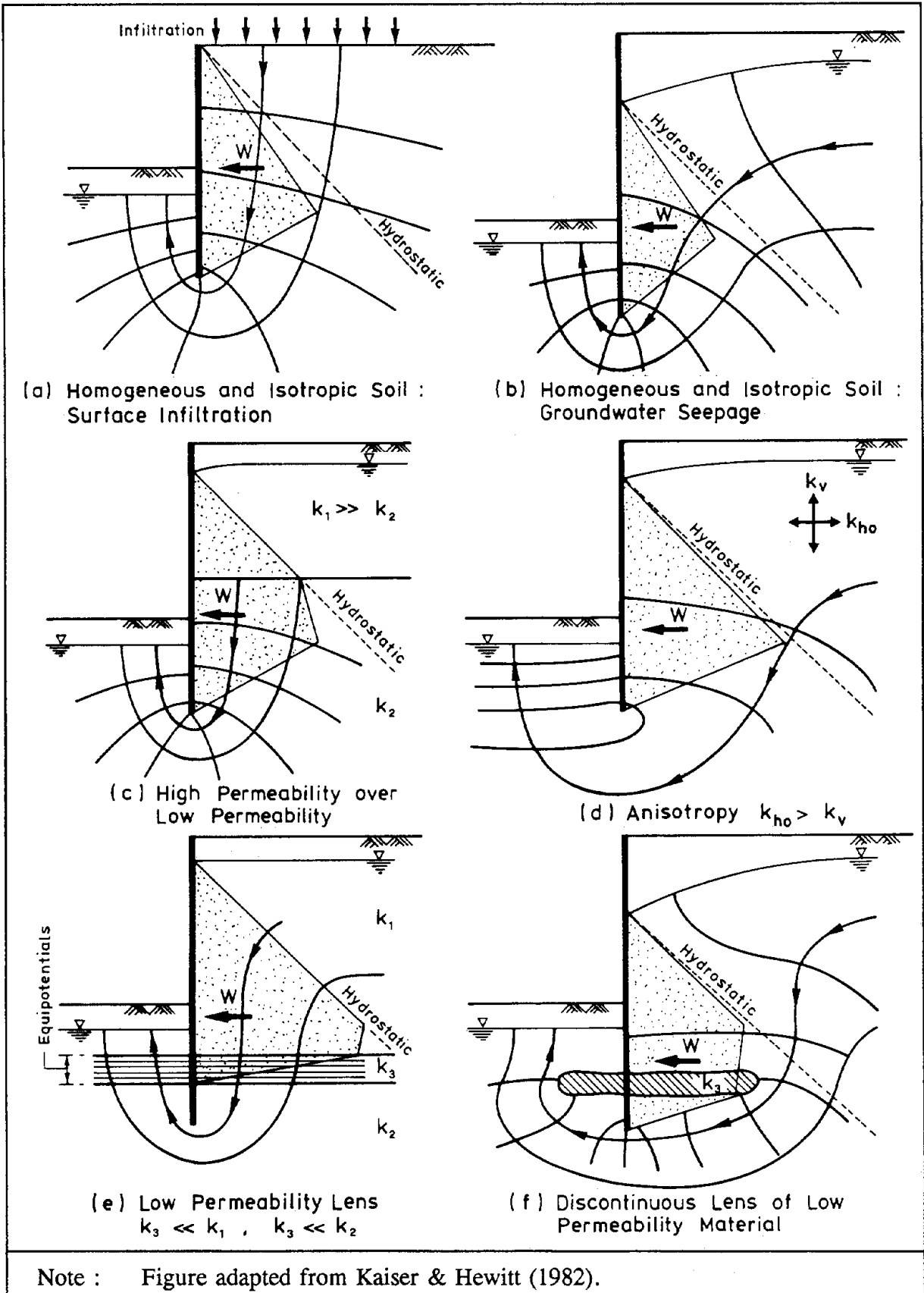
Figure 34 - Groundwater Flow Patterns and Resultant Water Pressures
Behind Impermeable Cantilevered Retaining Walls
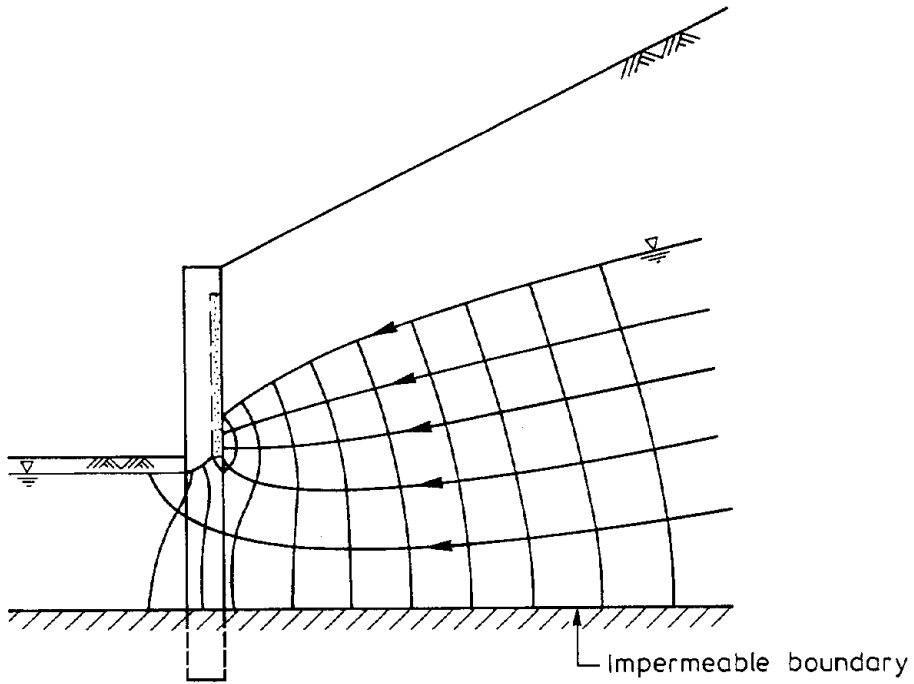
(a) Groundwater Seepage
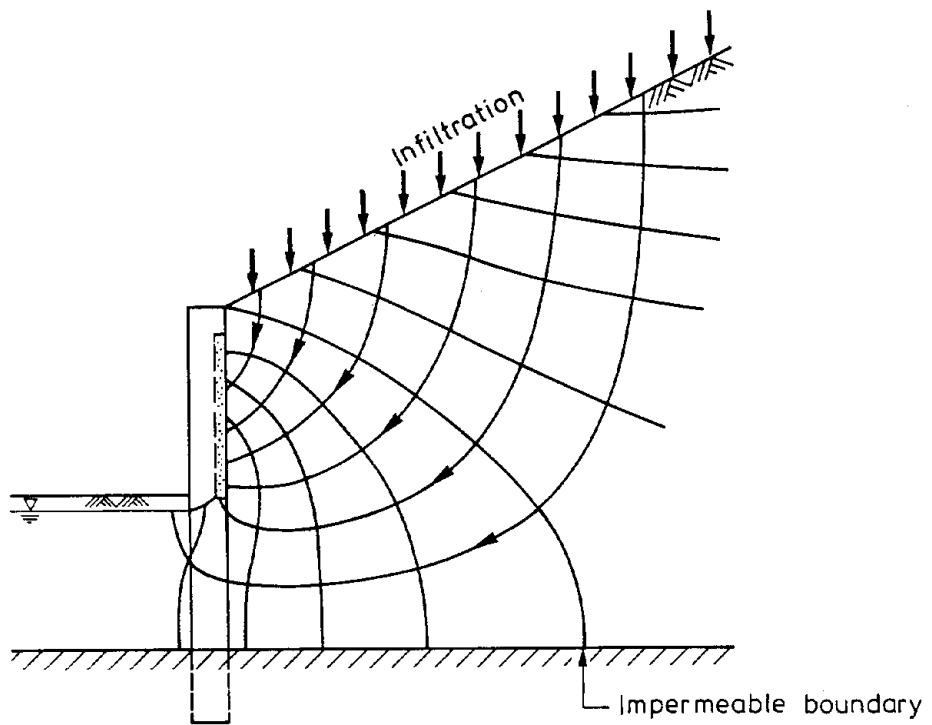
(b) Surface Infiltration

Note :   A homogeneous isotropic soil condition is assumed.

Figure 35 - Flow-nets Around Bored Pile and Caisson Walls

(a) Penetration Required for Cut-off Wall in Sands of Infinite Depth

(b) Penetration Required for Cut-off Wall in Dense Sand of Limited Depth

Notes :   (1) Figure taken from NAVFAC (1982a).

         (2) See Section 8.6 for definitions of factor of safety against heave and piping.
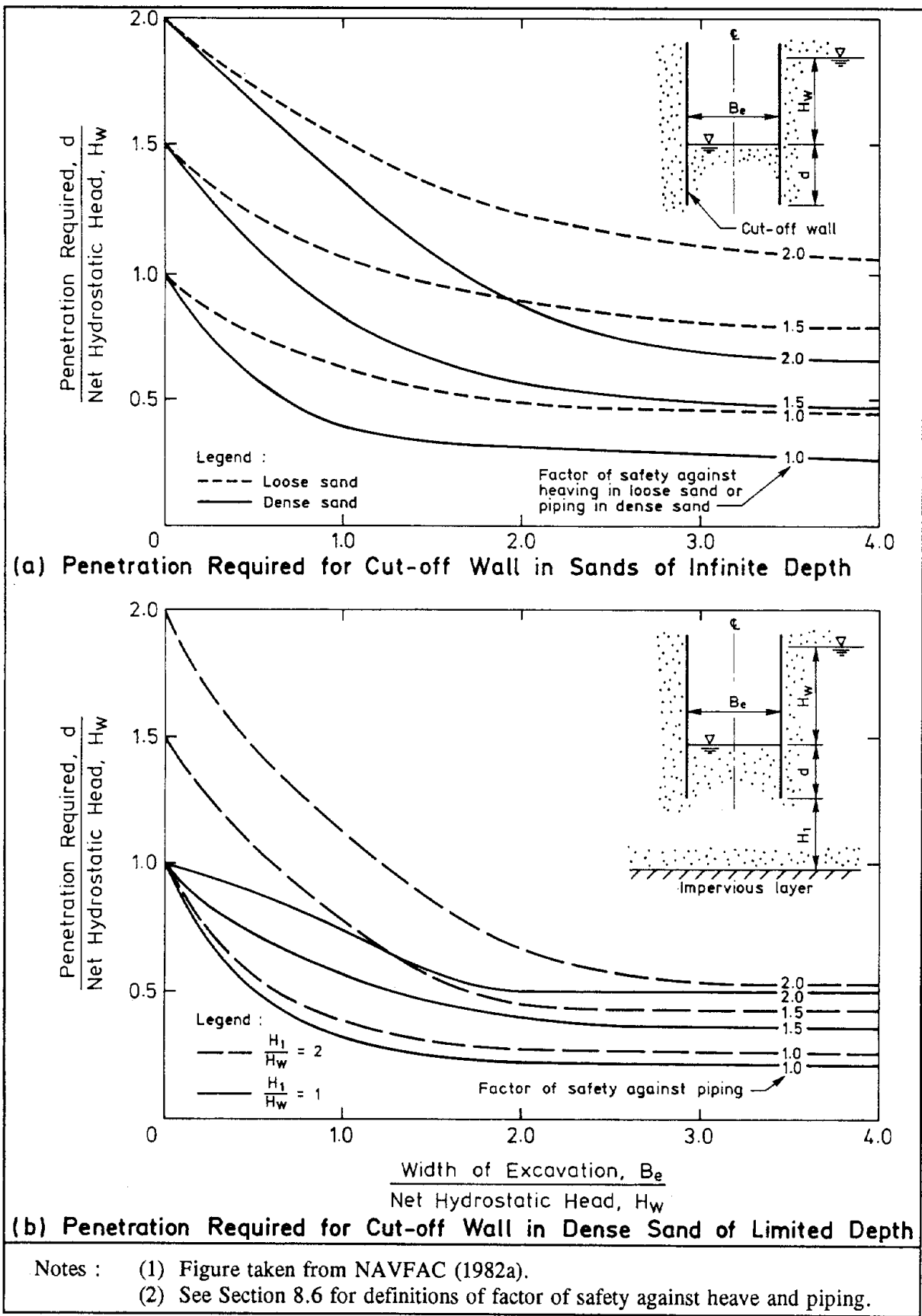
Figure 36 - Penetration of Cut-off Wall to Prevent Hydraulic Failure
in Homogeneous Sand
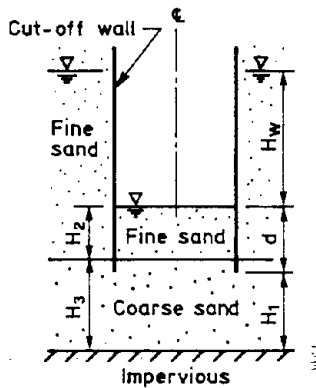
(a) COARSE SAND UNDERLYING FINE SAND

Presence of coarse layer makes flow in the fine material more nearly vertical and generally increases seepage gradients in the fine material compared to the homogeneous cross-section of Figure 36.

If top of coarse layer is below toe of cut-off wall of a depth greater than the width of excavation, safety factors of Figure 36(a) for infinite depth apply.

If top of coarse layer is below toe of cut-off wall at a depth less than the width of excavation, then uplift pressures are greater than for the homogeneous cross-sections. If permeability of coarse layer is more than ten times that of fine layer, failure head (H_w) = thickness of fine layer (H_2).
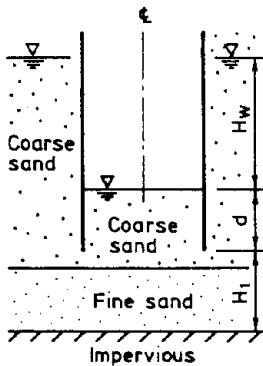
(b) FINE SAND UNDERLYING COARSE SAND

Presence of fine layer constricts flow beneath cut-off wall and generally decreases seepage gradients in the coarse layer.

If top of fine layer lies below toe of cut-off wall, safety factors are intermediate between those derived from Figure 36 for the case of an impermeable boundary at (i) the top of fine layer, and (ii) the bottom of the fine layer assuming coarse sand above the impermeable boundary throughout.

If top of fine layer lies above toe of cut-off wall, safety factors of Figure 36 are somewhat conservative for the penetration required.
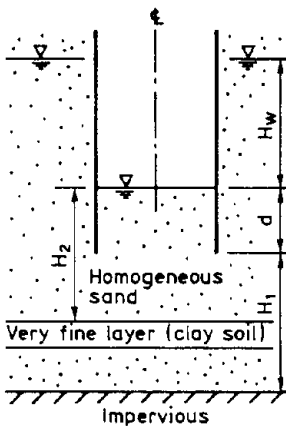
(c) VERY FINE LAYER IN HOMOGENEOUS SAND

If top of very fine layer is below toe of cut-off wall at a depth greater than the width of excavation, safety factors of Figure 36 assuming impermeable boundary at top of fine layer apply.

If top of very fine layer is below toe of cut-off wall at a depth less than the width of excavation, pressure relief is required so that unbalanced head below fine layer does not exceed height of soil above base of layer.
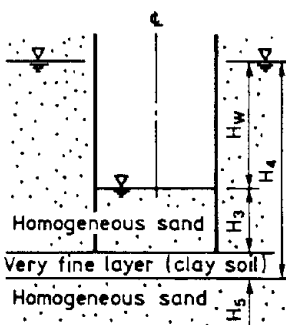
To avoid bottom heave when toe of cut-off wall is in or through the very fine layer, $(\gamma_s H_3 + \gamma_c H_5)$ should be greater than $\gamma_w H_4$, where

γ_s = saturated unit weight of the sand
γ_c = saturated unit weight of the clay
γ_w = unit weight of water

If fine layer lies above subgrade of excavation, final condition is safer than homogeneous case, but dangerous condition may arise during excavation above fine layer and pressure relief is required as in the preceding case.

| Note : | Figure adapted from NAVFAC (1982a). |

Figure 37 - Penetration of Cut-off Wall to Prevent Hydraulic Failure in Stratified Soil

(a) Loss of Overall Stability

(b) Sliding Failure

(c) Overturning Failure

(d) Bearing Capacity Failure
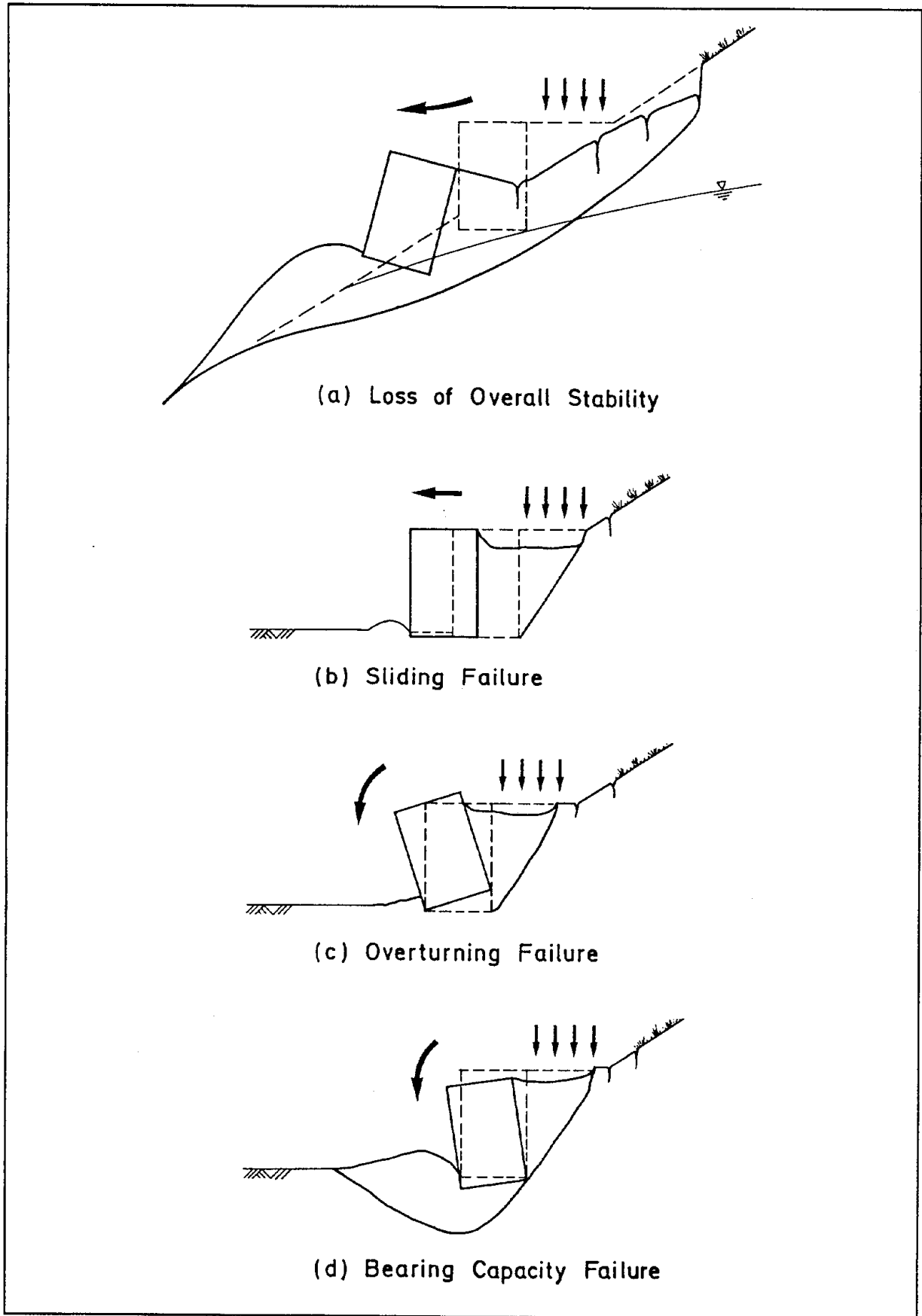
Figure 38 - Ultimate Limit States of External Instability for Gravity Retaining Walls

(a) Forces on Retaining Walls
(Water Forces Not Shown)

(b) Water Forces on Retaining
Wall

Sliding Failure

Activating force $F_a = P_{ah} + U_{1h}$

Resisting force $F_r = N_1 \tan \delta_b + R_p$, where $N_1 = W + P_{av} + U1_v - U_2$

Overturning Failure

Overturning moment $M_o = P_{ah}y_p - P_{av}x_p + U_{1h}y_{u1} - U_{1v}x_{u1} + U_2x_{u2}$

Resisting moment $M_r = Wx_w + R_py_r$

Eccentricity $e = B / 2 - (M_r - M_o) / N_1$

Bearing Capacity Failure

Effective normal load Q_n and shear load Q_s imposed on the foundation are given by $Q_n = N_1$ and $Q_s = F_a$ respectively. Guidance on design against bearing capacity failure of the foundation is given in Appendix A.

Notes :	(1) The total weight W equals the weight of the wall plus the weight of the hatched portion of soil.
	(2) The possibility of excavation in front of the wall should be considered in evaluating passive resistance. Where excavation is likely, a minimum trench depth of 1 m should be allowed for in the calculation.
	(3) Zero wall friction should be assumed for the vertical plane in soil on which the passive resistance acts.
	(4) Reference should be made to Chapter 8 for guidance on evaluation of water forces.
	(5) Checking of overturning failure is not required where the middle-third rule is complied with (see Section 9.2.4).

Figure 39 - Calculation Models for Checking Against Sliding, Overturning
and Bearing Capacity Failure of Gravity Retaining Walls

(a) Active Earth Pressure for Part of the Retaining Wall above X - X

(b) Active Earth Pressure for the Whole Retaining Wall

Notes : (1) Reference should be made to Figure 39 for calculation models for checking against sliding, overturning and bearing capacity failure.
(2) Where appropriate, the alternative sliding failure mechanisms shown in Figure 41 should be checked.

Figure 40 - Calculation of Active Earth Pressure for Stepped Gravity Retaining Walls

(a) Mechanism 1 : Sliding Along
    Soil/Structure Interface AB

(b) Mechanism 2 : Sliding Along
    a Foundation Soil Surface CB

For mechanism 1, resolve forces parallel and perpendicular to the base AB of the retaining wall to obtain S_1 and N_1 respectively. Passive resistance R_{p1} in front of the wall should be calculated down to point A.

Activating force $F_a = S_1$
Resisting force $F_r = N_1 \tan \delta_b + R_{p1} \cos \omega$

For mechanism 2, resolve forces parallel and perpendicular to selected foundation soil surface CB to obtain S_2 and N_2 respectively. Passive resistance R_{p2} in front of the wall should be calculated down to point C.

Activating force $F_a = S_2$
Resisting force $F_r = N_2 \tan \phi' + c'l + R_{p2}$

The value of Ω should be varied to obtain the worst design condition.

Notes :   Notes (1) to (5) of Figure 39 are also applicable.

Figure 41 - Alternative Sliding Failure Mechanisms for Retaining Walls with a Tilted Base
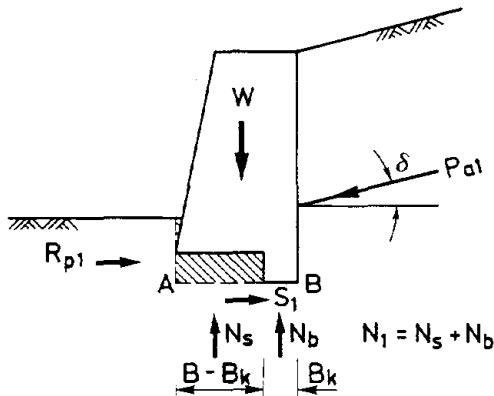
(a) Mechanism 1 : Sliding Along Horizontal Plane AB
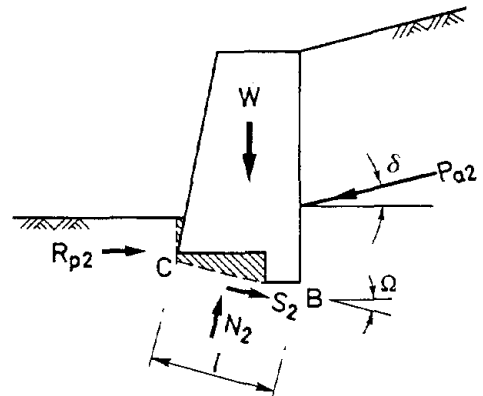
(b) Mechanism 2 : Sliding Along a Foundation Soil Surface CB

For mechanism 1, resolve forces parallel and perpendicular to horizontal plane AB to obtain S_1 and N_1 respectively. Calculate N_s and N_b assuming a trapezoidal distribution of ground bearing pressures. Passive resistance R_{p1} in front of the wall should be calculated down to point A.

Activating force $F_a = S_1$

Resisting force $F_r = N_s \tan \phi' + c'(B - B_k) + N_b \tan \delta_b + R_{p1}$

For mechanism 2, resolve forces parallel and perpendicular to selected foundation soil surface CB to obtain S_2 and N_2 respectively. Active force P_{a2} and passive resistance R_{p2} should be calculated down to points B and C respectively.

Activating force $F_a = S_2$

Resisting force $F_r = N_2 \tan \phi' + c'l + R_{p2} \cos \theta$

The value of Ω should be varied to obtain the worst design condition.

Notes : (1) Notes (1) to (5) of Figure 39 are also applicable.

(2) Similar principles may be applied for retaining walls with a shear key located between the toe and the heel.

Figure 42 - Alternative Sliding Failure Mechanisms for Retaining Walls with a Shear Key

Notes : (1) For the preferred drainage scheme A, the extent of the inclined drain is dependent on the design groundwater level behind the retaining wall. To intercept infiltration, the inclined drain should be installed to a level of at least two-thirds of the height of the wall.

(2) The filter/drainage layers may be omitted if a free-draining granular backfill is used. However, a drainage pipe should be provided to discharge water safely.

(3) The vertical and horizontal filter/drainage layers may be replaced by suitable prefabricated drainage composites.

(4) For a retaining wall with level backfill, the top 1.5 m layer of the fill should be a suitable material of relatively low permeability. For sloping backfill, the same provision should be made for a vertical thickness of at least 3 m (see Section 3.7).

Figure 43 - Typical Drainage Schemes for Gravity Retaining Walls
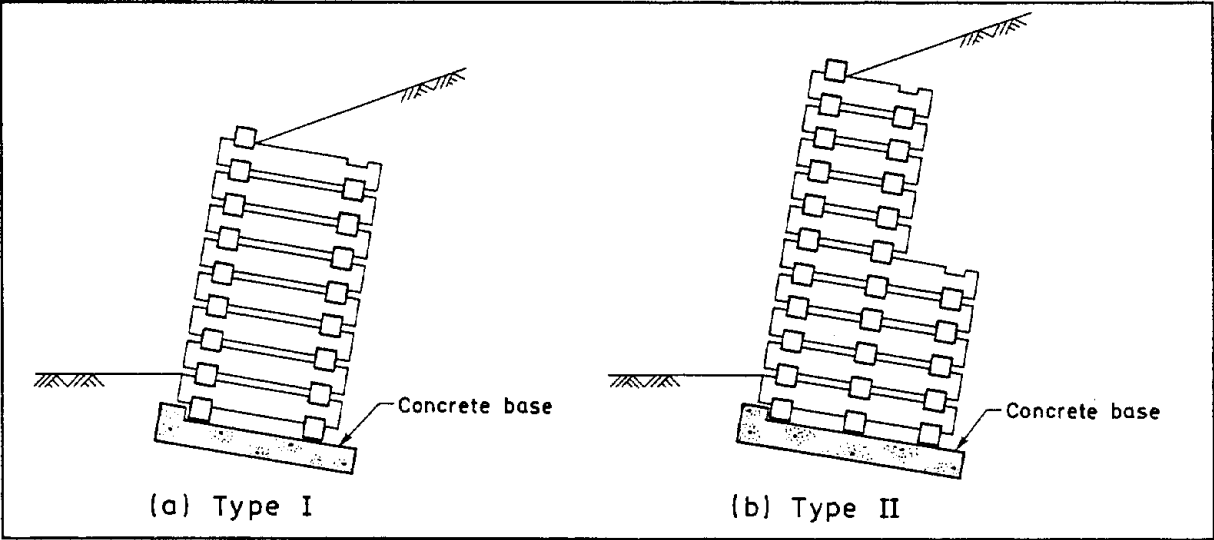
Figure 44 - Cross-sections of Typical Crib Walls

Note :   Type I is an open-faced walling system. Types II and III are both
closed-faced walling systems.
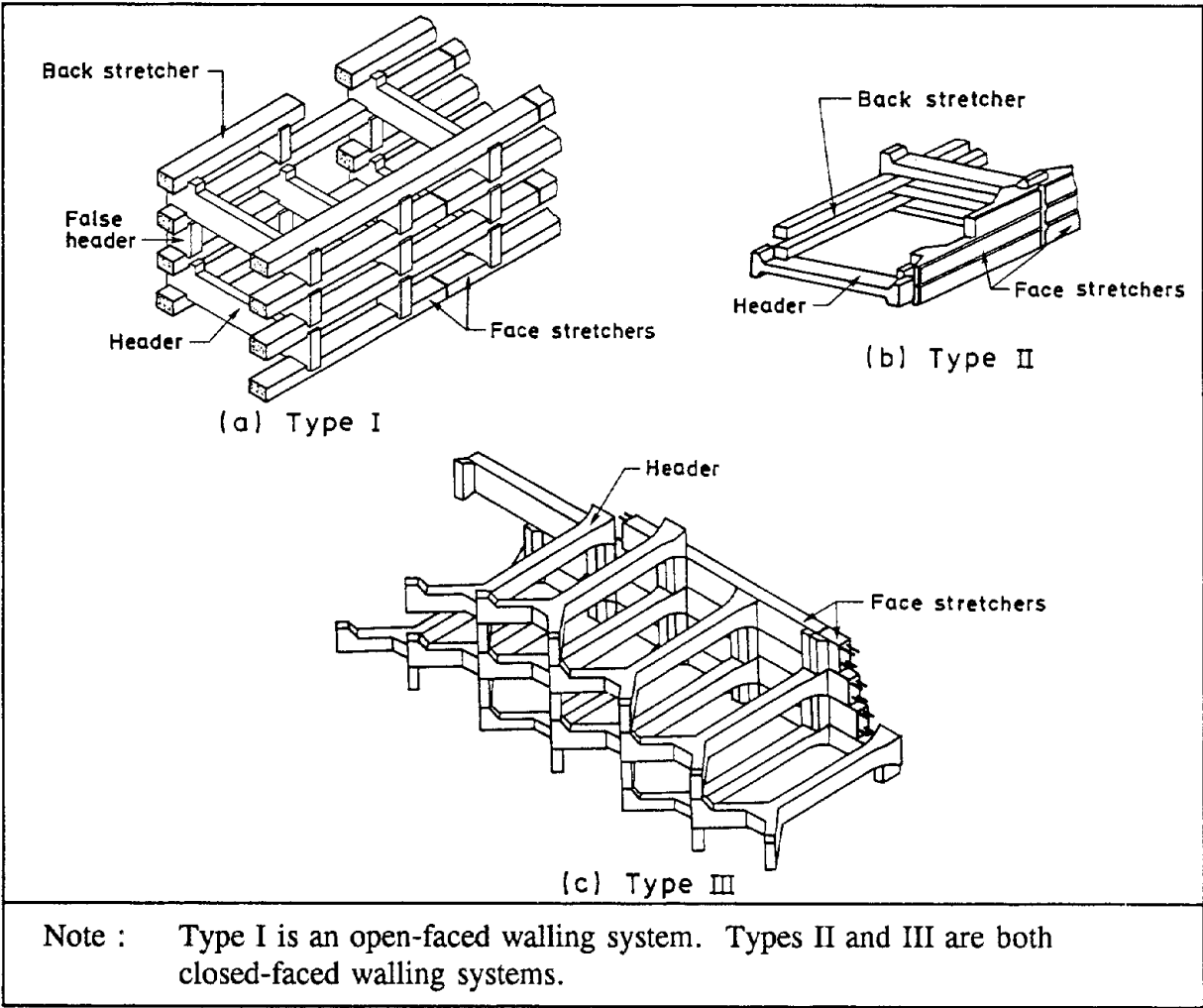
Figure 45 - Details of Typical Reinforced Concrete Crib Wall Systems

(a) Type I   (b) Type II
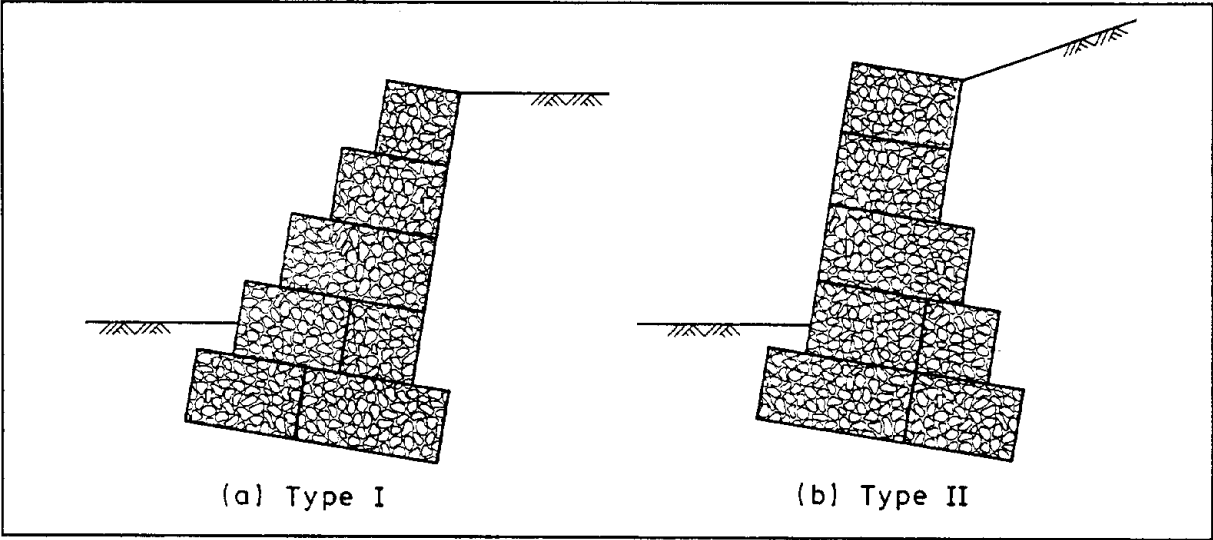
Figure 46 - Cross-sections of Typical Gabion Walls

Lid

Selvedge

Back

End   Diaphragm   Base   End

Front

(a) Before Assembly (PLAN)

Detail 'X'

(b) After Assembly (ISOMETRIC VIEW)
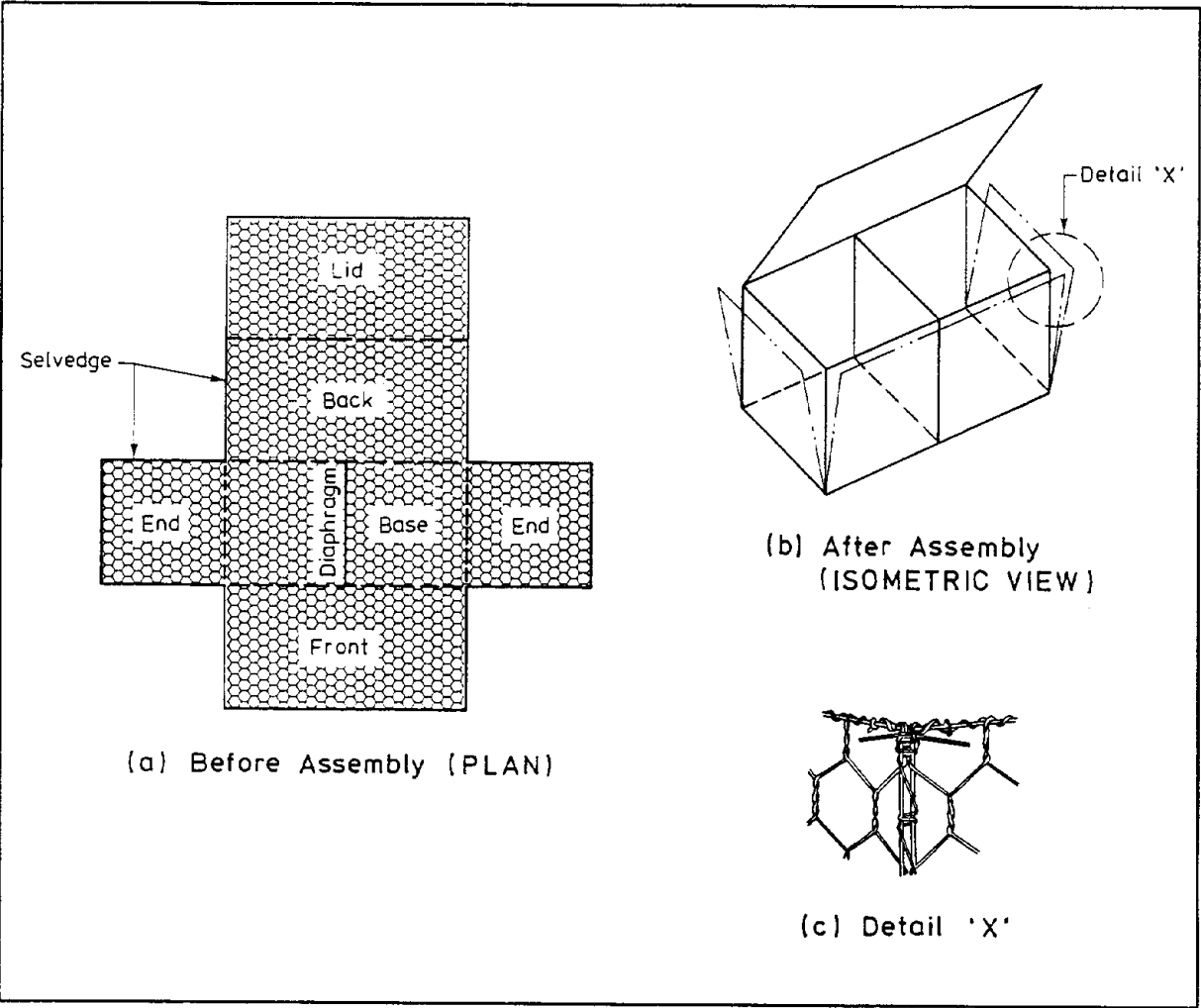
(c) Detail 'X'

Figure 47 - Details of Hexagonal Woven-mesh Gabions

(a) L-shaped Cantilever Retaining Wall

(b) Inverted T-shaped Cantilever Retaining Wall

(c) Reversed L-shaped Cantilever Retaining Wall with Key

(d) Inverted T-shaped Cantilever Retaining Wall with Key

(e) Retaining Wall with Counterforts

(f) Retaining Wall with Buttresses
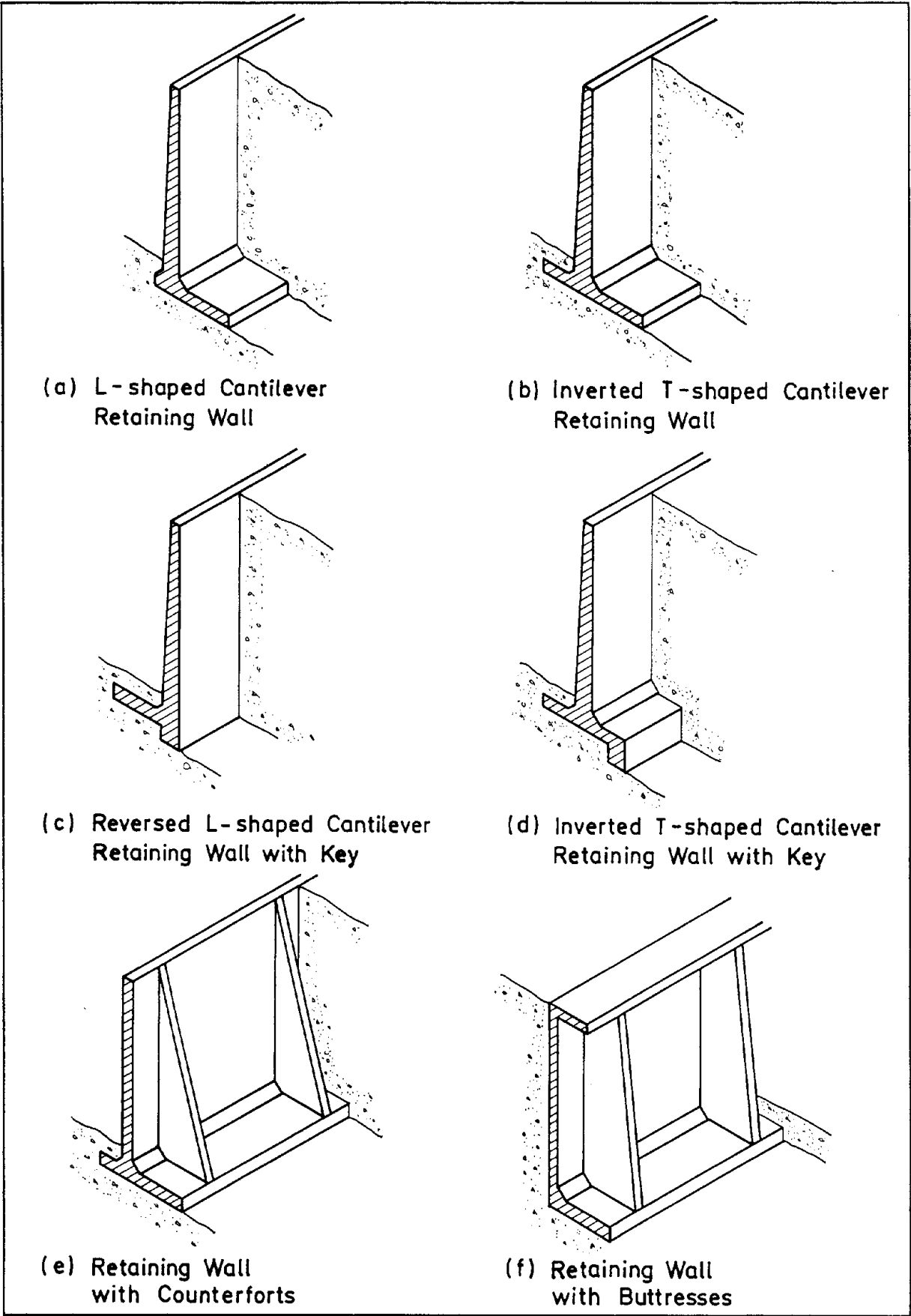
Figure 48 - Types of Reinforced Concrete Retaining Walls

(a) Preferred Drainage Scheme A

(b) Drainage Scheme B

Notes : (1) For the preferred drainage scheme A, the extent of the inclined drain is dependent on the design groundwater level behind the retaining wall. To intercept infiltration, the inclined drain should be installed to a level of at least two-thirds of the height of the wall.

(2) The filter/drainage layers may be omitted if a free-draining granular backfill is used. However, a drainage pipe should be provided to discharge water safely.

(3) Notes (3) and (4) of Figure 43 are also applicable.

Figure 49 - Typical Drainage Schemes for Reinforced Concrete Retaining Walls

(a) Deflected Shape

(b) Idealised Pressure Distribution

(c) Simplified Pressure Distribution

Notes : (1) Figure taken from Padfield & Mair (1984).

(2) p_a and p_p are used here to denote active and full passive earth pressures, the actual distribution of which should be determined in accordance with the guidance given in Chapter 6 of this Geoguide.

(3) The possibility of excavation in front of the wall should be allowed for in evaluating passive resistance.

Figure 50 - Pressure Distribution for a Cantilevered Retaining Wall Embedded Entirely in Soil at Limiting Equilibrium Condition

(a) Idealised Pressure Distribution

(b) Shear Force Diagram

(c) Bending Moment Diagram

(d) Forces Acting in Rock Socket

(e) Design Equations

$$d_r = \frac{V}{q_u} \left(\sqrt{2 + \frac{4 q_u M}{V^2}} + 1 \right)$$

$$d_1 = \frac{1}{2} \left(d_r + \frac{V}{q_u} \right)$$

Legend :

F_1, F_2	Reactions in top and bottom portions of rock socket respectively
M, V	Bending moment and shear force respectively at the top of rock socket
q_u	Ultimate lateral bearing capacity of rock

Notes :

(1) For cantilevered sheet retaining walls, M, V, F_1 and F_2 should be calculated per unit width of the wall.

(2) For a wall composed of vertical members of width D at a spacing S, the values of M and V to be used in the above equations should be calculated based on active and passive earth pressures acting over a width equal to S. An exception to this is where S > 3D, in which case the contribution from the active and passive earth pressures below excavation level should be based on an effective width of 3D only. F_1 and F_2 should always be based on a width equal to D, i.e. q_u should be multiplied by D.

(3) The possibility of excavation in front of the wall should be allowed for in evaluating passive resistance.

Figure 51 - Design of Rock Socket Against Bearing Failure for the Case Where the Top of the Socket is Above the Point of Zero Shear

(a) Idealised Pressure Distribution

(b) Shear Force Diagram

(c) Bending Moment Diagram

(d) Forces Acting in Rock Socket

(e) Design Equations

$$d_r = \frac{V}{q_u} \left(\sqrt{2 + \frac{4 q_u M}{V^2}} - 1 \right)$$

$$d_1 = \frac{1}{2} \left(d_r - \frac{V}{q_u} \right)$$

Legend :

F_1, F_2	Reactions in top and bottom portions of rock socket respectively
M, V	Bending moment and shear force respectively at the top of rock socket
q_u	Ultimate lateral bearing capacity of rock

Notes : (1) Notes (1) to (3) of Figure 51 are also applicable.

(2) Where the depth of soil cover in the passive zone is large, the calculated d_1 value can be negative. For such cases, reference should be made to Figure 53.
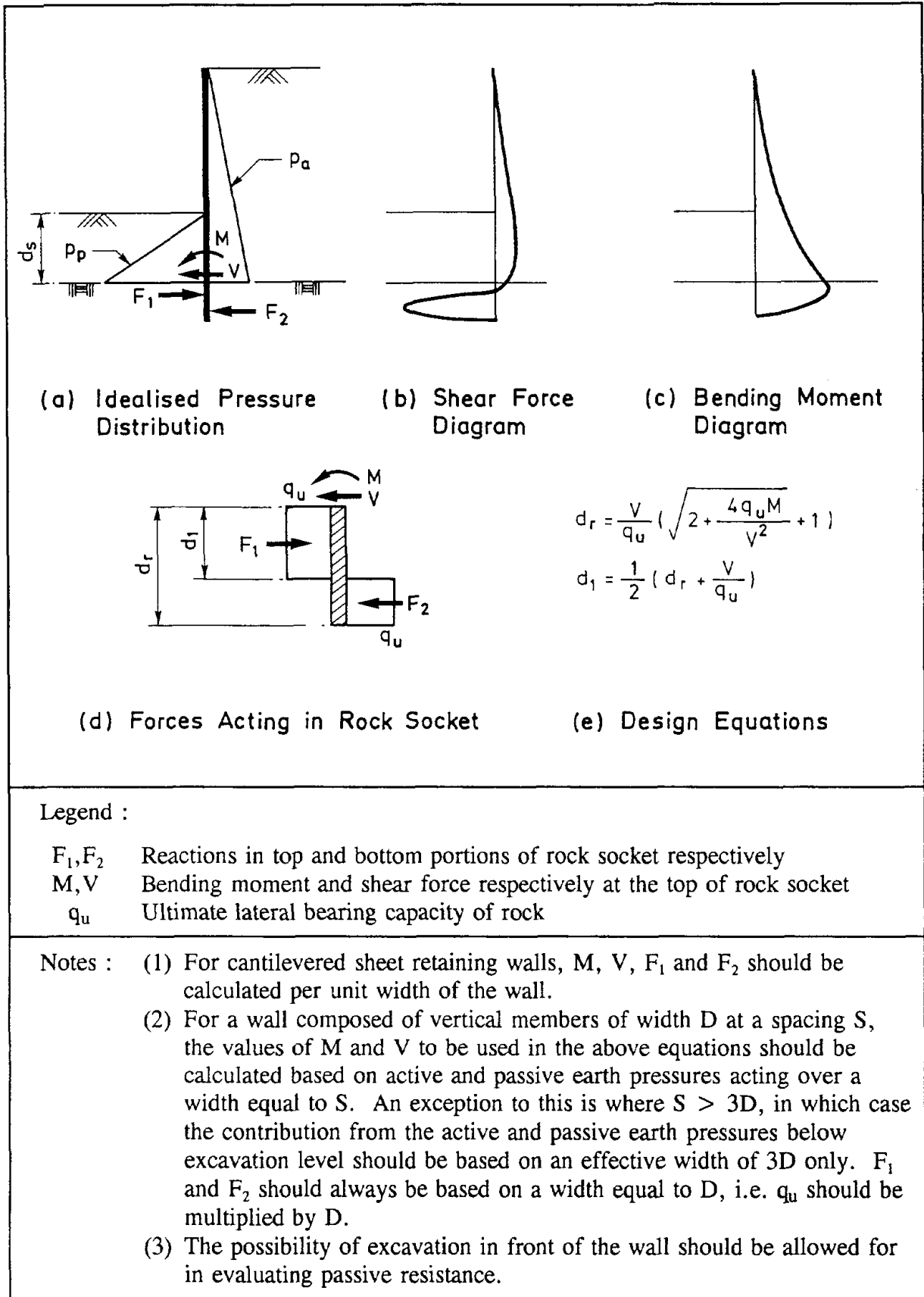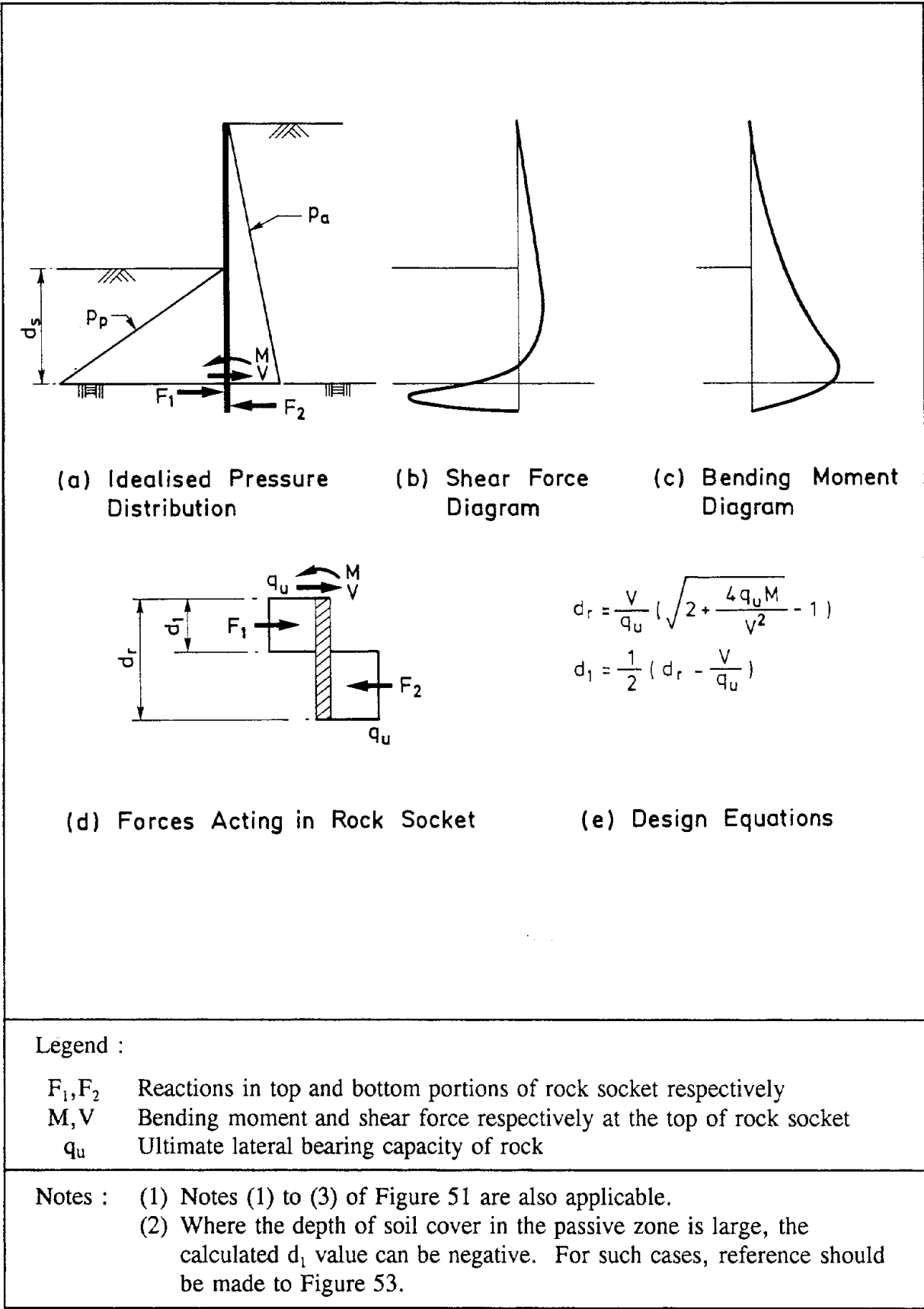
Figure 52 - Design of Rock Socket Against Bearing Failure for the Case Where the Top of the Socket is Below the Point of Zero Shear

(a) Idealised Pressure Distribution

(b) Shear Force Diagram

(c) Bending Moment Diagram

(d) Forces Acting in Rock Socket

(e) Design Equation

$$d_r = \frac{V}{q_u}$$

Legend :

F_2	Reaction in the rock socket
M,V	Bending moment and shear force respectively at the top of rock rocket
q_u	Ultimate lateral bearing capacity of rock

Notes :

(1) For cantilevered sheet retaining walls, M, V, and F_2 should be calculated per unit width of the wall.

(2) For a wall composed of vertical members of width D at a spacing S, the values of M and V to be used in the above equations should be calculated based on active and passive earth pressures acting over a width equal to S. An exception to this is where S > 3D, in which case the contribution from the active and passive earth pressures below excavation level should be based on an effective width of 3D only. F_2 should always be based on a width equal to D, i.e. q_u should be multiplied by D.

(3) The possibility of excavation in front of the wall should be allowed for in evaluating passive resistance.
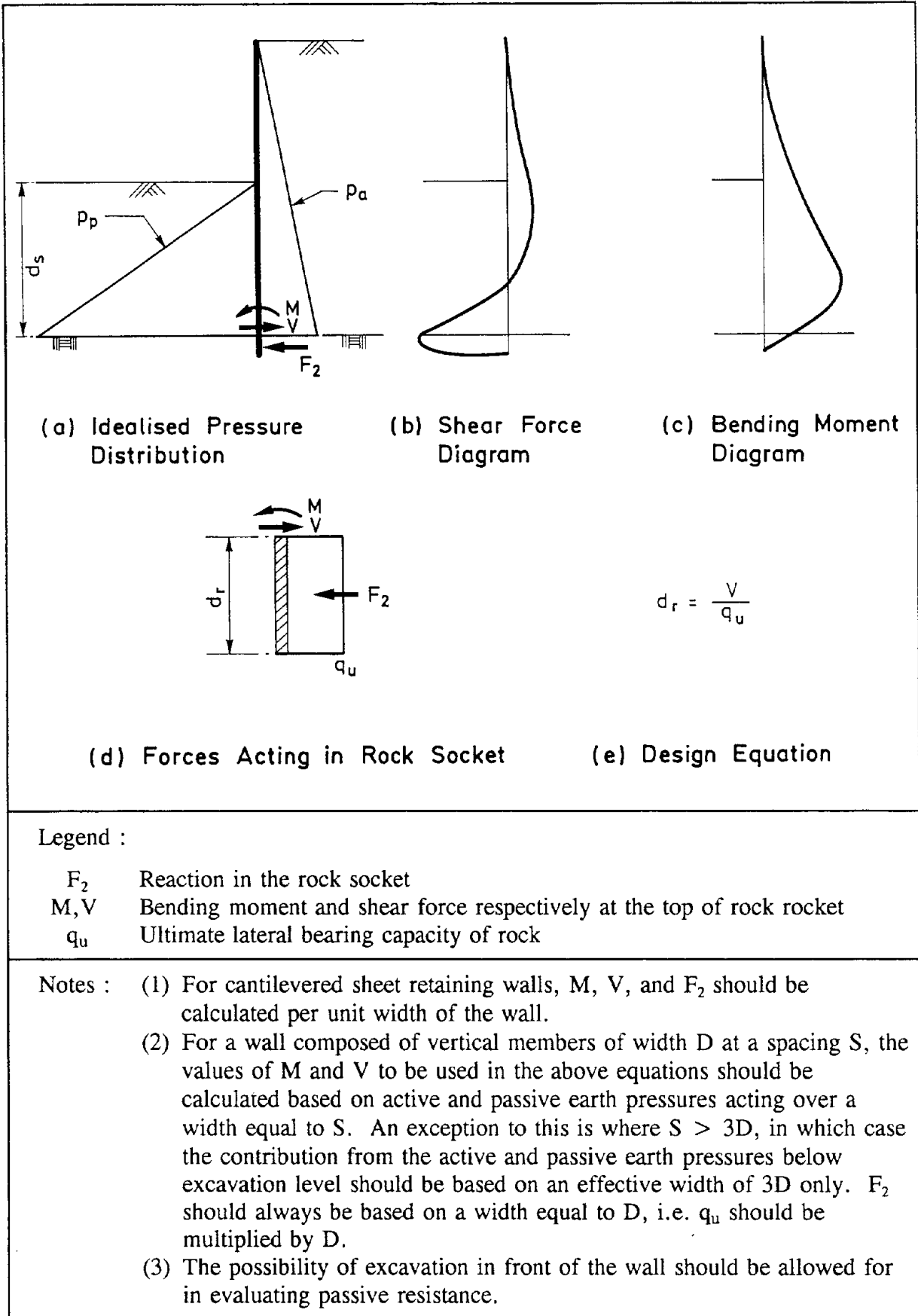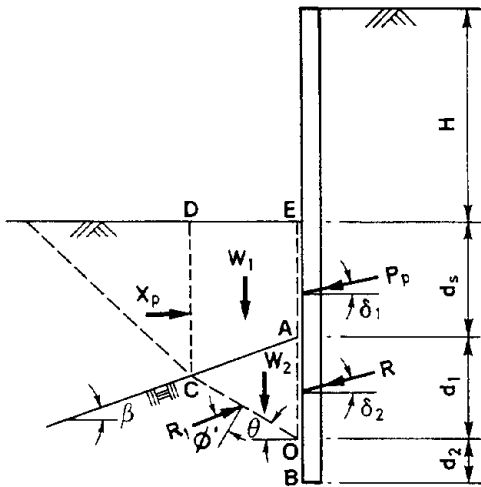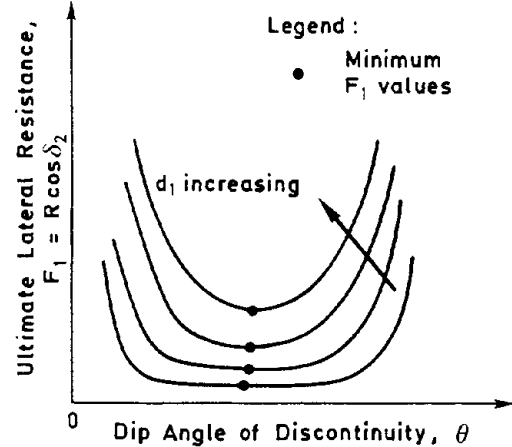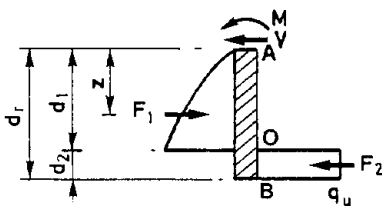
Figure 53 - Design of Rock Socket Against Bearing Failure for the Case Where $d_1 < 0$

(a) Model for Calculating Rock Socket Capacity Against Planar Discontinuity-controlled Failure

(b) Variation of Rock Socket Capacity with Dip Angle of Discontinuity

(c) Forces Acting in Rock Socket

(d) Design Equations

$$d_1 = \frac{M + F_1 z}{F_1 - V} - \frac{F_1 - V}{2q_u} \quad \text{(full equilibrium)}$$

$$d_1 = \frac{M}{F_1 - V - F_1 (z / d_1)} \quad \text{(simplified)}$$

$$d_r = d_1 + \frac{F_1 - V}{q_u}$$
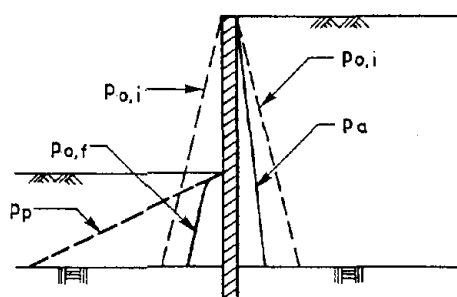
Legend :

F_1, F_2	Reactions at the top and bottom portions of the rock socket respectively
M, V	Bending moment and shear force respectively at the top of rock socket
P_p, X_p	Passive forces acting on the vertical boundaries AE and CD
R, R_1	Reactions acting on the presumed rock wedge
W_1, W_2	Weight of soil in ACDE and weight of rock in AOC respectively
δ_1, δ_2	Mobilised angles of wall friction in soil and in rock respectively
ϕ'	Angle of shearing resistance of the presumed discontinuity OC

Notes : (1) If point A is below the point of zero shear, then the direction of V reverses.

(2) For cantilevered sheet retaining walls, M, V, F_1 and F_2 and R should be calculated per unit width of the wall.

(3) For a wall composed of vertical members of width D at a spacing S, the values of M and V to be used in the above equations should be calculated based on active and passive earth pressures acting over a width equal to S. An exception to this is where S > 3D, in which case the contribution from the earth pressures below excavation level should be based on an effective width of 3D only. The values of R, and hence F_1, may be based on a width equal to S. However, F_2 should always be based on a width equal to D only, i.e. q_u should be multiplied by D.

(4) The possibility of excavation in front of the wall should be allowed for in evaluating passive resistance.

(5) Where the depth of soil cover in the passive zone is large, the calculated d_1 value can be negative. For such cases, reference should be made to Figure 53.
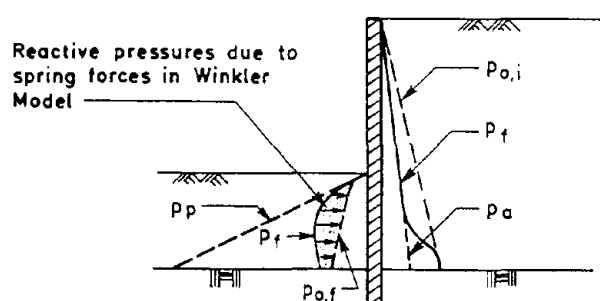
Figure 54 - Design of Rock Socket Against Planar Discontinuity-controlled Failure
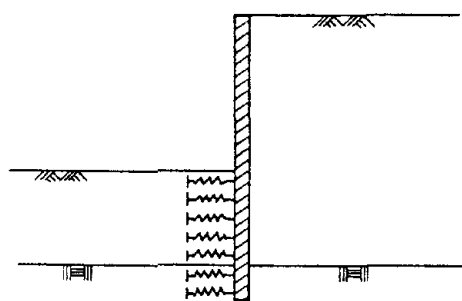
(a) Winkler Model

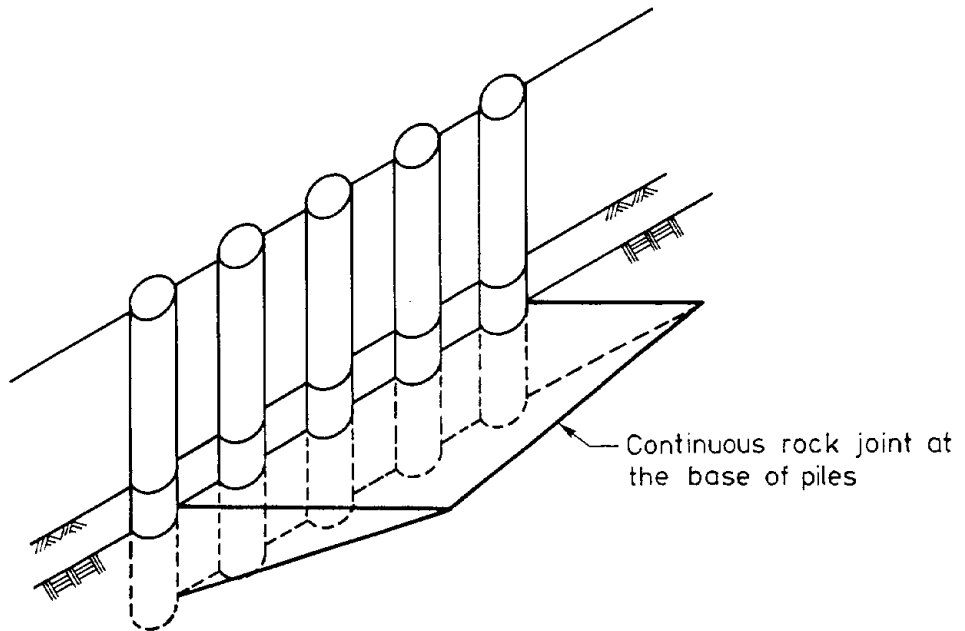(b) Initial Pressure Distribution to be Used in Analysis

(c) Final Pressure Distribution Obtained from Analysis

Legend :

p_a	Active earth pressures
p_f	Final earth pressures
$p_{o,i}$	'At-rest' pressures prior to bulk excavation
$p_{o,f}$	Locked-in horizontal stresses after bulk excavation but prior to any wall movement
p_p	Passive earth pressures

Note :   Reference should be made to Sections 6.9, 11.2.4 and 11.2.5 for further guidance on the analytical procedure. Notes (2) and (3) of Figure 50 are also relevant.

Figure 55 - Winkler Model and Pressure Distribution for Soil-structure Interaction Analysis of Cantilevered Retaining Walls

(a) Planar Failure Along Discontinuities

Continuous rock joint at the base of piles
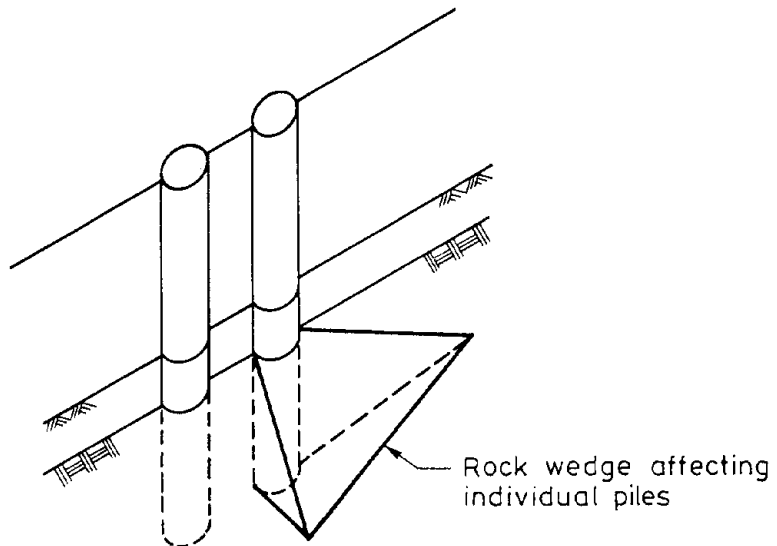
(b) Wedge Failure Along Discontinuities

Rock wedge affecting individual piles

Note :   Adapted from Greenway et al (1986b).

Figure 56 - Discontinuity-controlled Failure Modes for Rock Sockets in Bored Pile and Caisson Walls
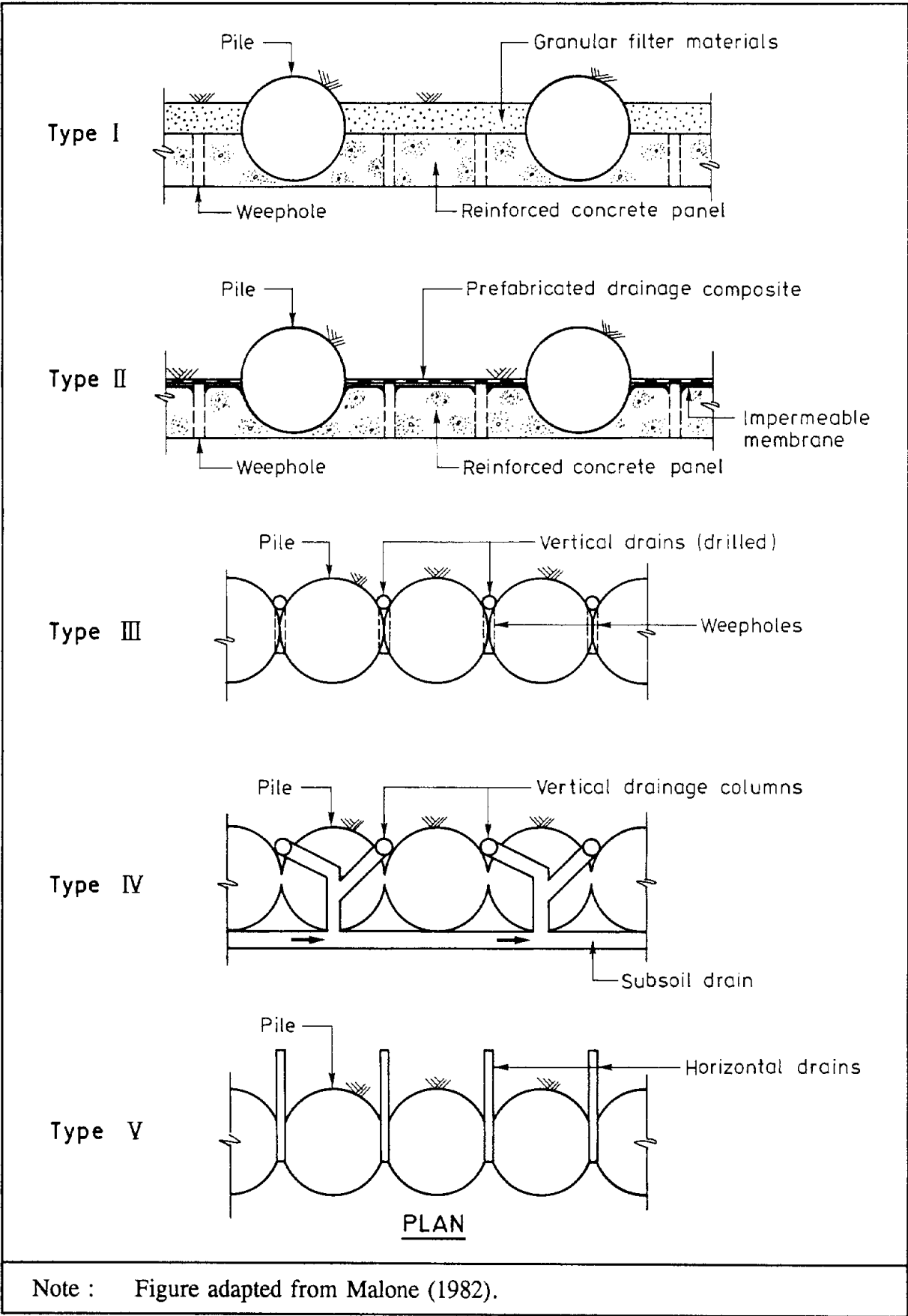
Note : Figure adapted from Malone (1982).

Figure 57 - Alternative Drainage Arrangements for Bored Pile and Caisson Walls

RETAINING WALL BASIC RECORD	
Location of Retaining Wall : Retaining Wall Reference No. :	Compiled by : Signature : Date :
Risk to Life : High / Low / Negligible	Economic Risk : High / Low / Negligible
Designed by :	Project File Reference :
Constructed by :	Drawing Nos :
Client :	Maintenance Office :
Date of Submission :	Approval Date :
Construction Commenced on :	Completion Date :
Ground Investigation Reports (nos. and location) : Laboratory Testing Reports (nos. and location) : Geotechnical Reports (nos. and location) :	
DETAILS OF RETAINING WALL	LOCATION PLAN WITH GRID
Type : (e.g. mass concrete, R.C., crib, gabion, reinforced fill or caisson wall) Ground level in front : _____ mPD Length : _____ m Retained height : _____ m Embedment below ground : _____ m Width of wall / diameter of caissons : _____ m Critical design assumptions : Surcharge : ___ kPa Groundwater level : ___ mPD Type of backfill or retained earth : Details of foundation materials :	
RELEVANT STRUCTURAL DETAILS	TYPICAL SECTION OF WALL
(e.g. grade of concrete, type of crib units or gabion baskets, type and spacing of reinforcing elements and type of facing units for reinforced fill, spacing of caissons and length of sockets in rock, etc.)	(Section should show salient features of surrounding areas)
Surface and Subsurface Drainage Details : (e.g. filter and drainage materials and dimensions, surface channels, horizontal drain type and spacing, etc.)	Service Conduits / Existing Foundations _____ m for crest / _____ m for toe
Details of Instrumentation and Post Construction Monitoring Required : (e.g. groundwater levels, movements, drain flows, loads, etc.)	Maintenance Required : Routine / others

Figure 58 - Example of a Retaining Wall Basic Record Sheet

APPENDIX A

DESIGN OF RETAINING WALL
SPREAD FOUNDATIONS

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CONTENTS

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A.1   GENERAL

This Appendix gives general guidance on the design of spread foundations for retaining walls.   The need to establish the nature and characteristics of the ground for foundation design cannot be overstated.   Guidance on the planning, execution and supervision of ground investigations is given in Geoguide 2 : Guide to Site Investigation (GCO, 1987).

The nature of the foundation material should be taken into account in selecting the form of retaining walls to be used.   Reinforced fill structures and gabion walls can tolerate more ground movement than 'rigid' structures such as reinforced concrete retaining walls and crib walls.   The use of such structures should be considered where the ground is composed of compressible materials.

In some cases, it may be necessary to replace weak foundation materials by compacted fill or to pre-treat the ground, e.g. by preloading the foundation and by drainage.   Reference should be made to specialist literature for guidance on ground treatment methods.

A.2   DEPTH OF FOUNDATION

The following factors should be considered in choosing the depth of the base of a retaining wall :

(a) the need to reach an adequate bearing material,

(b) possible ground movements and reduction of bearing capacity due to seepage or climatic effects, or due to construction procedure,

(c) the groundwater level and problems which may occur if excavation for the foundation is required below this level,

(d) the possibility of erosion, scour or future excavation in front of the wall which may undermine the foundation, and

(e) for clayey soils, the depth above which shrinkage and swelling may cause appreciable movements.

It is recommended that for retaining walls founded on soil, the founding level should be at least 0.5 m below the ground level in front of the retaining wall.

A.3   DESIGN OF FOUNDATIONS IN SOIL

A.3.1   Ultimate Limit State Design

A.3.1.1   Performance Criteria

The design should show that the foundation will support the loading due to the retaining wall with adequate safety against bearing capacity failure.   The following condition

should be satisfied for all ultimate limit state load cases and load combinations :

$$Q_n \leq Q_{ult} \qquad \ldots \ldots \ldots \ldots \ldots \ldots \ldots \text{(A1)}$$

where Q_n = ultimate load acting normal to the foundation base (Figure A1)
 Q_{ult} = ultimate resistance against bearing capacity failure of the foundation

Both Q_n and Q_{ult} should be calculated using factored loadings and material parameters. The partial safety factor approach given in Chapter 4 of this Geoguide should be followed.

Long-term design situations should be assessed using an effective stress analysis. Short-term design situations should also be considered, particularly for fine-grained soils with a low permeability where the load-induced pore water pressures under the 'undrained' condition may result in reduced shear strengths, and hence a lower foundation bearing capacity.

A.3.1.2 Calculation of Ultimate Bearing Capacity

General equations for calculating the ultimate bearing capacity of a c' - ϕ' soil foundation and of a ϕ = 0 (or s_u) soil foundation are given in Figures A2 and A3 respectively. It should be noted that these equations are only applicable to shallow foundations, i.e. those with a depth of embedment D less than or equal to the foundation width B. The beneficial effects of foundation embedment is unreliable for shallow foundations. Therefore, no 'depth factor' is given and the weight of soil above the level of the foundation base should be treated as a surcharge (with zero shear strength) in the calculations.

Retaining wall foundations are usually loaded eccentrically. For an eccentrically loaded foundation, the calculation of ultimate bearing capacity should be based on the effective area A' of the foundation base. A' is the reduced area of the base whose centroid is the point through which the resultant load acts (Figure A1). For zero eccentricity, A' equals the full area of the base. Where the value of e_b is negative, i.e. the resultant load acts at a distance greater than B/2 from the toe of the foundation, the calculation models given above are conservative. For such cases, e_b may be taken as zero for design purposes.

In using the general bearing capacity equations, account should be taken of the shape and base tilt of the foundation, the effect of inclination of the applied resultant load and the ground slope in front of the foundation.

The bearing capacity equations are applicable to soils which can be treated as a homogeneous isotropic material. The presence of geological features such as layering or discontinuities in general can result in failure mechanisms different from those assumed in the theories on which the equations are based. Therefore, the presence of such features, in particular weak soil layers, should be checked in ground investigations. Where appropriate, the evaluation of bearing capacity and the design shear strength parameters should take account of the geological characteristics of the ground.

A.3.1.3   Design Shear Strength Parameters

The design shear strength parameters to be used should be representative of the foundation soil and appropriate to the relevant failure mechanism.   As a guide, the parameters may be based on tests carried out on the foundation soil within a depth approximately equal to the width of the foundation. However, the strength of soils at larger depths should also be tested where weak layers exist. General guidance on the evaluation and selection of shear strength parameters for various types of soils is given in Section 5.5.

Where laboratory tests are used to determine the effective stress shear strength parameters of the foundation soil, the following relevant stress range may be used in planning the testing schedule (Figure 5) :

(a)   for a triaxial compression test, from $p' = 0$ to $0.6\ Q_{ult}/A'$, where p' is the mean effective stress of the test specimens at failure, and

(b)   for a direct shear test, from $\sigma_{nf}' = 0$ to $0.5\ Q_{ult}/A'$, where σ_{nf}' is the effective normal stress of the test specimens at failure.

An estimate of Q_{ult} is required to calculate the relevant stress range.   This should be evaluated based on realistically assumed shear strength parameters.

Field tests such as SPT and CPT may also be used to determine the shear strength parameters (see, for example, Sections 5.5.6 and 5.5.7). Such tests should be carried out within the relevant depth of the foundation soil. The limitations of the empirical correlations used to obtain the parameters should be recognized.

The ultimate bearing capacity can be very sensitive to variations in shear strength parameters, particularly variations in ϕ'. Therefore, the shear strength parameters for design should be carefully selected. Also, the ground model and the design parameters should be reviewed during construction.

A.3.1.4   Effects of Groundwater

The effects of groundwater should be carefully considered in the assessment of ultimate bearing capacity.

Where the groundwater level is at a distance of B' or more below the base of the foundation, γ' in the equation given in Figure A2 can be taken as the bulk unit weight γ. If the groundwater level is at the same level as the foundation base, the effect of water should be considered. For the case of a static groundwater level, γ' may be taken to be the submerged unit weight $\gamma - \gamma_w$, where γ_w is the unit weight of water. Groundwater flow with an upward hydraulic gradient can significantly reduce the bearing capacity of a foundation. In such cases, γ' may be taken to be $\gamma - \gamma_w(1 + i)$, where i is the upward hydraulic gradient.

For intermediate groundwater levels, the ultimate bearing capacity may be interpolated between the above limits.

If the groundwater level is above the foundation base, the effective surcharge q' should be used in the calculations. Similarly, the effective normal load should be used. This can be calculated by subtracting the water force at the foundation base from the total normal load Q_n.

In applying the equation given in Figure A3 in a total stress calculation, account should be taken of any softening which may occur in the foundation upon excavation. Softening is likely to occur for a fine-grained soil which has been 'unloaded' (i.e. experiencing a reduced effective stress) and water is available for swelling to take place. Therefore, where the groundwater level is high, the use of a reduced s_u value should be considered in design.

A.3.1.5   Foundations On or Near the Crest of a Slope

For retaining walls founded on level ground in competent soils, the ultimate bearing capacities are generally fairly high and settlement considerations often govern the design. However, bearing capacity can be a major consideration in cases where the retaining wall is founded on sloping ground. Where possible, retaining walls should be founded well away from the crest of a slope.

For a foundation on a slope, the equations in Figures A2 and A3 can be used to calculate the ultimate bearing capacity. Where the foundation is located close to the crest of a slope, the ultimate bearing capacity may be estimated using the linear interpolation procedure shown in Figure A4. This procedure assumes that the ultimate bearing capacity increases linearly from the edge of the slope to a maximum value at a distance of 4B' from the slope crest. This maximum value is assumed to be equal to the ultimate bearing capacity of a foundation of the same dimensions, placed on level ground at the same depth, and subjected to the same loading conditions.

Both slope stability and bearing capacity should be checked where the retaining wall is founded on or near the crest of a slope. Different soil shear strength parameters should generally be used in the two types of calculations, as the relevant stress levels involved in these two limit states are different. Also, different materials may be encountered, depending on the locations of the critical failure surfaces.

A.3.2   Serviceability Limit State Design

A.3.2.1   General

For gravity and reinforced concrete retaining walls, serviceability limit state criteria are deemed to be satisfied if the wall is proportioned in such a way that the resultant force acts within the middle third of the wall base. However, where sensitive services are connected to or taken through the retaining wall, where the wall is required to be watertight, or where the wall is to be founded on compressible foundations, the checking of

serviceability limit states should be considered.

In checking against serviceability limit states, the foundation loads should be calculated using unfactored loadings and material parameters. The possible range of movements of the foundation should be compared with the specified serviceability criteria. In specifying the serviceability limits, account should be taken of the type of retaining wall adopted and its use, and of the nature of nearby services and structures.

The following factors should be taken into account in calculating foundation movements :

(a) the ground and groundwater conditions, especially spatial variations in soil properties,

(b) the loading distribution,

(c) the method and sequence of construction, and

(d) where appropriate, the stiffness of the wall.

Apart from movements due to earth pressures, water pressures, surcharge loads and the self weight of the wall, movements which may be occurring at the site prior to construction and movements resulting from excavation of the foundation should also be considered.

Where appropriate, both 'immediate' settlement and long-term settlement (due to consolidation and creep) should be assessed. It should be noted that for a retaining wall, a large part of the loading is exerted on the foundation during construction. Any additional foundation movements are due to the imposition of additional loads, consolidation and creep and changes in foundation soil moisture content.

A.3.2.2   Calculation Methods

Only some general guidance is given in this Section on methods of estimating foundation movement. For comprehensive reviews of methods of predicting settlement, reference can be made to Sutherland (1974), Burland et al (1977), Jorden (1977) and Jeyapalan & Boehm (1986). A prime requirement for successful estimation of foundation movement will always be a good knowledge of the soil profile and groundwater conditions across the site. No amount of sophisticated analysis can compensate for a lack of such knowledge.

The modelling of a foundation as a homogeneous isotropic linear elastic material is usually adequate for the purpose of estimating the order of magnitude of foundation movement, provided that appropriate soil deformation parameters have been obtained. However, the assumptions of such a simple model should be carefully scrutinized and a more refined model, with due account taken of anisotropy, non-homogeneity (e.g. layering) and non-linearity due to yielding, should be used where appropriate. Before using a refined model, the designer should consider whether a real problem actually exists and ascertain what

advantages and economy can result from refinements in the prediction. Some guidance on the evaluation of soil deformation parameters is given in Section 5.6.

Based on elasticity theory, the total long-term settlements of a foundation can be calculated using an equation of the following form :

$$s = \frac{p'B'f}{E_s'} \qquad \ldots \ldots \ldots \ldots \ldots \ldots \quad (A2)$$

where p' = mean effective ground bearing pressure
    B' = effective width of foundation
    E_s' = Young's modulus of foundation soil in terms of effective stress
    f = coefficient whose value depends on the shape and dimensions of the foundation area, the variation of soil stiffness with depth, the thickness of the deforming formation, the value of the Poisson's ratio ν_s', the distribution of the bearing pressure and the point for which the settlement is calculated

Poulos & Davis (1974) give a comprehensive list of elastic solutions.

Immediate settlement can also be estimated using elasticity theory. The equation is of the same form as equation (A2), except that the Young's modulus and Poisson's ratio for 'undrained' loading should be used. The effect of foundation embedment is sometimes taken into account.

Apart from the elasticity method, foundation settlement may also be evaluated by using the one-dimensional stress-strain method or by means of empirical correlations. The former involves computing the stress distribution in the ground using elasticity theory, calculating the strains from the computed stresses using soil deformation moduli values obtained from appropriate tests, and then integrating the vertical strains to find the settlement. Empirical methods are often used to estimate settlement in granular materials. These are usually based on field tests such as SPT or CPT (see, for example, Schmertmann, 1970; Parry, 1971 and Burland & Burbidge, 1985).

For fine-grained soils, settlement due to consolidation should also be considered. An estimate of the consolidation settlement can be made using the settlement-time curve obtained from an oedometer test. The rate of consolidation can be assessed using the coefficient of consolidation of the soil (see Section 5.7.2).

For soft cohesive soils which 'yield' upon loading, the simple classical stress-strain method can be used to calculate the consolidation settlements with sufficient accuracy. The immediate settlements are difficult to estimate but in any case they are unlikely to exceed 10% to 15% of the long-term settlement.

Burland et al (1977) found that for stiff cohesive soils which are approximately 'elastic' in their response to vertical loads, the simple classical stress-strain method using the one-dimensional modulus can be used to calculate the long-term settlements as accurately as many of the more sophisticated methods. For these soils, the immediate settlement will

usually be between 1/3 and 2/3 the long-term settlement.

Differential settlement between adjacent structural units (i.e. relative movements at joints) and relative rotations along the length of the wall (i.e. angular distortion) may be evaluated using the elasticity or the stress-strain method mentioned above. However, the resulting estimates tend to be over-predictions if the stiffness of the wall is not taken into account. Also, the actual differential settlement and relative rotations are very much influenced by the construction sequence.

Tilting of the foundation can be estimated by calculating the settlements at the front and rear edges of the foundation, assuming a linear ground bearing pressure distribution. The corresponding settlements can be calculated by using the vertical stress distribution in the ground beneath each location and the stress-strain method. Alternatively, if the foundation is rigid, tilting due to the moment applied to the foundation may be estimated by using the elastic solutions for rigid loaded areas (see Poulos & Davis, 1974).

Ground heave due to excavation for the foundation should be taken into account in evaluating the total settlement. Heave is caused by relief of vertical stress in the soil, as the weight of overburden is removed. The effect is largely elastic and the Young's modulus for unloading should be used in the calculations. The net uplift is practically reduced to zero when a ground bearing pressure equal to that of the original overburden has been applied. Therefore, the total settlement of the foundation should be assessed using the net loading intensity, viz. the maximum ground bearing pressure at the retaining wall base minus the effective overburden pressure at the same level. The former should be calculated by assuming a trapezoidal base pressure distribution, while the latter should be that related to the final formation level in front of the wall. Unfactored loadings and material parameters should be used in the calculations. Where necessary, heave should be calculated specifically. Reference can be made to Perloff (1975) and Zeevaert (1982) for methods of calculating heave due to excavation.

A.4 DESIGN OF FOUNDATIONS IN ROCK

The bearing capacities of the rock masses commonly encountered in Hong Kong are generally much higher than the loading intensities from retaining walls.

Where discontinuities with weak infillings are present in the rock mass, or where the rock is decomposed, disintegrated or altered, the bearing capacity will reduce. Therefore, careful assessment of the rock mass should be carried out during the ground investigation and any adverse features encountered should be taken into account in the design.

Certain types of rock can deteriorate rapidly upon exposure or slake and soften when in contact with water, e.g. weathered shales, sandstones and siltstones. Final excavation in such materials should be deferred until just before construction of the retaining wall is ready to commence. Alternatively, the exposed surfaces should be protected with a blinding layer immediately after excavation (see Section 3.6.2).

For a retaining wall to be founded on or near the crest of a rock slope, the stability of the slope should also be checked, with due account taken of the loading from the retaining

234wall. Limit equilibrium methods outlined by Hoek & Bray (1981) can be used to analyze planar and wedge failures. For more complex two- or three-dimensional discontinuity-controlled failure modes, the methods given by Kovari & Fritz (1984) are appropriate. The Geotechnical Manual for Slopes (GCO, 1984) gives further guidance on rock slope design.

Where possible, a foundation on or near the crest of a rock slope should be constructed at a level below adverse discontinuities in the rock mass. Alternatively, the stability of the rock foundation can be improved by means of dowelling, bolting, etc. For steeply sloping rock surfaces, it is often economical to construct a stepped foundation to reduce the amount of rock excavation.

Where appropriate, the settlement of a foundation on rock can be estimated using elasticity theory (see, for example, Hobbs (1974)). The characteristics of any discontinuities and infillings present in the rock mass, as well as the effects of non-homogeneity, anisotropy and relative scale, should be taken into account in the evaluation of a representative modulus of deformation for the rock mass (see Section 5.9). For example, settlement will be larger for a foundation with horizontal infilled joints than for one with vertical joints. Barton (1986) has proposed a model for assessing the deformation of a rock joint under external loads (see Section 5.9). The model can be extended to estimate the settlement of a foundation. Kulhawy (1978) has also proposed a geomechanical model for estimating the settlement of foundations on rock masses. This model provides a means of assessing the reduction of rock material properties caused by the presence of discontinuities and can be used to provide an estimate of settlement for isotropic, transversely isotropic or orthogonally jointed rock masses.

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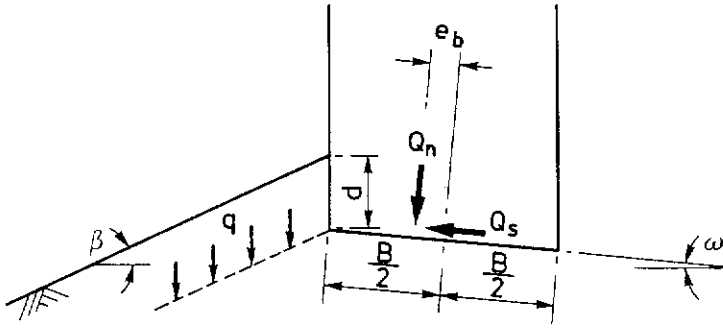
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LIST OF FIGURES

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(a) Forces Acting on Spread Foundation

(b) Effective Dimensions of Foundation Base

Legend :

B' , L'	Effective width and length of foundation base
B , L	Width and length of foundation base
d	Depth of embedment of foundation
e_b , e_l	Eccentricity of resultant load in the B and L directions
Q_n , Q_s	Normal and shear component of resultant load
q	Surcharge
β	Angle of slope in front of foundation
ω	Angle of foundation base tilt

Figure A1 - Generalised Loading and Geometric Parameters for a Spread Foundation

$$q_{ult} = \frac{Q_{ult}}{A'} = c' N_c s_c i_c t_c g_c + 0.5 \gamma' B' N_\gamma s_\gamma i_\gamma t_\gamma g_\gamma + q' N_q s_q i_q t_q g_q$$

where

$$N_c = (N_q - 1) \cot \phi'$$
$$N_\gamma = 2 (N_q + 1) \tan \phi'$$
$$N_q = \exp (\pi \tan \phi') \tan^2 (\frac{\pi}{4} + \frac{\phi'}{2})$$

      } - Bearing capacity factors

$$s_c = 1 + \frac{N_q}{N_c} \frac{B'}{L'}$$
$$s_\gamma = 1 - 0.4 \frac{B'}{L'}$$
$$s_q = 1 + \tan \phi' \frac{B'}{L'}$$

      } - Shape factors

$$i_c = i_q - \frac{1 - i_q}{N_c \tan \phi'}$$
$$i_\gamma = (1 - K_i)^{m_i + 1}$$
$$i_q = (1 - K_i)^{m_i}$$

      } - Inclination factors

$$t_c = t_q - \frac{1 - t_q}{N_c \tan \phi'}$$
$$t_\gamma = (1 - \omega \tan \phi')^2 , \quad \omega \ne 45°$$
$$t_q = t_\gamma$$

      } - Tilt factors

$$g_c = \exp (-2\beta \tan \phi')$$
$$g_\gamma = g_q = (1 - \tan \beta)^2 \quad \text{for } \beta \le 45°$$
$$= 0 \quad \text{for } \beta > 45°$$

      } - Ground slope factors

Legend :

A'	Effective area of foundation base $= B'L'$
K_i	Equals $Q_s /(Q_n + c'A'\cot\phi')$
Q_{ult}	Ultimate resistance against bearing capacity failure of foundation
q_{ult}	Ultimate bearing capacity of foundation
q'	Effective surcharge $= \gamma' d \cos \beta$
m_i	Equals $(2 + B'/L')/(1 + B'/L')$
c', ϕ'	Shear strength parameters of soil in terms of effective stress
γ'	Effective unit weight of soil

For other symbols, see Figure A1

Notes :   (1) The above equation is applicable to the calculation of ultimate bearing capacity of a shallow foundation (with $d \le B$) composed of soils modelled as $c' - \phi'$ materials.

              (2) For a foundation on a slope, the slope stability should also be checked.

Figure A2 - General Equation for the Calculation of Ultimate Bearing Capacity of Spread Foundations on $c' - \phi'$ Soils

$$q_{ult} = \frac{Q_{ult}}{A'} = (\pi + 2) s_u s_c i_c t_c g_c + q$$

where

$$s_c = 1 + 0.2 \frac{B'}{L'}$$ - Shape factor

$$i_c = 0.5 \left(1 + \sqrt{1 - \frac{Q_s}{A' s_u}}\right)$$ - Inclination factor

$$t_c = 1 - \frac{2\omega}{\pi + 2}$$ - Base tilt factor

$$g_c = 1 - \frac{2\beta}{\pi + 2}$$ - Ground slope factor

Legend :

A'	Effective area of foundation base $= B'L'$
Q_{ult}	Ultimate resistance against bearing capacity failure of foundation
q_{ult}	Ultimate bearing capacity of foundation
q	Surcharge $= \gamma d \cos \beta$
s_u	Undrained shear strength of soil
γ	Total unit weight of soil

For other symbols, see Figure A1

Notes :   (1) The above equation is applicable to the calculation of ultimate bearing capacity of a shallow foundation (with d ≤ B) composed of soils modelled as $\phi = 0$ materials.

          (2) For a foundation on a slope, the slope stability should also be checked.

Figure A3 - General Equation for the Calculation of Ultimate Bearing Capacity of Spread Foundations on $\phi = 0$ Soils
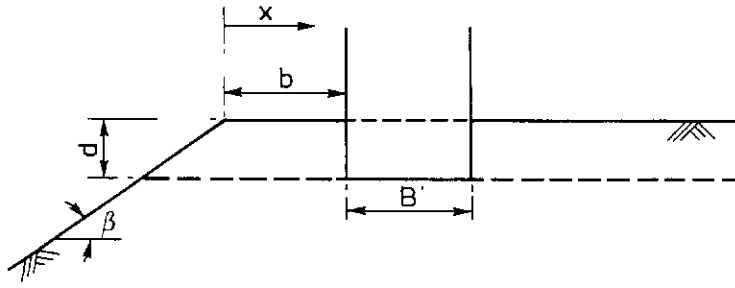
(a) Foundation at a Distance of b from Slope Crest
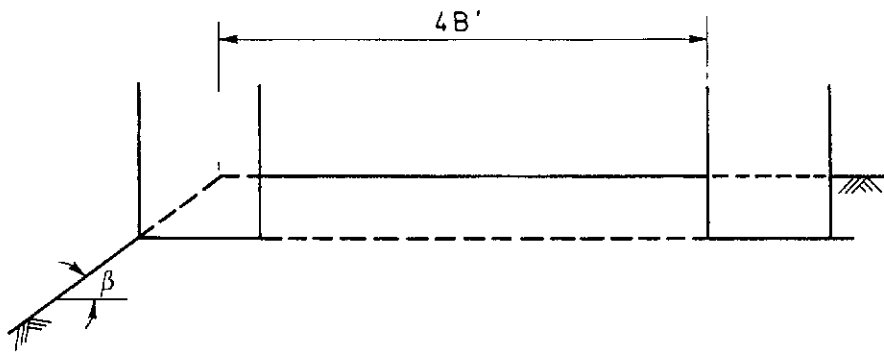
(b) Foundation at the Edge of Slope and at a Distance of 4B' from the Slope Crest
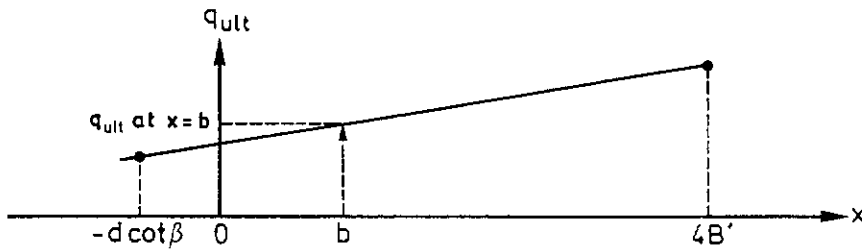
(c) Linear Interpolation of Ultimate Bearing Capacity

PROCEDURE :

1. Calculate the ultimate bearing capacity of a foundation placed at the edge of the slope and at a distance of 4B' from the slope crest. The latter may be assumed to be equal to that of a foundation of the same dimensions, placed on level ground at a depth d, and subjected to the same loading conditions.

2. Interpolate linearly to obtain the ultimate bearing capacity q_{ult} at x = b.

Figure A4 - Linear Interpolation Procedure for the Determination of Ultimate Bearing Capacity of Foundations Near the Crest of a Slope

GLOSSARY OF SYMBOLS

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GLOSSARY OF SYMBOLS

A	cross-sectional area of drainage material
A'	effective area of foundation base
A_1, A_2, etc.	parameters for calculating the coefficient A_g (Table 19)
A_g	a coefficient for calculating the required permittivity ψ of a geotextile (Table 19)
B	width of wall or pile
B'	effective width of foundation base (Figure A1)
B_e	width of excavation
B_k	width of a shear key
b	distance of foundation base from slope crest
C_1, C_2, etc.	parameters for calculating the coefficient C_g (Table 19)
C_g	a coefficient for calculating the maximum filtration diameter O_f of a geotextile (Table 19)
C_u	uniformity coefficient of soil
c', c_1', c_2'	apparent cohesion of soil in terms of effective stress
c_f'	factored cohesion of soil in terms of effective stress
c_j	apparent cohesion of a rock joint
c_v	coefficient of consolidation of soil
D	width of vertical members of cantilevered wall, e.g. diameter of a caisson
D_n	grain size of a soil corresponding to $n\%$ passing in a grading test (Table 18)
D_r	relative density of sand
$D_{10}F$	10% size of a filter material
$D_{15}F_c, D_{15}F_f$	15% size of a filter material, with subscripts c and f denoting the coarse side and the fine side respectively of a grading envelope
$D_{60}F$	60% size of a filter material

$D_{10}S$	10% size of a base soil
$D_{15}S_c$	15% size of a base soil on the coarse side of a grading envelope
$D_{85}S_f$	85% size of a base soil on the fine side of a grading envelope
$D_{90}S$	90% size of a base soil
d	depth of embedment of retaining wall in soil
d_1	depth of the upper part of a rock socket which is acted upon by force F_1
d_2	depth of the lower part of a rock socket which is acted upon by force F_2
d_o	depth of embedment of wall calculated from simplified pressure distribution (Figure 50)
d_r	rock socket depth
d_s	depth of soil cover in the passive zone of a cantilevered retaining wall socketed in rock
E_l	laboratory modulus of deformation of rock material
E_m	modulus of deformation of rock mass
E_o'	one-dimensional modulus of soil
E_s'	Young's modulus of soil in terms of effective stress
E_{sj}'	Young's modulus of soil at depth z_j in terms of effective stress
E_u	Young's modulus of soil for 'undrained' loading
e, e_b, e_l	eccentricity of the normal component of the resultant force on a retaining wall base, with subscripts b and l denoting eccentricities in the direction of the width and length of the base respectively
F	loading
F_1, F_2	reaction force in rock socket on the 'passive' and 'active' side of the retaining wall respectively (Figures 51 to 54)
F_a	activating force (Figure 39)
F_f	factored values of loading F
F_r	resisting force (Figure 39)

f	a coefficient used in calculating total elastic settlement
f_m	strength parameters of structural materials
G_s	specific gravity of rock material
g_c, g_q, g_γ	foundation ground slope factors (Figure A2)
H	height of wall
$\overline{H}$	length of sloping rear surface of retaining wall
H_1, H_2, etc.	thickness of soil layers
H_e	depth of retaining wall over which the pressure generated by a horizontal surface line load is assumed to act (Figure 29)
H_w	net hydrostatic head (Figures 36 and 37)
h_c	critical depth of fill where the lateral pressure due to compaction equals the active or at-rest earth pressure (Figure 27)
h_w, h_{w1}, etc.	depth of water table below ground surface
I_R	a coefficient used in the empirical method of Bolton (1986) for estimating ϕ' of sand (Figure 7)
i	hydraulic gradient
i_a	angle of inclination of active force P_a to the normal to the back of the retaining wall
i_c, i_q, i_γ	foundation inclination factors (Figure A2)
i_r	geometrical component of rock joint friction angle due to roughness of the joint surface
j	mass factor relating E_m to E_l
K	earth pressure coefficient
K_a, K_{a1}, etc.	coefficient of active earth pressure
K_h	modulus of horizontal subgrade reaction
K_{hj}	modulus of horizontal subgrade reaction at depth z_j
K_i	a coefficient used in calculating foundation inclination factors i_c, i_q, i_γ

K_o	coefficient of earth pressure at rest
$K_{o,nc}$	coefficient of earth pressure at rest for level ground composed of normally consolidated soil
$K_{o,oc}$	coefficient of earth pressure at rest for level ground composed of lightly overconsolidated soil
$K_{o\beta}$	coefficient of earth pressure at rest for sloping backfill
K_p	coefficient of passive earth pressure
k, k_1, etc.	coefficient of permeability of soil or rock
k_f	coefficient of field permeability of soil or rock
k_h	coefficient of horizontal subgrade reaction
k_{ho}	coefficient of permeability of soil in the horizontal direction
k_n	coefficient of permeability of geotextile (Table 19)
k_s	spring constant in Winkler model
k_{sj}	spring constant of the jth spring in Winkler model (Figure 9)
k_v	coefficient of permeability of soil in the vertical direction
L	length of foundation base (Figure A1)
L'	effective length of foundation base (Figure A1)
l, l_1, etc.	length of trial failure surface
M	bending moment
$\overline{M}$	modulus ratio of rock
MDD	maximum dry density of soil
M_o	overturning moment
M_r	resisting moment
m	a coefficient used in calculating horizontal force P_h due to surface line or point load
m_h	constant of horizontal subgrade reaction of soil for wall analysis

m_i	a coefficient used in calculating foundation inclination factors i_c, i_q, i_γ
m_v	coefficient of volume change of soil
N	SPT value
N_b, N_s, N_1, N_2	normal reactions on a retaining wall base
N_c, N_q, N_γ	bearing capacity factors (Figure A2)
N_j	SPT 'N' value at depth z_j
n	a coefficient used in calculating horizontal force P_h due to surface line or point load
n_h	constant of horizontal subgrade reaction of soil for pile analysis
n_r	porosity of rock infill in gabion
OCR	overconsolidation ratio of soil
O_f	filtration diameter of geotextile (Table 19)
O_{90}	opening size of geotextile measured by testing the particle size at which 90% by weight of particles are retained on the geotextile upon dry sieving using ballotini (glass beads)
P	reaction force in spring in Winkler model
P_1, P_2, P_3	forces acting on retaining wall due to earth pressure (Figure 25)
P_a, P_{a1}, P_{a2}	forces acting on retaining wall due to active earth pressure
P_{ah}	horizontal component of force P_a
P_{an}	normal component of force P_a
P_{at}	tangential component of force P_a
P_{av}	vertical component of force P_a
P_h	horizontal force acting on retaining wall due to surface, line or point load
P_p, P_p'	forces acting on retaining wall due to passive earth pressure
P_{ph}	horizontal component of force P_p
P_{pn}	normal component of force P_p

P_{pt}	tangential component of force P_p
P_{pv}	vertical component of force P_p
P_{w1}, P_{w2}	water pressures
PLI_{50}	point load index strength equivalent to that for 50 mm diameter rock core
p	contact pressure resulting from reaction forces in springs of Winkler model
p'	mean effective ground bearing pressure
p_a	active earth pressure
p_f	final earth pressures obtained in Winkler model (Figure 55)
p_f'	mean effective stress at failure
p_h'	horizontal effective stress
p_{hm}'	maximum horizontal earth pressure induced by compaction
$p_{o,f}$	locked-in horizontal stresses after bulk excavation but prior to any wall movement (Figure 55)
$p_{o,i}$	earth pressures at rest prior to bulk excavation (Figure 55)
$p_{o\beta}'$	earth pressure at rest in backfill with slope β
p_p	passive earth pressure
Q_h	horizontal line load
Q_l	vertical line load
Q_n	effective normal load imposed on foundation
Q_p	vertical point load
Q_s	effective shear load imposed on foundation
Q_{ult}	ultimate resistance against bearing capacity failure of foundation
q, q_l, etc.	surcharges
q'	effective surcharge
q_a	allowable lateral bearing pressure of rock

q_c	uniaxial compressive strength of rock
q_d	design seepage flow rate
q_f	deviator stress of soil at failure
q_n'	mean effective bearing pressure on the ground from wall foundation
q_u	ultimate lateral bearing capacity of rock
q_{ult}	ultimate bearing capacity of foundation
R, R_1, etc.	reaction or resultant forces
R_a	resultant resisting force acting on a trial failure surface in soil due to active earth pressure
R_d	reduction factor for wall friction in Caquot & Kerisel's charts
R_p	resultant resisting force acting on a trial failure surface in soil due to passive earth pressure
RMR	Rock Mass Rating
RQD	Rock Quality Designation
R_{p1}, R_{p2}	passive resistances of soil
r	radius of circular arc in trial failure surface in soil in the evaluation of passive earth pressure
S	spacing of vertical members of cantilevered retaining wall
S_1, S_2	resisting forces acting against sliding of retaining wall
s	settlement of foundation
s_c, s_q, s_γ	foundation shape factors (Figure A2)
s_u	undrained shear strength of soil
T_g	thickness of geotextile
t_c, t_q, t_γ	foundation tilt factors (Figure A2)
U, U_1, etc.	forces acting on a trial failure surface in soil due to water pressure
U_{1h}	horizontal component of force U_1

U_{1v}	vertical component of force U_1
u, u_1, etc.	pore water pressures in soil
u_f	seepage pressure at the toe of cantilevered retaining wall (Figure 33)
u_c	maximum net water pressure acting on cantilevered retaining wall with seepage
V	shear force
VWFR	volume water flow rate (Table 18)
W, W_1, etc.	weights
X	material parameter
X_a	inter-slice force in soil on the 'active' side of a wall
X_f	factored values of material parameter X
X_p, X_{p1}, X_{p2}	inter-slice force in soil on the 'passive' side of a wall
x	horizontal distance measured from a reference line, e.g. crest of slope
x_p, x_w, etc.	horizontal distances between the toe of a retaining wall and points of action of forces acting on the wall (Figure 39)
y	lateral deflection of a retaining wall
y'	lateral deflection of cantilevered retaining wall at mid-depth of embedment in soil
$\bar{y}$	elevation of the centroid of a pressure diagram
y_p, y_r, etc.	vertical distances between the toe of a retaining wall and points of action of forces acting on the wall (Figures 29 and 39)
z	depth below ground surface
z_c	depth of point of maximum residual compaction pressure below final fill surface (Figure 27)
z_j	depth of spring j in Winkler model (Figure 9)
z_o	vertical depth of tension crack in cohesive soil
α	tilt angle of retaining wall back face to the vertical

α_a, α_{a1}, etc.	angles at which potential failure surfaces made with the vertical
β	angle of ground slope
β_a	angle of slip plane to the horizontal
β_d	angle of inclination of a drain to the horizontal
γ, γ_1, γ_2	saturated unit weights of soil
γ'	effective unit weight of soil
$\bar{\gamma_1}$	unit weight of soil above water table
γ_c	saturated unit weight of clay
γ_d	dry unit weight of soil
γ_f	partial load factor
γ_g	unit weight of gabion wall material
γ_m	partial material factor
γ_s	saturated unit weight of soil
γ_w	unit weight of water
Δ_d	traverse distance in levelling (Table 23)
Δ_h	vertical distance between equipotential lines at the free surface of a flow net
Δ_z	length of wall or pile over which a Winkler spring acts
Δ_{zj}	length of wall or pile served by spring j in Winkler model
ΔH	loss in water head (Table 19)
δ, δ_1, δ_2	angles of wall friction
δ_b	angle of base shearing resistance
δ_{bf}	factored angle of base shearing resistance
δ_{max}	maximum value of mobilised angle of retaining wall friction
δ_s	angle of skin friction between soil and retaining wall material
ϵ	parameter for calculating angles α_a and β_a

θ	dip angle of discontinuity in rock mass
ν_1	laboratory Poisson's ratio of rock material
ν_s	Poisson's ratio of soil
ν_s'	Poisson's ratio of soil in terms of effective stress
ν_u	Poisson's ratio of soil corresponding to 'undrained' loading
σ'	effective stress
σ_h, $\sigma_{h\theta}$	horizontal earth pressure induced by surcharge
σ_n'	effective normal stress
σ_{nf}'	effective normal stress at failure
σ_v'	vertical effective stress
τ	shear stress
τ_f	shear strength of soil or rock joint
ϕ	angle of shearing resistance of soil in terms of total stress
ϕ', ϕ_1', etc.	angle of shearing resistance of soil in terms of effective stress
ϕ_b	basic friction angle of a smooth planar discontinuity in rock
ϕ_{cv}'	critical state angle of shearing resistance of soil in terms of effective stress
ϕ_f'	factored angle of shearing resistance of soil in terms of effective stress
ϕ_t'	angle of shearing resistance of soil obtained from triaxial compression test in terms of effective stress
ψ	permittivity of geotextile (Table 19)
Ω	angle between base of a retaining wall and sliding surface
ω	base tilt angle of retaining wall foundation

GLOSSARY OF TERMS

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GLOSSARY OF TERMS

Best-estimate parameter. Value of parameter which is representative of the properties of material in the field.

Bored pile wall. Retaining wall formed by construction of a row of large diameter machine bored piles, which are either contiguous, or closely spaced with infilled concrete panels in between.

Caisson wall. Retaining wall formed by construction of a row of hand-dug cast insitu reinforced concrete caissons, which are either contiguous, or closely spaced with infilled concrete panels in between.

Cantilever retaining wall. Reinforced concrete retaining wall with a stem which cantilevers from a base slab.

Cantilevered retaining wall. Partially embedded retaining wall, the stability of which is provided by virtue of the bending strength and stiffness of the cantilever and the passive resistance of the ground, e.g. bored pile or caisson wall.

Factored value. Value of geotechnical parameter or loading obtained by factoring the selected value by a partial safety factor.

Gravity retaining wall. Retaining wall, the stability of which is provided by virtue of its self-weight, e.g. mass concrete wall, crib wall, gabion wall and reinforced fill structure.

Lifeline. System which provides essential services to a community, e.g. pumping station, electric and gas transmission lines, water distribution system, sewage treatment and disposal system, telephone network, airport, etc.

Limit state. State at which a retaining wall, or part of it, fails to satisfy any of its performance criteria.

Partial load factor. Partial safety factor which is applied to the selected value of a loading.

Partial material factor. Partial safety factor which is applied to the selected value of a geotechnical parameter.

Partial safety factor. Factor which is applied to the selected value of a geotechnical parameter or a loading to account for the effect of uncertainties in the design of retaining walls.

Reinforced concrete retaining wall. Retaining wall, the stability of which is provided by virtue of the strength and stiffness of the reinforced concrete (R.C.) and the weight of the retained fill, e.g. R.C. L-shaped and inverted T-shaped cantilever, counterfort and buttressed retaining wall.

Residual soil. Soil derived from insitu rock weathering in which all trace of the original rock texture, fabric and structure has been destroyed.

Saprolite. Soil derived from insitu rock weathering in which evidence of the original rock texture, fabric and structure is retained.

Selected value. Value of a geotechnical parameter or a loading which is selected for a retaining wall design and requires factoring by a partial safety factor for a limit state check.

Serviceability limit state. A state at which specified serviceability criteria are no longer met, e.g. excessive strains or movements in a retaining wall which may render the wall unsightly, result in unforeseen maintenance or shorten its expected service life.

Ultimate limit state. A state at which a failure mechanism can form in the ground or in the retaining wall, e.g. sliding of the wall or severe structural damage occurring in principal structural elements.

Worst credible groundwater conditions. Groundwater conditions that would arise in extreme events selected for design, which should be assessed with consideration of catchment area, water recharge potential, drainage etc., and should not be taken as the worst possible ground water condition out of context, e.g. ground water level up to the top of the retaining wall.